Hybrid steel sheet pile cofferdam with soil-cement improved ground for the Westridge Marine Terminal Bulkhead

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ABSTRACT: The Westridge Marine Terminal was developed in the 1950s and is currently undergoing expansion. Initial Terminal development included filling along the Burrard Inlet shoreline to create space for oil-handling and loading facilities. As part of the Terminal construction, the foreshore will be expanded with a new bulkhead. Existing soils underlying the expansion are liquefaction-susceptible fills and marine deposits overlying competent glacially overridden soils. New bulkhead concepts considered included (a) tied-back sheet piles, (b) ground improvement (GI) with sheet pile fascia, and (c) free-standing cellular cofferdams. A hybrid system of cellular cofferdams and GI was ultimately selected and designed. The steel sheet pile cellular cofferdam will consist of 21.1-metre-diameter circular cells connected via arc cells. An approximately 45-m-wide concrete slab will be constructed atop the cofferdam to support new facilities. Initial cofferdam concepts included excavating the loose fill and marine deposits and replacement with select granular fill. In addition, excavation and replacement of existing loose soils with select granular fill within 3 m upland of the cofferdam was needed to reduce the lateral pressures acting on the cofferdam. The cellular cofferdam concept was initially developed using U.S. Army Corps of Engineers methodologies and supplemented by three-dimensional dynamic soil-structure interaction analyses using 475- and 2,475-year return period earthquake ground motions. These analyses indicated that excavation and replacement of the loose soil alone would not provide acceptable lateral deformations under the design earthquake ground motions. These analyses also indicated that the existing soils upland of the cofferdam would be subject to liquefaction and excessive settlement under both the 475-year and the 2,475-year return period earthquake motions. To provide the required bulkhead seismic performance and support the on-land infrastructure, the final bulkhead and foreshore design utilizes a hybrid system of cellular cofferdams and soil-cement GI. The final design is based on installing the cofferdam sheet piles, leaving existing soils within the cells and behind in the foreshore area in place and backfilling to grade as needed, and installing jet grout and deep soil mixing GI to form continuous, interconnected soil-cement panels and struts within the cofferdam cells and the foreshore area behind the cofferdam. Three-dimensional non-linear time history analyses were used for the final performance-based design.

Introduction

The Westridge Marine Terminal (Terminal) in Burnaby, B.C. (see Fig. 1) was developed in the 1950s and is currently undergoing further development as part of the Trans Mountain Expansion Project (TMEP). The initial Terminal construction included filling along the Burrard Inlet shoreline to create new land for berthing and oil-handling facilities. The current expansion plan includes creation of additional land along the foreshore to accommodate new berthing and oil-handling facilities. Additional land will be created by construction of an offshore bulkhead and backfilling upland of the bulkhead structure.

The Kiewit-Ledcor joint venture (KLTP) is responsible for design management and construction of the Westridge Terminal, and the geotechnical design scope for this development is being completed by Shannon & Wilson.

Site geologic and seismic conditions

The existing soils in the expansion area are liquefaction-susceptible fills and marine deposits overlying non-liquefiable, glacially overridden soils. The foreshore is underlain at shallow depths by Vashon Drift deposits consisting of lodgement till, glaciolacustrine silt and clayey silt, and occasional zones of glaciofluvial deposits of sand and gravel. These glacially consolidated materials are very dense to hard and are overlain by Capilano sediments consisting of marine and glaciomarine shell-rich silt, sand, and occasional gravel deposits. The Capilano unit varies in thickness, but it is typically less than 5 m thick. Fill material overlies the marine deposits in the foreshore area and consists of a mixture of sand, silt, gravel, and cobbles. The fill contains wood debris, boulders, and other potential obstructions to planned construction at the site.

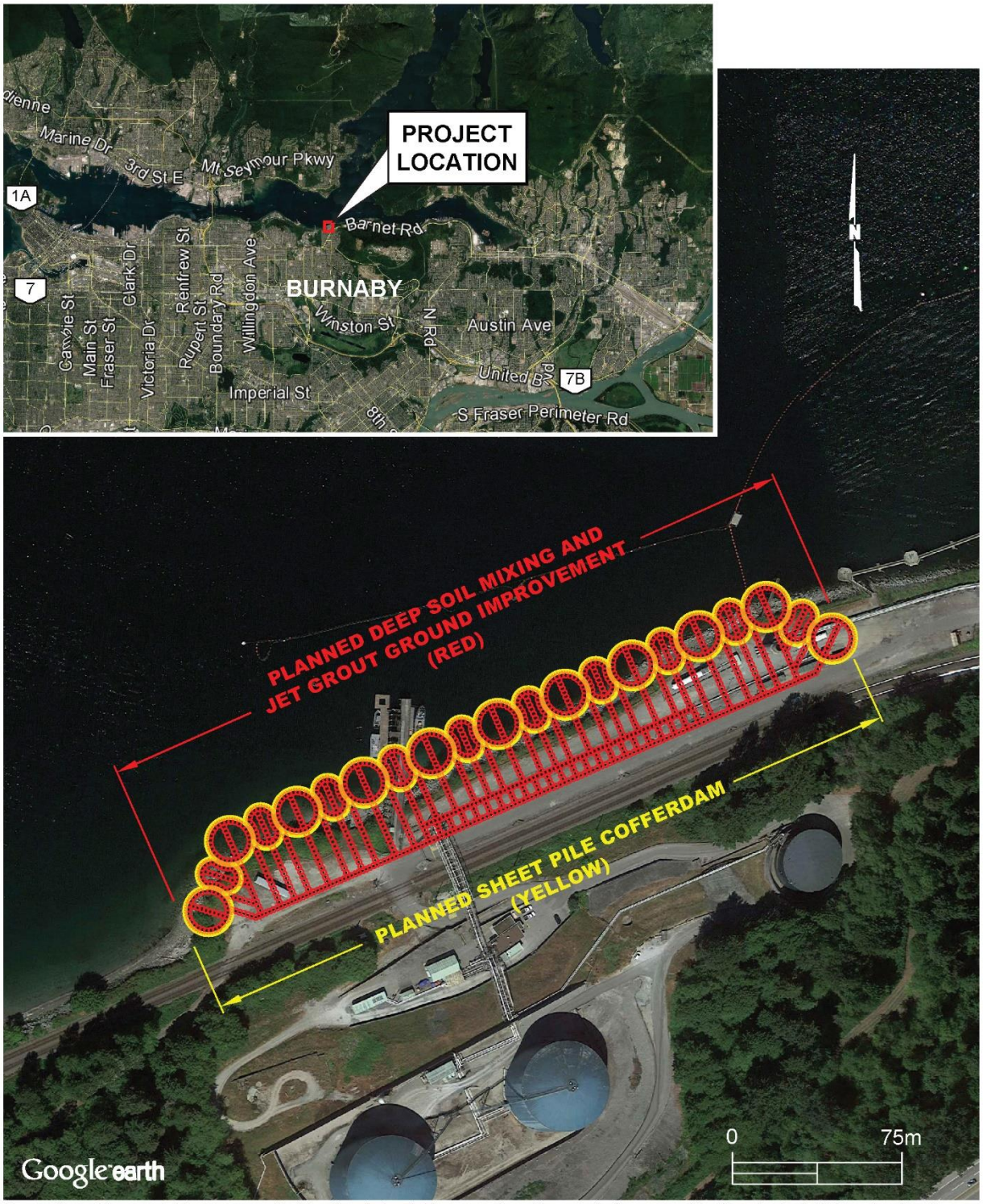
The TMEP site is in a tectonic convergent zone where the northeast-directed, oceanic Juan de Fuca plate is subducting beneath the continental North America plate. This active fault zone, the Cascadia Subduction Zone (CSZ), extends offshore from central Vancouver Island south to northern California and has generated earthquakes up to

Mw 9.0. In the Greater Vancouver area, this complex tectonic setting produces earthquakes in three zones:

1. The *Interface* between the subducting and overriding plates in the CSZ (depths to more than 70 km at site),
2. The *Intraslab* where the subducting plate bends as it moves beneath the North America plate (depth of 50 to 70 kilometers at site), and
3. The *Crustal* zone where compression in the overriding plate produces shallow earthquakes (depth less than 35 km at site).

Moment magnitude on the Richter scale for the *Intraslab* and *Crustal* earthquakes is about 7.0, and about 9.0 for the *Interface* earthquakes.

Figure 1. Site and Planned Hybrid Cofferdam / Ground Improvement



Maps adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

Seismic design criteria

It was realized early in the design process that the design would be governed by the seismic performance criteria (summarized in Table 1). The ground motions are based on the 2015 *National Building Code of Canada* (NBCC) or the 2010 NBCC, whichever is more demanding.

Table 1. Seismic Performance Criteria

Earthquake Ground Motion Level	Probability of Exceedance	Return Period	Duration to Return to Full Service	Damage Level
Operating Level (OLE)	10% in 50 years	475 years	Almost Immediate	Minimal
Contingency Level (CLE)	5% in 50 years	975 years	Many Months	Repairable; defined structural performance
Design Level (DLE)	2% in 50 years	2,475 years	Several Years	Extensive; no collapse or significant loss of containment

Bulkhead and ground improvement design concepts

Bulkhead design concepts

Several bulkhead concepts were considered, including (a) tied-back sheet piles, (b) ground improvement with sheet pile fascia, and (c) free-standing cellular cofferdams. A collaborative process among the owner, designers, and contractors was used to select and design a hybrid bulkhead system using cellular cofferdams and ground improvement (see Fig. 1).

The initial cofferdam construction concept consisted of driving interlocking sheet piles, excavating the loose fill and marine deposits in the completed cofferdam cells, and backfilling the cofferdams with select granular fill to maintain stability of the cofferdams. Excavation and replacement of existing soils upland of the cofferdams was also required to reduce lateral pressures acting on the cofferdams.

The cellular cofferdam design concept of 21.1-m-diameter cells with connecting arc cells was initially analyzed with U.S. Army Corps of Engineers methodologies (U.S. Army Corps of Engineers, 1989).

Subsequently, the cellular cofferdam concept was analyzed with a three-dimensional, dynamic soil-structure interaction program using 475- and 2,475-year return period design earthquake ground motions. The dynamic analyses indicated that excavation and backfilling alone would not provide acceptable lateral deformations under the design earthquake ground motions. The dynamic analyses also indicated that the existing soils upland of the cofferdam would liquefy, resulting in excessive upland settlement under the 475-year and the 2,475-year return period design earthquake motions.

Ground improvement design concepts

To provide the required bulkhead seismic performance and support for the upland processing and loading facilities, we designed a hybrid system of cellular cofferdams and soil-cement ground improvement (Fig. 1). The final design concept consisted of driving the interlocking cofferdam sheet piles, leaving in-place existing soils in the cells and upland of the cofferdams, and backfilling to grade as needed. Jet grout and deep soil mixing ground improvement would then be used to form continuous, interconnected soil-cement panels and struts in the cofferdam cells and in the area upland of the cofferdams.

The ground improvement in the cofferdam cells would increase the strength and stiffness of the cofferdam, thereby reducing permanent ground displacements. Ground improvement in the upland area would reduce the seismic lateral loads acting on the cofferdam, further reducing the bulkhead and foreshore permanent ground displacements. The ground improvement would also reduce potential liquefaction of the existing soils between the interconnected soil-cement panels and struts, and limit settlement upland of the cofferdams. The new facilities would be supported directly on the soil-cement panels and struts to meet the seismic design criteria.

Bulkhead and ground improvement seismic analysis

Analysis of liquefaction susceptibility

Soil liquefaction susceptibility of existing soils in the foreshore area was assessed using the semi-empirical procedures of Boulanger and Idriss (2014) and the results of Cone Penetration Test (CPT), Standard Penetration Test (SPT), and laboratory Atterberg Limits Test results obtained from probes and borings completed in the foreshore area. Atterberg Limits criteria by Boulanger and Idriss (2006) and Seed et al. (2003) were used to evaluate the liquefaction susceptibility of fine-grained marine soils.

Liquefaction-susceptible soils are typically loose, saturated, cohesionless soils. However, some fine-grained soils exhibit cohesionless or “sand-like” behavior. The liquefaction susceptibility of the soils at the Terminal expansion site can be characterized as follows:

- **Fill – Very Low to High**. Where the fill was too dense to probe with a CPT, the fill does not appear to be susceptible to liquefaction. However, loose zones within the fill that could be probed with the CPT typically exhibited a factor of safety against liquefaction (FS-L) of less than 1.0 for even OLE.
- **Coarse-Grained Marine Deposit – High**. The cohesionless, fine-grained portions of this deposit with sand-like behavior typically have an FS-L of less than 1.0 for the OLE ground motion.

- **Fine-Grained Marine Deposit – Very Low**. The utilized semi-empirical liquefaction procedures suggest that fine-grained soils with a plasticity index of less than 7 to 12 exhibit cohesionless or "sand-like" behavior. Based on the reported plasticity indices ranging from 8 to 38, this unit does not appear to be susceptible to sand-like liquefaction behavior.
- **Basal Marine Deposit – Low**. The CPT soundings frequently encountered refusal in this unit indicating it is sufficiently dense to preclude liquefaction for DLE ground motions. This conclusion is consistent with the recorded high SPT N-values.
- **Glacial Soils – Very Low**. CPT soundings were typically unable to penetrate these soils, indicating they are very dense/hard, and therefore not susceptible to liquefaction even for DLE ground motions. SPT N-values in the borings also indicate these soils are typically very dense/ hard, and therefore are not susceptible to liquefaction.

Dynamic, three-dimensional numerical analysis

The detailed analyses completed for the Terminal bulkhead and upland fill design consisted primarily of three-dimensional finite difference modeling to predict the performance of the proposed structures under dynamic loading conditions. The software tool used for the analysis was FLAC3D 5.01 (Itasca, 2016). A dynamic analysis consists of modeling the site soils and structures in a static, pre-earthquake condition, and then applying one or more earthquake loads to observe the performance of the structures and soils during and after shaking. The selected static and dynamic soil properties used for the analysis are based on the subsurface conditions inferred from subsurface explorations and laboratory test results.

The initial cofferdam design concept of installing sheet piles, dredging loose and soft soils from the interior and upland of the cofferdams, and backfilling the cells and the area upland of the cofferdams to grade with select fill was analyzed using several time histories representative of 475-year and 2,475-year return period ground motions. Results of these analyses indicated that the initial design concept would not be acceptable, because of excessive lateral deflection of the cofferdams and excessive settlement in the area upland of the cofferdams.

An alternative cofferdam design concept was then considered that consisted of installing the sheet piles, backfilling the cells to grade without dredging the existing soils from within the cofferdam cells, and installing ground improvement. Approximately 75 cases were analyzed to evaluate various ground improvement layout and strength scenarios with respect to acceptable cofferdam lateral displacement and ground settlement. The objective was to optimize the ground improvement with respect to cost, constructability, and performance of the cofferdam/ground improvement system.

The final cofferdam/ground improvement design concept was analyzed using a suite of seven representative 475-year return period earthquake time histories and seven representative 2,475-year return period time histories as to guidelines in ASCE 61-14. Seven time-histories were used for each return period to allow subsequent design calculations to be based on the mean or geometric mean of the dynamic analysis results. The seven time-history set was comprised of 2, 2 and 3 *Intraslab*, *Crustal* and *Interface* ground motions, respectively.

The geologic cross section used in the analyses was based on subsurface profiles and cross sections. The water-side and land-side toe of the cofferdam was located at about elevation -13 m and -7.4 m, respectively. This generally represents a worst-case condition corresponding to the tallest cofferdam.

Groundwater on the upland side of the structures was assumed to be at elevation +1 m, and the water level in the bay was assumed to be at elevation -3.0 m. As shown in Fig. 2, the soil profile consists of existing fill soils and marine sediments overlying glacially overridden soils. The estimated range of soil properties at the project site is presented in Table 2. Engineering judgement was used to select the best estimate of soil properties (Table 3) to be used in the numerical analyses.

Figure 2. Numerical Model Mesh

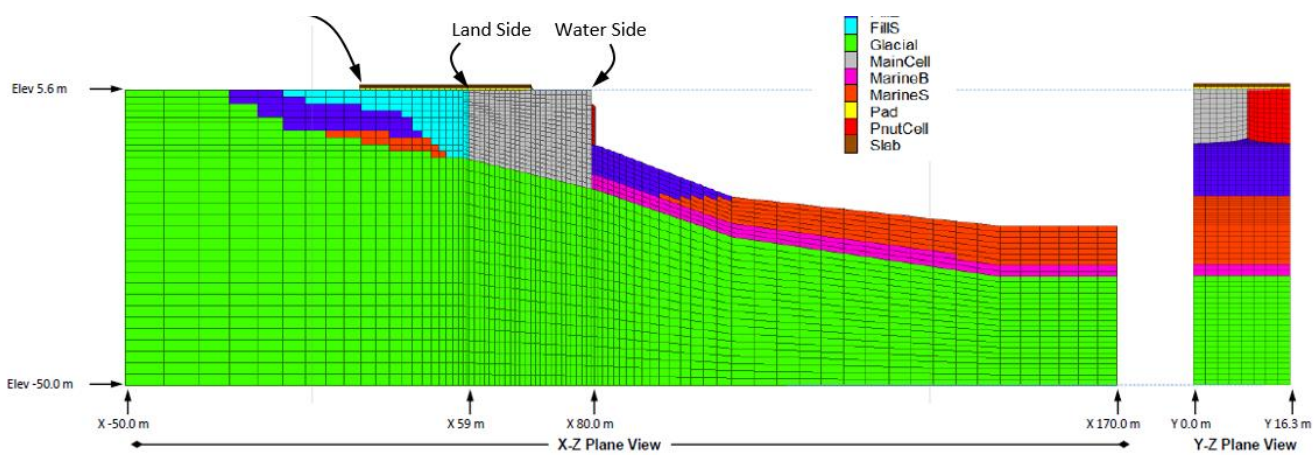


Table 2. Estimated Range of Soil Properties

	Marine Soil		Basal Marine		Glacial Soil		Backfill		Existing Fill	
Unit Weight, kg/m^3	1680	1840	1680	1760	2000	2080	1760	2240	1760	1920
Friction Angle, deg.	30	32	32	32	32	35	40	45	30	33
Cohesion, kPa	0	0	0	0	0	5	0	0	0	0
Poisson's Ratio	0.20	0.20	0.20	0.20	0.30	0.30	0.20	0.20	0.20	0.20
Elastic Modulus E, kPa	$1.3x10^4$	$4.1x10^4$	$3.5x10^4$	$1.8x10^5$	$1.2x10^5$	$3.6x10^5$	$6.5x10^4$	$7.7x10^4$	$5.7x10^4$	$6.7x10^4$
Bulk Modulus K, kPa	$7.3x10^3$	$2.3x10^4$	$1.9x10^4$	$1.0x10^5$	$1.0x10^5$	$3.0x10^5$	$3.6x10^4$	$4.3x10^4$	$3.2x10^4$	$3.7x10^4$
Shear Modulus G, kPa	$5.4x10^3$	$1.7x10^4$	$1.4x10^4$	$7.5x10^4$	$4.6x10^4$	$1.4x10^5$	$2.7x10^4$	$3.2x10^4$	$2.4x10^4$	$2.8x10^4$

Table 3. Modeled Static and Dynamic Soil Properties

	Marine Soil		Basal Marine		Glacial Soil		Backfill		Existing Fill	
	Static	**Dynamic**	**Static**	**Dynamic**	**Static**	**Dynamic**	**Static**	**Dynamic**	**Static**	**Dynamic**
Unit Weight, kg/m^3	1700	1700	1700	1700	2000	2000	1800	1800	1800	1800
Friction Angle, deg.	30	30/22[(1)]	32	32	32	35	40	30/22[(1)]	30	30/22[(1)]
Cohesion, kPa	0	0	0	0	0	5	0	0	0	0
Poisson's Ratio	0.20	0.20	0.20	0.20	0.30	0.30	0.20	0.20	0.20	0.20
Elastic Modulus E, kPa	1.3×10^4	1.4×10^5	3.5×10^4	6.0×10^5	1.2×10^5	1.2×10^6	6.5×10^4	2.6×10^5	5.7×10^4	2.2×10^5
Bulk Modulus K, kPa	7.3×10^3	7.6×10^4	1.9×10^4	3.3×10^5	1.0×10^5	9.9×10^5	3.6×10^4	1.4×10^5	3.2×10^4	1.2×10^5
Shear Modulus G, kPa	5.4×10^3	5.7×10^4	1.4×10^4	2.5×10^5	4.6×10^4	4.6×10^5	2.7×10^4	1.1×10^5	2.4×10^4	9.3×10^4

Note: [(1)] Friction angle above and below water table for soils that are assumed to liquefy below the water table during earthquake loading.

A mesh of approximately 61,000 quadrilateral zones was created to represent the soil and cofferdam structures. The mesh length (X-axis) is 170 m and the vertical extent (Z-axis) of the model is from elevation -50.0 m at the base to elevation +5.6 m at the ground surface at the southern edge of the mesh. The mesh is 16.3 m wide (Y-axis), extending from the centerline of the cofferdam main cell to the centerline of the cofferdam arc cell, based on the assumption that plane-strain conditions exist in the X-Z plane through the centerlines of the cofferdam main and arc cells during static or dynamic loading.

The at-grade foundation slab in the TMEP model was represented with continuum elements and a purely elastic constitutive equation. Ground improvement (deep soil mixing or jet grouting) was simulated by modifying the properties of the soil zones in the numerical model at proposed improvement locations upland of the cofferdam and inside the cofferdam cells. The ground improvement was assumed to extend from the top of the glacially overridden soils to the proposed final ground surface elevation. The soils to be improved are existing marine sediments as well as existing and planned granular fill soils. Properties of the improved soil were estimated based on the assumed soil type and a target shear strength. Properties of the improved soil that were used in the analyses are listed in Table 4.

Table 4. Modeled Improved Ground Properties

	Marine Soil	Basal Marine	Cofferdam Backfill	Existing Fill
Unit Weight, kg/m^3	2080	2080	2080	2080
Friction Angle, deg.	0	0	0	0
Cohesion, kPa	690	690	690	690
Tension, kPa	34	34	69	34
Poisson's Ratio	0.20	0.20	0.20	0.20
Elastic Modulus E, kPa	2.6×10^5	7.2×10^5	7.2×10^5	3.4×10^5
Bulk Modulus K, kPa	1.4×10^5	3.8×10^5	3.8×10^5	1.8×10^5
Shear Modulus G, kPa	1.1×10^5	3.0×10^5	3.0×10^5	1.4×10^5

The sheet piles of the cofferdam structure were modeled using FLAC3D's liner element. A liner element provides the structural behavior of a shell with resistance to bending and membrane forces. A FLAC3D liner element also provides a shear interface between the structure and the soil on either side of the liner element. The properties used for the sheet piles, sheet pile interlocks, and soil-steel interface are listed in Table 5.

The behavior of the sheet pile interlocks was modeled by adding additional nodes between adjacent sheet pile liner elements. The additional nodes were defined to simulate the tangential tensile capacity and the vertical sliding resistance of the interlocks. Liner element (sheet pile) and liner-soil interaction properties are listed in Table 5.

Soils in the TMEP model were assigned a Mohr Coulomb constitutive equation that defines a material stress-strain relationship as purely-elastic-purely-plastic. Static and dynamic soil properties used in the analyses are listed in Table 3.

Initial stress state modeling

Before performing dynamic analyses, it is necessary to develop the model with a stress state that approximates conditions in the soil and structural components immediately prior to the dynamic event. For the TMEP model, this included establishing a stress state that approximates the existing undeveloped site conditions, and then simulating construction and backfilling of the cofferdam cells, backfilling the area upland of the cofferdam, constructing the improved ground, and finally adding at-grade structures and loads. The sequence of steps to develop an initial stress state are illustrated in Fig. 3.

The first stage of cofferdam construction consisted of adding liner elements to the model; the second stage consisted of backfilling the cofferdam main cell. This was accomplished by incrementally adding soil zones to the interior of the main cell in eight 2.0 to 2.3 m thick "lifts." The model was run to static equilibrium after each lift was added. The third stage consisted of backfilling the cofferdam arc cell. This was accomplished using the same incremental procedure that was used for the cofferdam main cell. Placement of fill upland of the cofferdam was simulated by adding soil zones to the model to raise the ground surface to elevation +5.6 m everywhere upland of the cofferdam.

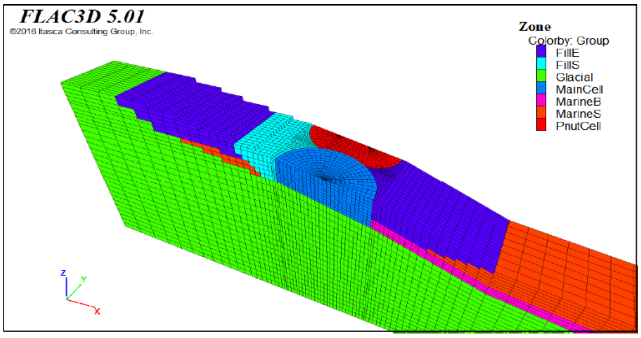
Construction of ground improvement was simulated by changing the properties of the soils above the glacially overridden soils to properties deemed appropriate for each soil type. An at-grade granular soil zone and the overlying concrete slab were added to the model by adding continuum soil and concrete zones to the ground surface of the model. A surcharge of 30 kPa was then applied to the surface of the slab to model loads from permanent structures and facilities to be constructed on top of the slab.

Table 5. Sheet Pile Properties

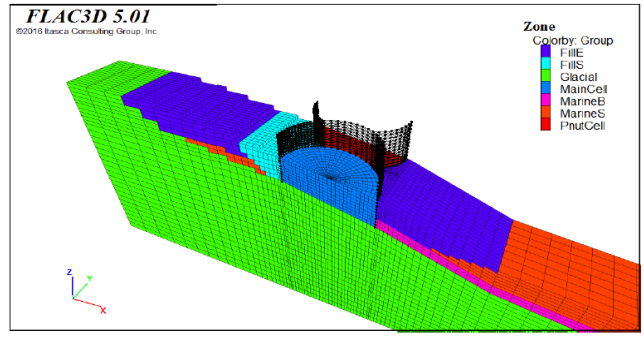
Steel Sheets	**Nominal Thickness, mm**	12.7
	Unit Weight, kg/m^3	7850
	Poisson's Ratio	0.3
	Elastic Modulus E, kPa	2.0 X 10^8
	Shear Modulus G, kPa	1.2 X 10^8
Interlocks	**Friction Angle, deg.**	17
	Tensile Strength, kPa	3.8 X 10^5
Steel-Soil Interface	**Friction Angle, deg.**	20
	Cohesion, kPa	0
	Shear Stiffness, kPa/m	1.0 X 10^8

Figure 3. Steps to Initial Stress State

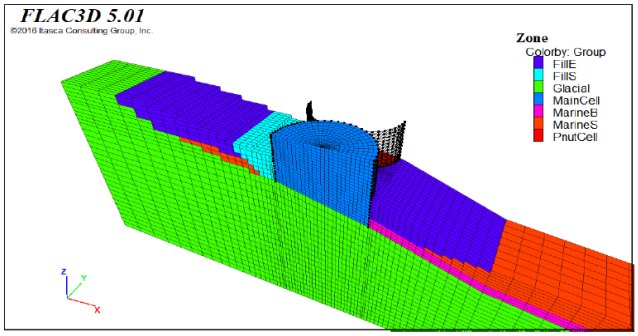
Step 1: Existing Condition



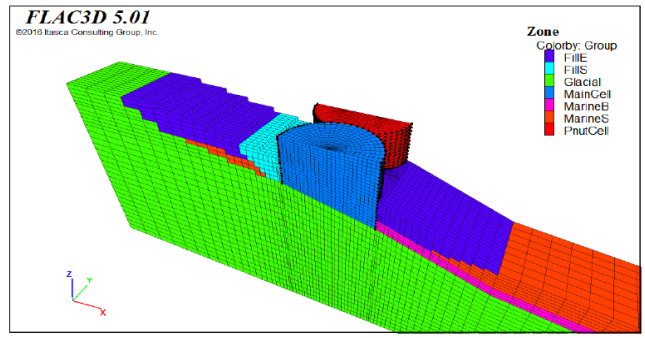
Step 2: Sheet Installation



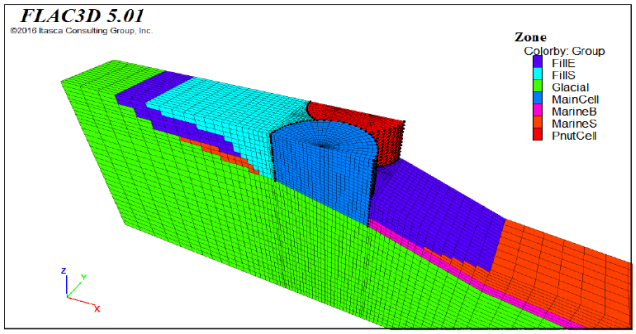
Step 3: Backfill Main Cell



Step 4: Backfill Arc Cell



Step 5: Backfill Upland of Cells



After the cofferdam construction was complete in the model, dynamic loading was accomplished by applying a time history of velocity to the nodes at the base of the model. Liquefaction of the granular marine sediments and fill soils beneath the water table was simulated by setting the shear strength and elasticity properties of these soils to values equal to the liquefied properties of the soils. The liquefied properties and distribution were estimated from an evaluation of the available soil data and two-dimensional numerical models of the site.

The liquefied soil properties were set at the beginning of the input time history because an appropriate three-dimensional soil liquefaction constitutive equation is not available. Liquefaction typically does not occur immediately in an earthquake; however, modeling liquefaction from the beginning of the earthquake event is a conservative approach that did not appear to significantly impact the results of the dynamic analyses.

Deflection during cofferdam construction

The maximum differential deflection of the cofferdam (land side to water side at the centerline of the cofferdam main cell) due to cofferdam construction and upland fill placement was estimated to be about 10 to 20 mm. Deflections of the arc cell and soils below the arc cell were also about 10 to 20 mm.

Dynamic analysis results

Horizontal displacement

Results of the dynamic analyses indicate that lateral movements (horizontal displacement toward Burrard Inlet) occur up to about 70 to 75 m inland from the water side of the cofferdam. Horizontal displacement versus depth was evaluated at three locations (water side of the cofferdam, land side of the cofferdam, and at the back of the slab). The results for the 2,475-year return period ground motions are shown in Fig. 4. Minimum, maximum, and geometric mean displacements for 475-year and 2,475-year return period ground motions are listed in Table 6.

Figure 4. Horizontal Displacement Profiles, 2,475-Year Return Period

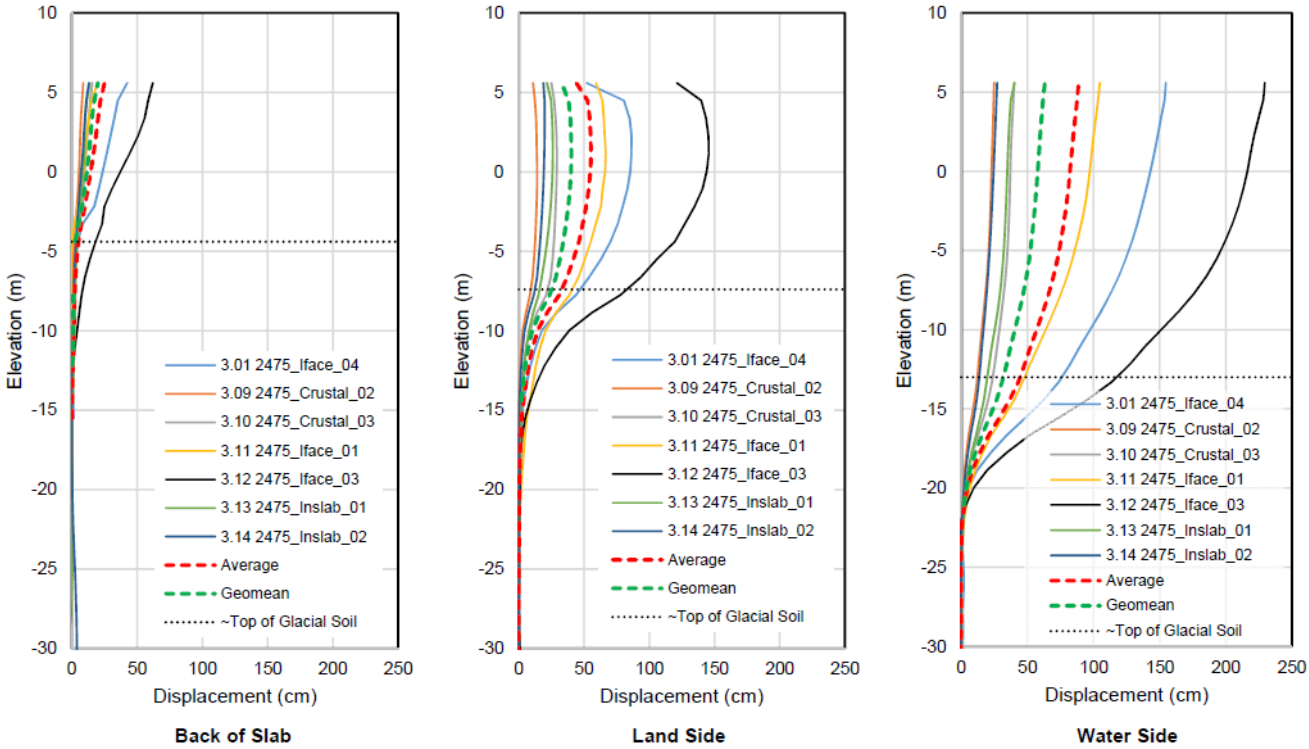


Table 6. Dynamic Analysis Results

		Earthquake Return Period (yrs.)	
Cofferdam Performance		**475**	**2,475**
Horizontal Displacement at Water-Side Face (cm)	Minimum	11	25
	Maximum	55	229
	Geo. Mean	22	63
Horizontal Displacement at Land-Side Face (cm)	Minimum	7	14
	Maximum	37	146
	Geo. Mean	15	40
Change in Cell Diameter (mm)	Minimum	40	80
	Maximum	250	1080
	Geo. Mean	80	300

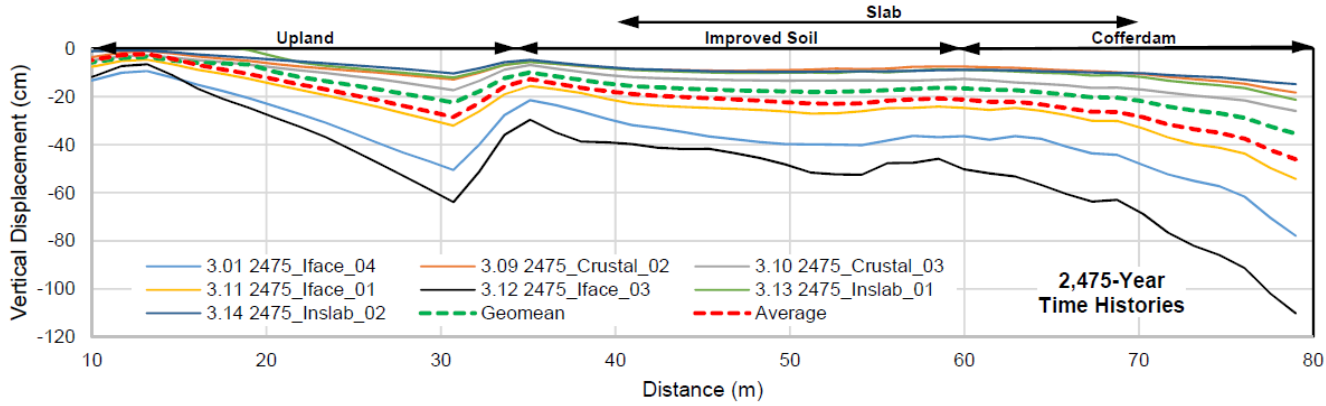
The maximum horizontal displacements at the land side of the cofferdam occur 3 to 5 m below subgrade level, because of the resistance created by the at-grade slab and surcharge loading. The profiles of displacement at the water- and land-side faces of the cofferdam generally indicate sliding displacement of the structure and in the soil below the cofferdam. Displacement of the soil beneath the cofferdam generally diminishes to negligible at 5 to 10 m depth, which corresponds to the top of the glacially overridden soil.

The differential displacement (change in diameter) between the water and land side of the cofferdam is also listed in Table 6. The change in diameter of the main cofferdam cell is less than one percent of the nominal diameter of 21 m for the 475-year return period ground motions, and 1 to 5 percent for the 2,475-year return period ground motions.

Settlement

Settlement (vertical displacement) of the cofferdam structure and the ground upland of the cofferdam occur because of lateral displacement of the cofferdam and liquefaction of the unimproved soils. A plot of settlement profiles for the 2,475-year return period ground motions is shown in Fig. 5. Settlement occurs upland from the face of the cofferdam approximately the same distance (70 to 80 m) as the lateral displacements.

Figure 5. Settlement, 2,475-Year Return Period



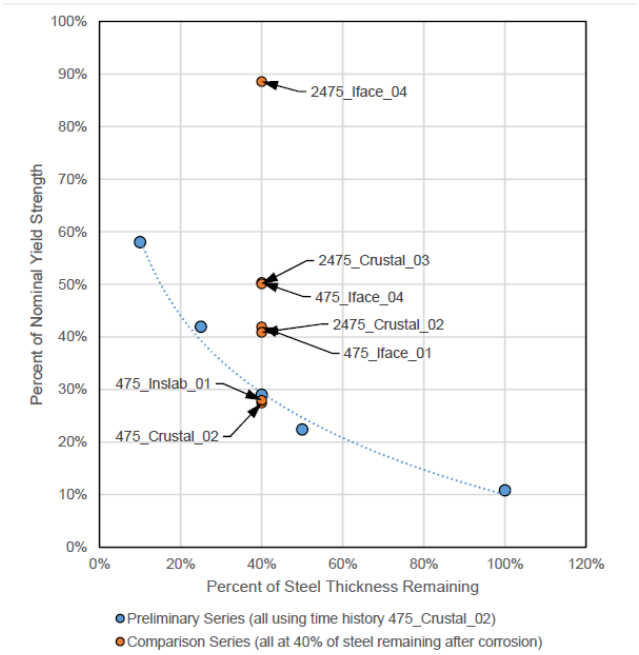
Sheet pile tensile stress

Stresses in the steel sheets and sheet pile interlocks were evaluated for each dynamic case. In general, with a few localized exceptions, the calculated sheet pile tensile stresses were lower than the nominal yield strength (380 MPa) of the steel. The exceptions to this, however, are higher tensile stresses greater than the maximum yield stress of 380 MPa at the top and bottom edges of the sheets at the land side of the cofferdam. The higher tensile stresses appear to be due to bending of the sheets as they slide beneath the surcharged at-grade slab, and as they slide on the glacially overridden soils.

Corrosion effects

The effects of potential corrosion of the steel sheet piles was analyzed by performing several dynamic cases with less than full thickness of steel throughout the cofferdam. The corrosion conditions analyzed were likely more severe than may occur. Corrosion would be limited to certain areas of the cofferdam and would not be uniform. Corrosion studies indicated that the maximum corrosion over the 50-year service life of the facility could result in a remaining sheet pile thickness of about 55 percent (about 7 mm) of the sheet's initial thickness (12.7 mm). As shown in Fig. 6, the maximum sheet and interlock stress increases as the steel sheet thickness remaining after corrosion decreases. In general, the calculated tensile stress of the sheets was less than the nominal yield strength of the sheet piles assuming that 40 percent of the sheet pile thickness (less than the estimated 55 percent) remains under the design-level earthquake ground motion (2,475-year return period). Considering that the steel sheet piles generally provide protection for the soil-cement from wave erosion and freeze-thaw action rather than structural support, corrosion of the sheets would not adversely impact the performance of the cofferdam/soil-cement bulkhead.

Figure 6. Percent of Yield Strength vs. Percent of Steel Thickness After Corrosion



Observations and Conclusions

The calculated displacement and stresses from the dynamic, three-dimensional numerical modeling show that the hybrid cofferdam/ground improvement design is seismically stable. To meet seismic performance criteria for 2010 and 2015 NBCC ground motions, the displacements also provide the basis for displacement-based seismic design of infrastructure located on and adjacent to the bulkhead and GI area.

Lateral movement at the face of the cofferdam is limited to an average displacement of about 22 cm under the 475-year return period ground motions and about 63 cm under the 2,475-year return period ground motions. For the 475-year ground motions, 10 to 12 cm of lateral displacement occurs upland of the GI area, 4 to 6 cm in the GI area, and 7 to 9 cm across the cofferdam structure. For the 2,475-year ground motions, 12 to 15 cm of lateral displacement occurs upland

of the GI area, 10 to 15 cm in the GI area, and about 30 cm across the cofferdam structure.

The maximum vertical displacement for the 475-year ground motions range from about 4 to 14 cm with a maximum geometric mean displacement of about 7 cm. The maximum geometric mean slope for the 475-year ground motions is approximately ±0.3 percent. For the 2,475-year ground motions, the maximum vertical displacement ranges from 10 to 60 cm with a maximum geometric mean displacement of about 20 cm. The maximum geometric mean slope for the 2,475-year ground motions is about -0.5 percent.

The analyses also show that the stresses in the sheet piles do not exceed the nominal shear strength of the sheet pile or interlocks even under long-term corrosion conditions.

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