Seismic evaluation and retrofit of a major electrical transmission crossing at Annacis Island

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Abstract: BC Hydro has been implementing seismic upgrades on components critical to the electrical system for the last 12 years. This paper offers a case history of the most recent upgrade of a double circuit 230kV transmission crossing of the South Arm of the Fraser River at the west tip of Annacis Island, BC. The upgrade included the reinforcement of the original timber piles with 30 metre steel pipe piles at four structures. These new piles were designed to accommodate as much as 3 metres of displacement. The ground surrounding Tower 6/1 at the west tip of Annacis Island is susceptible to liquefaction-induced flow sliding. As a result, a ground improvement program was completed to provide a 'donut' of densified soil around the tower. Approximately 600 stone columns were installed both on land and under water using the dry bottom feed method. The Annacis Island site has been eroding at a rate of 3-5 metres/year. During the design stage, about half of the site was in the river, by the construction stage that had grown to about two thirds. This area had to be reclaimed and about 17,000 cubic metres of riprap was installed over the improved soils as an erosion protection berm.

Introduction

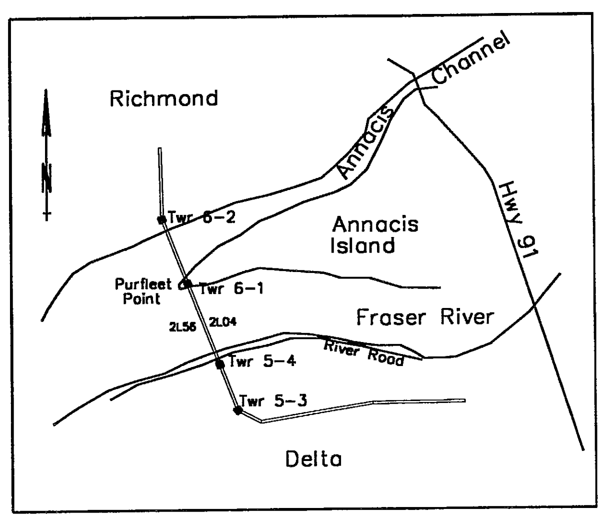
In 1989, BC Hydro's transmission towers were prioritized for upgrading according to seismic exposure, system importance, and local susceptibility to seismic damage.

World wide historical performance of overhead transmission lines under earthquake loads has been very good. BC Hydro transmission towers have been designed for large lateral loads due to wind and ice. As a result when individual towers have been destroyed by an avalanche or debris flow, it has rarely caused an adjacent tower to fail or a cascading failure of several towers to occur as has happened in many other areas of North America.

Most seismic related problems on overhead lines occur as a result of differential soils movement resulting from liquefaction affecting one or more of the tower legs. Tall lattice steel river crossing towers in British Columbia's Lower Mainland which are critical to the electrical grid have been identified as the most susceptible to this kind of damage. Circuits 2L04/56 have been identified as critical to system reliability. BC Hydro does not store spare towers of the heights required for navigable river crossings. Therefore the outage times caused by a failure of a river crossing tower could be in the order of months as the tower steel is procured, fabricated and assembled. In addition, the specialized equipment required to erect or repair the tower may not be available or able to access a site in a post earthquake scenario.

Failure of standard structures within the BC Hydro system have not been considered as serious since these towers can be replaced relatively quickly assuming crews and equipment are available. Alternatively, they may be replaced temporarily with woodpole structures.

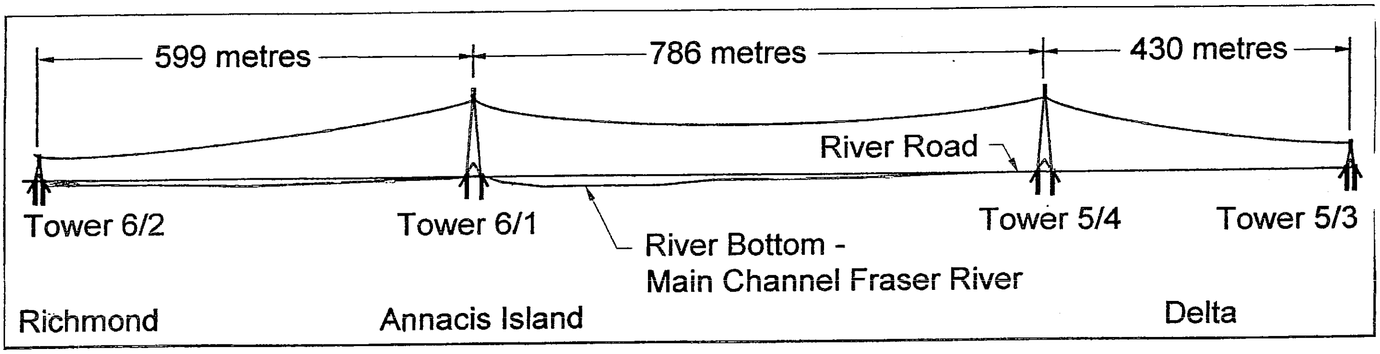
Fig. 1. Key plan of Fraser River Seismic Upgrade Project.



The two 230 kV Circuits 2L04/56 are supported on double circuit towers and cross the South Arm of the Fraser River from Delta, across the west tip of Annacis Island to Richmond, BC. See Fig. 1.

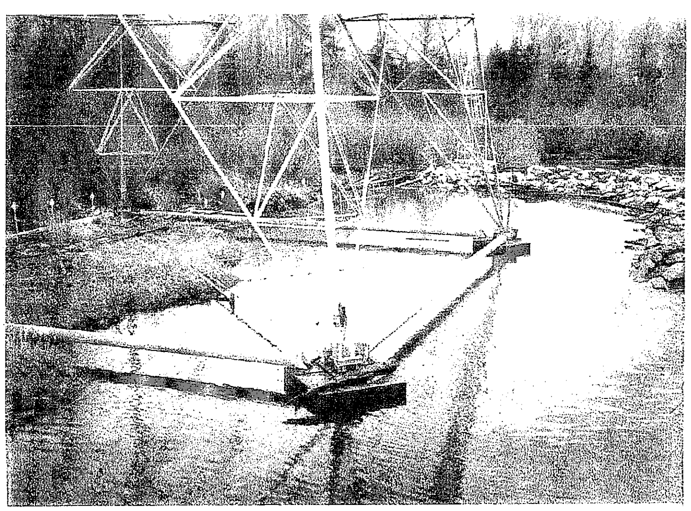
The crossing is made up of two 120 metre suspension towers and two anchor towers about 23 to 25 metres high that take most of the conductor loading from the span crossing the river. The foundations at all four of these

Fig. 2. Profile of the 2L04/56 crossing



towers and several other critical crossing towers in the Lower Mainland had previously been braced in the early 1990's. That is, the foundations were tied together with steel bracing which enables these towers to withstand a smaller earthquake, typically a 1:50 year event. Fig. 2 shows a profile of the crossing, and Fig. 3 shows the Annacis Island site and the style of bracing used on these towers which are the two tallest towers in the BC Hydro system.

Fig. 3. Tower 6/1 of Circuit 2L04/56 showing the site geometry and the seismic bracing.



Soils in the tower areas ranged from essentially clean sands at Towers 5/4 and 6/1 to sands overlain with silts and peat lenses at Towers 6/2 and 5/3. At Tower 5/3, the site was underlain by about 4 metres of fills, 2 metres of peat, 8 metres of silt and then medium dense sand to more than 30 metres depth. At Tower 5/4, there was about 1.5 metres of stiff surficial silts underlain by fine to medium sands to more than 46 metres. This material was generally loose to 20 metres depth and compact below that level. The sands at Tower 6/1 were loose to about 18 m depth and were medium dense below that level. At Tower 6/2, the soils encountered consisted of silts to 14 metres depth with a layer of organic silt and peat between 5 and 9 metres depth. Compact sands were encountered below the silts to more than 38 metres depth.

Seismic criteria

BC Hydro has established seismic withstand criteria for specified seismic return periods for all components based on their relative importance to the transmission system. All new 69 kV to 500 kV transmission circuits feeding major load centres, must be capable of withstanding the one in 475 year seismic event with no serious damage. Damage, if it occurs, must not result in an outage longer than two hours. For the one in 1000 year seismic event, any sustained component damage must be repairable within three days. For secondary 69kV to 230kV circuits these limits can be reduced to 1:100 and 1:200 seismic events respectively.

To determine the accelerations and magnitudes which correspond to these return periods, a probabilistic seismic hazard analysis (PSHA) was done using a modified version of the computer program EQRISK (McQuire, 1976). The procedure, originally developed by Cornell (1968), is based on geographically defined seismogenic source zones. Each source zone has an associated earthquake magnitude-recurrence relation. This information, coupled with ground motion attenuation relationships, allows EQRISK to calculate the peak firm ground acceleration that can be expected at a particular site for a specified return period.

BC Hydro's seismogenic source zone model incorporates a significant portion of northwestern North America and the eastern margin of the Pacific Ocean extending beyond the Queen Charlotte Islands. Deep and shallow earthquakes can be handled separately using appropriate attenuation relationships. The results of the PSHA for the Fraser River crossing at Annacis Island are shown in Table 1.

Table 1. Seismic parameters for the Fraser River crossing.

Annual Probability of Exceedance	Firm Ground PGA (%g)	Scenario Magnitude
0.0021 (1:475)	22	7.25
0.001 (1:1000)	29	7.25

The scenario magnitude is a reflection of the fact that the calculated PGA is an aggregate of many seismic events, each with a unique magnitude. The selected magnitude is based on a conservative assessment of each event's quantified contribution to the calculated PGA. The results in Table 1 do reflect possible contributions from a large Cascadia subduction earthquake. Local practice related to this type of earthquake is still evolving. Nevertheless, trial computations suggest that subduction generated ground motions will govern only on the west coast of Vancouver Island.

Seismic upgrade design

Liquefaction analysis

The liquefaction susceptibility of the foundation soils at the four tower sites was determined using the US National Earthquake Engineering Research Committee (USNCEER, 1996) modifications to the conventional procedure developed by Seed and his coworkers (1971, 1983). This approach compares the stresses induced in soil by an earthquake with the soil's resistance to liquefaction.

Earthquake induced cyclic soil stresses were calculated using a modified version of the dynamic response analysis program SHAKE (Schnabel *et al*, 1972). SHAKE is a one dimensional procedure that employs a columnar soil model that is layered to match the stratigraphy of a site. The soil column is assumed to rest on firm ground, preferably bedrock. An existing earthquake acceleration time history record, typically scaled to some maximum value, is applied to the base of the column. The cyclic shear stresses induced in the soil by the seismic event are calculated using one dimensional shear wave propagation theory. The analysis is an equivalent linear, total stress approach with no direct provision for the softening effects of pore pressure build up. Non-linear stress strain soil behaviour is accounted for by adjusting soil modulus and damping, iteratively, until there is consistency between these parameters and the computed dynamic shear strains. The shear stress time history is calculated for each layer in the soil model. The equivalent average uniform shear stress, τ_{avg}, is taken as 65% of the maximum computed stress. τ_{avg} is normalized by the effective overburden pressure, σ'_{vo}, to obtain the cyclic stress ratio or CSR.

The following dynamic soil properties must be defined for each layer in a SHAKE model:

- the maximum dynamic shear modulus under low strain, G_{max};
- the reduction of shear modulus with increasing strain;
- the equivalent viscoelastic damping.

G_{max} was estimated using the equation proposed by Seed and Idriss (1970).

[1] $G_{max} = 1{,}000\,(K_2)_{max}\,(\sigma'_m)^{1/2}$

where: $(K_2)_{max}$ = maximum shear modulus parameter
σ'_m = mean normal effective stress
G_{max} is in units of pounds per square foot

Appropriate $(K_2)_{max}$ values for the soil units were based on corrected SPT $(N_1)_{60}$ values using [2] (Seed *et al*, 1986).

[2] $(K_2)_{max} = 20\,[(N_1)_{60}]^{1/3}$

If shear wave velocity measurements are available, G_{max} can be obtained directly according to the physical relation given by [3].

[3] $G_{max} = \rho\, v_s^2$

where: ρ = mass density of the soil
v_s = shear wave velocity of the soil

The modulus reduction factors and damping ratios required to run SHAKE were based on work by Sy *et al* (1991) on Fraser River Delta soils in the Richmond area.

The dynamic analyses were run using five acceleration time histories (Table 2) measured from two earthquakes: the February 9, 1971 magnitude 6.6 event in San Fernando, California and the April 15, 1979 magnitude 7.0 earthquake in Monte Negro, Yugoslavia.

Table 2. Acceleration time histories used in SHAKE.

Earthquake	Accelerograph Station	Component
San Fernando	Caltech B Lab	S90W
	Griffith Park	S00W
	Stn. 126, Lake Hughes	N21E
	Lankershim Street.	S90W
Monte Negro	Albatross Hotel, Ulcinja	N00W

These acceleration time histories were selected primarily on the basis of their response spectra. A response spectrum defines the acceleration of a standard single degree of freedom oscillator to an earthquake record. The response spectra of the selected time histories are similar to a target spectral shape identified by Seed and Idriss (1982). All of the acceleration time histories were recorded on rock simplifying the preparation required before use with SHAKE. Finally, the chosen earthquakes had magnitudes similar to the scenario magnitude developed for the Annacis Island area (Table 1).

The peak accelerations recorded at the five accelerograph stations listed in Table 2 ranged from 0.14g to 0.19g; less than the 1:475 and 1:1000 values established for this project. However, the records were uniformly scaled to the required peak accelerations of 0.22g and 0.29g.

The assumed depth to firm ground and thus the corresponding depth of the SHAKE models was about 250 metres for the four tower sites. Recall that the selected acceleration time histories were measured at the surface on rock. Because the records are applied to the base of a soil column at some depth, adjustment for the effects of overburden, is required. SHAKE makes the necessary adjustments automatically. The program then calculates a profile of CSR, a measure of the stresses induced in the soil column by an earthquake, versus depth.

The susceptibility of a soil to liquefaction can be estimated from Seed's empirically derived curves which specify a quantity called cyclic resistance ratio or CRR. CRR is a function of soil density, typically measured using the $(N_1)_{60}$ parameter, and fines content. The former quantity was determined directly using standard penetration testing (SPT) in conformance with ASTM standards. Fines contents were measured in laboratory testing of the SPT samples.

A number of corrections are required to convert a raw SPT blowcount, N, to $(N_1)_{60}$ as shown in [4] and [5] below.

[4] $N_{60} = N\,(ER_m)/60$

where: N_{60} = blow count normalized to standard hammer efficiency of 60% of theoretical maximum

ER_m = measured or assumed efficiency of SPT hammer

On the Fraser River crossing project, repeated energy measurements were made to quantify ER_m so that accurate corrections for hammer efficiency could be made.

[5] $(N_1)_{60} = C_N\,N_{60}$

where: C_N = correction to normalize the blow count to a reference stress of 95.8 kPa

ER_m = measured or assumed efficiency of SPT hammer

C_N values were obtained by the method recommended in USNCEER (1996).

With $(N_1)_{60}$ and the fines content established for a soil horizon, a CRR value for the unit was obtained using the USNCEER (1996) version of Seed's original graph. Before proceeding, the CRR value must in turn be subjected to several additional corrections. These are defined in [6] below.

[6] $CRR_c = CRR \times K_\sigma \times K_\alpha \times K_m$

where: K_σ = correction for effective confining stress

K_α = correction for static shear stress

K_m = magnitude correction factor

K_σ and K_α values appropriate to the stresses acting at the depth being considered were obtained from USNCEER (1996). K_σ is less than one for effective confining stresses greater than 95.8 kPa thus reducing CRR at depths beyond about 10 metres. K_α is a function of soil density and the ratio of static shear stress and the effective vertical stress, τ_{st}/σ'_v. At flat sites, with τ_{st}/σ'_v equal to zero, K_α is one. At Tower 6/1, the only site surrounded by sloped ground, the best conservative estimate of K_α also resulted in a value of one.

The magnitude correction factor K_m is necessary because Seed originally developed his CRR chart for magnitude 7.5 earthquakes. K_m values, again published by USNCEER (1996), are greater than one for earthquakes with magnitude less than 7.5, thus increasing CRR. The converse occurs for magnitudes greater than 7.5.

For the Fraser River crossing project, cyclic resistance ratios were also determined by several alternate means. Cone penetration testing was completed at all four tower sites. A good CPT to SPT correlation was developed by comparing to the cone tip resistance measured at the location of an SPT test. With q_c measured in bars, this resulted in a q_c/N_{60} ratio of about 3.75. Applying this ratio to an entire CPT record converts q_c to N_{60} allowing a continuous, rather than a discrete, profile of CRR to be obtained.

Since Seed's original SPT based work, several direct correlations of CPT q_c to CRR have been developed. The procedure recommended by USNCEER (1996) was used in this study.

Finally, CRR values were obtained from cyclic simple shear laboratory testing. A limited program of testing was carried out on undisturbed silt samples obtained during site investigations. CRR values of 0.18 were obtained, results in line with the $(N_1)_{60}$ approach after correcting for fines content.

The liquefaction assessment is completed by comparing the profile of earthquake induced cyclic shear stresses (CSR) to the cyclic resistance ratio (CRR) profile of the soil. Because five earthquakes and two acceleration levels were considered, ten CSR profiles were obtained. CSR profiles obtained using the same acceleration level can be averaged or considered individually. Those areas where CSR exceeds CRR are identified as being liquefiable. The results of this assessment at the four tower sites are given in Table 3.

Table 3. Depth of liquefied zones at the four tower sites.

	Depth of Liquefaction (m)	
	1:475 Event	1:1000 Event
Tower 5/3	5 - 14	5 - 14
Tower 5/4	1 - 12	1 - 25
Tower 6/1	0 - 12	0 - 15
Tower 6/2	1 - 14	1 - 14

Deformation analysis

Based on analysis of case histories, Barlett and Youd (1995) have correlated the ground deformations associated with liquefaction to a number of factors: earthquake magnitude, distance from the seismic event, thickness and gradation of the liquefied layer and the topography of the site. The slope of the terrain and the presence of a free face, such as a ditch or riverbank, strongly influence the magnitude of ground displacement.

The Bartlett and Youd procedure has been applied to three of the four tower sites. The exception, Tower 6/1, was omitted because overall stability of the nearby riverbank slope governed at this location. Deformation predictions for the remaining three towers are given in Table 4.

Table 4. Estimates of liquefaction induced deformations.

	Deformation Estimates (m)	
	1:475 Event	1:1000 Event
Tower 5/3	0.4	0.5
Tower 5/4	1.5	1.7
Tower 6/1	-	-
Tower 6/2	0.5	0.5

For liquefied conditions at Tower 6/1 conventional slope stability analysis was completed using residual strengths derived from $(N_1)_{60}$ values (Idriss, 1998). Failure of the submerged slope was predicted with flow sliding considered likely. This necessitated the ground densification program discussed below.

Tower foundation upgrades

The four towers were originally supported on timber pile foundations. These foundations were expected to fail under the 1:475 year seismic event as a result of excessive lateral deformation and/or by bearing failure. As a result, concrete filled steel pipe piles have or soon will be installed at each tower. Design details are provided in Table 5.

The axial load capacities of the steel pipe piles were estimated using three well known CPT based procedures: the European method, the LCPC method and the Schmertmann method. Campanella (1991) gives an excellent summary of these procedures. All of the procedures derive pile end bearing and shaft friction capacities by factoring, according to rules that vary between methods, measured CPT tip resistance and sleeve friction values. The predicted axial load capacities are summarized in Table 6 for static conditions.

The minimum predicted end bearing and shaft friction values give adequate factors of safety for normal axial and uplift loading. Critical conditions exist for both liquefaction and post earthquake consolidation of liquefied soil. In the former case, available skin friction within the liquefied zone reduces to zero. For the latter condition, negative skin friction, equal to the pre-liquefied state but acting downwards, was assumed. Adequate factors of safety were achieved by driving the piles a sufficient depth below the predicted zones of liquefaction.

Table 5. Characteristics of new piled foundations.

Type	Concrete filled steel pipe piles
Number of piles	4 per tower leg, 16 per tower
Outside diameter	406.4 mm (16 inches)
Wall thickness	12.7 mm (1/2 inch)
Yield strength of steel	240 MPa (35 ksi)
Base plate thickness	38 mm (1.5 inches)
Rebar	Grade 400, #10 bar spiraled at 100 mm pitch, six #20 longitudinal bars
Centre Tendon	50 mm Dywidag "Tempcor" 500 MPa steel bar
Design lengths	24 to 28 m
Infill concrete	35 MPa, 0.5 water:cement ratio, 175 max/125 min slump (mm), 6% ($\pm$2) air content, 10 mm max aggregate size
Corrosion protection	Coal tar epoxy over top 6 m

Table 6. Axial load capacities of the steel pipe piles with no liquefaction.

		Pile Capacity (kN)	
Tower	Design Method	End bearing	Shaft friction
5/3	European	1946	872
	LCPC	979	1364
	Schmertmann	1946	918
5/4	European	1816	1225
	LCPC	908	1912
	Schmertmann	1816	2024
6/1	European	1946	1501
	LCPC	1394	2365
	Schmertmann	1946	1503
6/2	European	1946	616
	LCPC	973	1326
	Schmertmann	1946	770

The pipe piles were designed to accommodate large seismically induced deformations. The reinforced concrete placed inside the steel pipes substantially increases the buckling and rupture capacity of the piles under extreme bending. In the event of rupture, the high strength, centre mounted Dywidag tendon serves to tie the top and bottom halves of the pipe together. Thus, even a broken pile should continue to deliver the uplift capacity provided by unliquefied soil at depth.

The behaviour of the pipe piles under extreme bending was checked using a Klohn-Crippen version of the Byrne and Janzen (1985) LATPILE computer program. In addition to non-linear soil response, this modified program allows the piles to undergo plastic deformations. The piles were subjected to linear deformation profiles that varied from zero at the base of the liquefied zone to a maximum of 3 metres at the surface. In all cases, the piles developed a plastic hinge at the base of the liquefied zone. Maximum predicted strains, while beyond elastic behaviour, ranged from 5 to 7%. This compares favourably with the rupture strain for mild steel which is approximately 25%.

Ground improvement

The ground improvement program was carried out at Tower 6/1 to prevent potential flow sliding. Since it was not practical to carry out a ground improvement program within the tower footprint, an improvement berm was built around the tower site. The ground improvement design consisted of determining an overall geometry and the level of improvement required within the "berm" or "donut", and providing placement and confirmation requirements during construction. The intent of the ground improvement design was primarily to provide protection from the southward movement of the tower site into the main channel of the Fraser River. The Fraser River is currently maintained at a depth of 15 metres in the centre of the channel for shipping traffic in this area. The design was also intended to provide protection from ground movements in other directions in the event that future erosion of the river steepens the essentially flat profiles on the other three sides of the tower.

The berm width and geometry was determined using a Newmark analysis. Although there was concern initially about the capability of the Newmark analysis to provide realistic post earthquake displacements, the method was considered acceptable as conservative assumptions were used and the resulting displacements in the tower area were an order of magnitude less than the 1 metre of horizontal and 0.5 metres of vertical displacement used as the allowable limits for the tower foundations.

The resulting geometry for the ground improvement berm on the south side of the tower was 29 metres wide with a base at -18 metres elevation. The remainder of the berm was less critical and as a result, it was 22 metres wide with a base at -16 metres elevation.

The densification requirements for the ground improvement were based on piezocone (CPT) tip resistance. The design was derived from the calculated q_c required to resist liquefaction for several reference earthquakes. The average required resistance plus one standard deviation was used to derive a "minimum required improvement" line. This was simplified for tender purposes to 150 bars resistance for clean sands and 100 bars for silty sands. The main concern was to obtain densification in the zone below about 4 metres beneath the river bed mudline. The use of gravel columns was not considered to be realistic for densification above this level for the underwater (water based) portion of the densification. However, columns were installed to the mudline for drainage and reinforcement purposes.

The mass of riprap required for erosion protection on the south side of the tower, was estimated to be sufficient to consolidate the shallow soils which were not otherwise improved by the gravel columns. The amount of consolidation was assessed by survey monitoring of the crest of the riprap berm and by the amount of material required to dress the riprap slope later.

The final layout and spacing of the gravel columns was determined by the contractor as these parameters depend on the equipment available and the methods used. The final pattern consisted of columns at 3 metre centres on a square grid. This was easier to layout underwater than a triangular grid and provided an option to come back over the grid and install additional columns if the densification was insufficient.

Erosion protection berm

BC Hydro routinely protects tower sites that are subject to erosion with shoreline protection berms or riprap donuts. Under typical design circumstances the scour depth and water velocity is established and riprap volumes and critical stone sizes are determined based on those parameters. All erosion protection designs installed at BC Hydro transmission structures have a significant additional volume of riprap to allow for 'self-launching'. During self-launching, the additional material that is built into the design is meant to fall into place and fill in voids that are subsequently created by natural scouring and undercutting processes.

Calculating volumes for the 'root' of a riprap berm or donut is straightforward. Determining the additional volumes required to allow for the self-launching process was tricky at this site where a variable scour depth was used in recognition of the fact that the bulk of the erosion was caused by river traffic rather than scour, and was therefore affecting mostly the top two metres. As the environmental approval process to carry out this work was extremely arduous, both the size and volume of rock had to be sufficient to ensure that reinforcing the berm would not be required in the foreseeable future.

The initial design allowed for a scour depth ranging from -2.0 metres elevation at the east wing (upstream) of the permanent berm to -7.0 metres at the west wing (downstream) of the permanent berm. The initial volume for the permanent berm was about 7,000m^3, and about 1,500m^3 for the temporary berm. The purpose of the temporary berm was to establish a work site inside a donut lined with a geosynthetic that would contain the fines that would otherwise be released into the river during the reclamation of the southern tip of the island, stone column installation, pile driving and other related construction activities. BC Hydro did not seek approvals to build a permanent riprap donut around the tower. The north part of the berm would have to be removed and the site restored to original condition including replanting the native vegetation that was stripped and stored during site preparation, and re-establishing the original grades so that the island could continue to be naturally inundated from the north after construction was complete.

Fig. 4 shows a schematic of the ground improvement layout for both the water and land based work, and the location of the permanent and temporary riprap protection berms. Fig. 5 shows a cross section of the site.

Fig. 4. Schematic diagram of ground improvement layout and erosion protection berm at Tower 6/1.

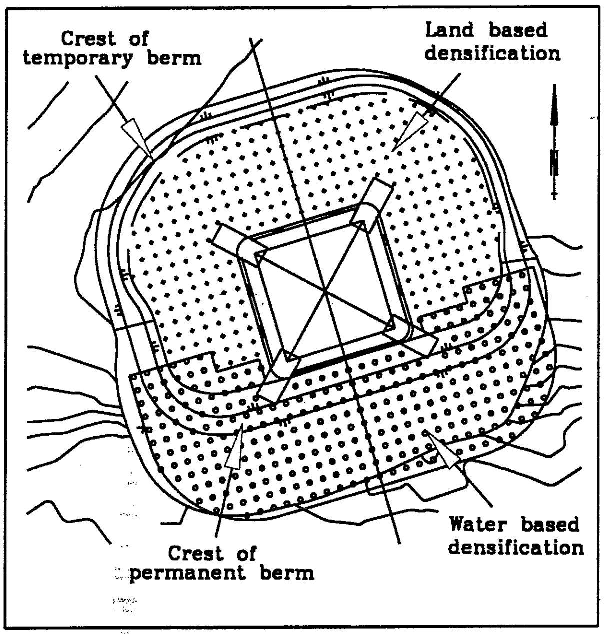
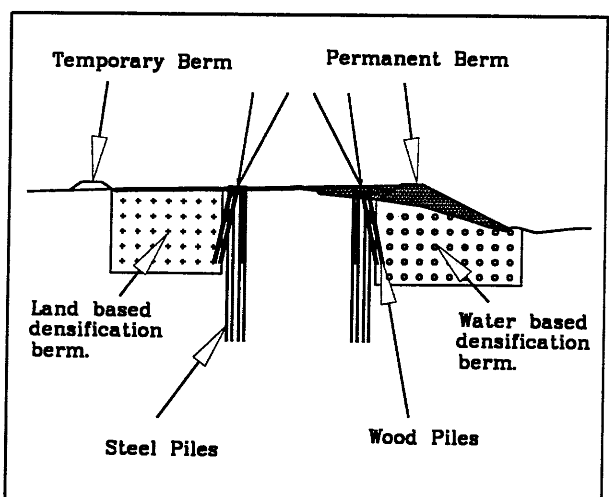


Fig. 5. Section of the site at Tower 6/1.



Construction

Utilities monitoring

At the anchor Tower 5/4 in Delta, two 1.2 metre diameter pipes run parallel to the tower within about 5 metres of the pipe piles. One of these was a City of Delta forced sewer main that is the only collection system from Tsawwassen and therefore would have caused an environmental disaster had it failed. Immediately adjacent to the Delta pipe is a GVRD water main. Failure of either of these pipes would have cause major erosion at the existing timber piles. To further complicate matters, both of these pipes were placed in unimproved peats under and adjacent to an access road with the water table typically at the surface.

In addition to an extensive monitoring program a detailed construction plan was presented to the utilities which incorporated measures to mitigate the possibility of damage. All of the pile locations at Tower 5/3 were pre-augured to reduce vibration, and the piles furthest away from the pipelines were driven first to allow for modification of the construction method prior to driving the piles closest to the services if necessary.

At Tower 5/3, three tri-axial strain gauge rosettes were placed on each of the pipes. Two of the locations were offset from the tower legs where the pile driving was to take place and where the highest pulse loads were expected. The third location was about 10 metres east of the tower and was used as a control. Settlement gauges were placed over and beside every strain gauge location and a geophone was fixed to the Delta main as it was the closer of the two to the tower and was expected to have higher pulse loadings.

At the remainder of the sites, roaming geophones were used to monitor each leg during pile driving, tiltmeters were temporarily fixed to the towers, and at Tower 6/1, tiltmeters were also read during ground improvement and during the installation of the riprap berm. The instrumentation was read using both automatic data acquisition systems and manually. Although the designers did not feel the risk of damaging an existing timber pile or a neighbouring utility was high, the data collection was a requirement of the insurance that was arranged for this project.

Other utility work included arranging for a temporary relocation of an 8" gas pipeline that was buried about 3 metres deep within about 1.5 metres of the existing tower footings along River Road (and therefore had a high water table), a vibration sensitive Microcell antenna mounted on one of the 120 metre towers, a 25 metre extension of a 1.8 metre storm sewer in Richmond, and a BC Tel cable installed in the Dyke along the Fraser River at the Richmond site. BC Hydro also had to arrange for alternating the outages of the two circuits during pile driving, installing the 18 metre re-bar cages and the full length Dywidag bars, and pouring the concrete at the two shorter anchor towers.

Pile installation and PDA results

The piles were driven to target elevations rather than to refusal criteria because of the loose ground prevailing at each of the towers. At Tower 5/3, the piles were driven with a 34 kN (7.5 kip) drop hammer falling a nominal height of 2.44 metres (8 ft). At the contractor's suggestion, a Berminghammer B300 Mark 2 single acting diesel hammer was used at Towers 5/4 and 6/2. This hammer has a nominal rated energy of 55 kN-m (40.3 kip-ft). Pile driving at Tower 6/1 will be carried out after ground improvement is complete.

Upon standing, the piles initially sank as much as 6 metres from self weight. The additional weight of the hammer, when placed on top of the pile, generated as much as 6 metres of further penetration. These zero blow conditions corresponded closely to the predicted depths of liquefaction. At Tower 5/3, blow counts varied from about 5 to 15 per foot. Values closer to the high end of this range occurred as the pile was advanced to its target elevation in denser soil. At Towers 5/4 and 6/2, where the diesel hammer was used, blow counts typically varied from 20 to 150 per foot. Again, values nearer the higher end of this range were obtained towards the conclusion of driving.

The piles were driven in two sections 12 to 15 metres long. After the lower section was driven, the upper section was welded to the bottom piece. All welds were visually inspected. Selected welds were X-ray tested. Upon acceptance of the weld, the pile was driven to completion.

To check on the predicted pile capacities, pile driving analyzer (PDA) testing was completed on selected piles at each tower. The tests were conducted with the piles driven to within about one metre of their target elevation. Two types of tests were conducted; at the end of continuous driving and during restrike measurements. The latter were made on the first few blows delivered to the pile after it had rested in the ground undisturbed for at least 24 hours. The restrike measurements should reflect the influence of any pile set up that acts to increase skin friction.

The results derived from restrike PDA measurements are summarized in Tables 7 and 8. Table 7 gives shaft friction results with comparisons to the best predictions based on the CPT methods. Table 8 does the same for end bearing.

Table 7. PDA versus CPT predictions for shaft friction.

	Shaft Friction (kN)	
Tower	PDA	Closest CPT Prediction
5/3	1579	1365 (LCPC)
5/4	1423	1225 (European)
6/2	1246	1326 (LCPC)

Table 8. PDA versus CPT predictions for end bearing.

	End Bearing (kN)	
Tower	PDA	Closest CPT Prediction
5/3	712	979 (LCPC)
5/4	289	908 (LCPC)
6/2	556	972 (LCPC)

It appears that the LCPC method is the most reliable of the CPT procedures for predicting pile capacity, at least for the soil conditions encountered. The same finding was reported by Robertson *et al* (1989) based on a Lulu Island field study. There is reason, however, to suspect that the PDA predictions are underestimates, particularly for end bearing. The net energy delivered by a hammer blow to the pile can be derived from PDA measurements. For the drop hammer, this quantity averaged about 55 kN-m, about 66% of the rated energy. For the diesel hammer, transfer energy averaged a much lower 16 kN-m, or 29% of the rated energy. In either case, it was recognized that the hammers used for driving were light for the type and length of pile being driven.

Conducting a PDA test with an undersized hammer is analogous to a static test with an applied load less than the pile's ultimate capacity. A weak hammer blow does not advance the pile enough to fully mobilize available soil resistance. In this case, the load capacities calculated from the PDA measurements, especially for end bearing, will be too low.

Ground improvement

The ground improvement was carried out to prevent potential flow sliding. About 300 water based columns were installed at a rate of about one hour each. The biggest difficulty during this stage of the construction was obtaining clean stone, it seems that the process of loading the material onto the barge introduced enough fines into the material to cause it to continually plug the bottom feed vibro equipment.

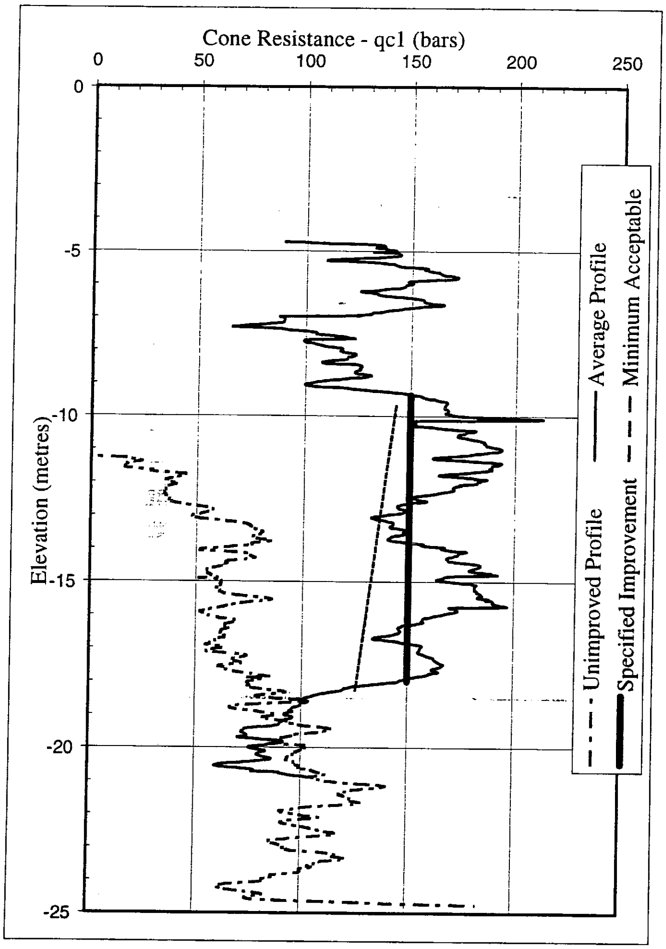
During the vibro-replacement work the engineering design was adjusted for the conditions that were encountered. During the course of the ground improvement work the installation procedure was continually adjusted to optimize the product.

The water based stone column densification work was assessed with two sets of CPT tests. The first set was performed after about 10% of the work had been completed to confirm that the layout and energy input was sufficient to obtain the required densification. The second set was performed at the end of the water based densification to confirm that the requirements were met before the area was covered with riprap. In addition to the CPT testing, volume of rock installed and the energy used was recorded by the contractor and by BCH personnel that were on site about two-thirds of the time.

The same test sequence was used for the land based densification.

During construction confirmation, CPT testing was also carried out in the water based area in unimproved zones as the initial vibroflot resistances appeared to be less than that indicated by initial exploratory testing. This latter testing indicated that the soils below the design berm were somewhat looser that initially expected. Analysis indicated that additional strain was likely to occur in spite of the fact the berm already was designed to extend well below the base of the river channel. As a result the depth of the toe area of the densification berm was increased by 2 metres to further reduce the potential for movement.

Fig. 6. Typical CPT before and after soil improvement.



Berm installation

The biggest challenge presented during the construction of the berm was the fact that the estimated volume of riprap almost tripled as a result of the original mudline being lowered by 4 to 6 metres due to ground improvement in addition to the one metre of elevation lost due to erosion. Most of this change in elevation is believed to be due to settlement and re-distribution of material within the worksite. The berm height was also raised by one metre because of the large wakes created by the river traffic. The D_{50} for this design was 0.7 metre and as the volumes increased the problem of getting material to site increased because only the initial volume was procured and therefore additional sources had to be located. Few companies would rent barges for transporting such large rock because of the wear and tear on the equipment, and as a result, several days were lost when the contractor could not get rock to the site.

Another result of the dramatic changes in the river bottom between the pre and post densification work was that the design had to be adjusted to prevent the riprap toe from interfering with the pile installation. In order to ensure that the final toe location of the berm did not vary too much from the original design, the original mud line was re-established using 200mm minus mattress material. This measure would also make the removal of the temporary berm easier as the riprap would be prevented from punching in on the north side of the island.

About 4,000m^3 of 200mm minus was placed under the permanent erosion berm.

Other challenges and lessons learned

One of the biggest challenges of this project which did not fall under the responsibility of the engineering design group was obtaining the approvals from all the regulatory bodies involved including the Department of Fisheries and Oceans (DFO), the North Fraser Harbour Commission, the Fraser River Estuary Management Program (FREMP), the City of Richmond and the City of Delta. Because of the number of agencies involved and the competing interests of the engineering design team, an environmental specialist was assigned to the project team at the planning stage. During the construction stage, in order to ensure that approvals once obtained are not revoked, it is equally important that an experienced environmental monitor with all of the appropriate responsibility and authority maintains a presence during construction, especially when entering a new phase of construction or during the first few days of using a new piece of equipment or method.

Compensation ratios for habitat which would be damaged or permanently alienated are calculated based on recognizing that different habitat components such as mud flat, marsh or riparian zones have different productivity values. For this project BC Hydro had to compensate for habitat components under the permanent berm by building a marsh elsewhere on the site.

For Canadian projects, allow at least one year for the permitting process. In preparing the documents for permit approvals, it is important to acknowledge that the workload of the permit agency with respect to life safety issues will take precedence over all other permit work. For the Fraser River upgrade, most of the regulatory bodies that BC Hydro had to deal with were occupied planning for the 1999 freshet which was predicted to be the 1:200 year flood, and therefore most other property and personal safety issues took precedence over the permits BC Hydro required for the in-river construction.

The project was approved in large part because of the importance of the two 230kV circuits to the BC Hydro transmission system. BC Hydro applied for the permits under the public health and safety component of the FREMP review process. Had this work been proposed to improve and reclaim the land for private development the conditions imposed could have been insurmountable if the application was not rejected outright.

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