Seismic upgrading of the BC Gas Fraser Gate Station

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Abstract: BC Gas Utility Ltd. (BC Gas) has undertaken an extensive seismic evaluation and upgrading program to minimize the risk of gas supply disruption to its customers in the event of an earthquake. This paper presents the details of a seismic vulnerability assessment of the Fraser Gate Station located on the North Bank of the North Arm of the Fraser River in Vancouver, B.C. The site-specific assessment included a geotechnical field investigation, prediction of geotechnical seismic hazards, and structural pipeline performance evaluation. The analyses indicated a high risk of liquefaction of loose sandy soils extending to depths up to 12 metres below the ground surface. Large seismic flow-slide ground movements towards the river were predicted and the structural vulnerability assessment indicated that the anticipated displacements exceed the available capacity of the pipelines by an order of magnitude. Densification of the loose soils along the shoreline using the method of vibrò-replacement, combined with reconfiguration of the foreshore river bank to a gentler rip-rap slope, was employed to improve seismic performance. Development of an Environmental Protection Plan, sediment control, environmental monitoring during construction, and restoration of riparian vegetation following ground densification were undertaken to minimize and mitigate the environmental impacts during construction.

Introduction

BC Gas Utility Ltd. (BC Gas) supplies natural gas to the Lower Mainland region of British Columbia. Since the gas distribution area is situated within a region of high seismicity, the seismic performance of the pipeline system is of particular concern to BC Gas. Over the past six years, BC Gas has undertaken an extensive seismic evaluation and upgrading program to minimize the risk of gas supply disruption to its customers as a result of an earthquake event.

Given the lack of redundancy in certain areas of the supply system, combined with the resulting social, economic, and business impacts, a very low risk of disruption to the gas supply is considered acceptable to BC Gas. An annual probability less than 0.05 percent (equivalent return period 2,000 years) has been adopted as a target level for the acceptable risk of loss of gas pressure from the pipeline system. This is based on comparison with recent studies for other lifeline systems and the operational goals of BC Gas.

Initially, the seismic vulnerability of the transmission system was studied on a regional basis. The objective was to identify the system components that are most vulnerable to damage resulting in disruption to gas supply. The study included evaluation of seismic geotechnical hazards and structural vulnerability assessment. Components that failed to meet the performance criteria were ranked based on the anticipated vulnerability. The results of the regional study were used to prioritize emergency response planning and remedial measures.

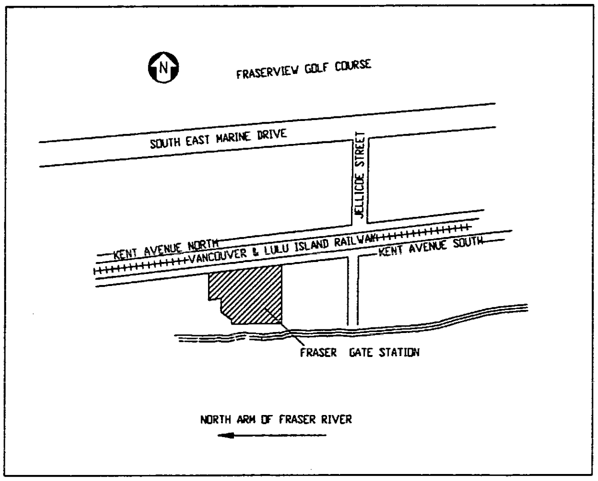
Based on the regional study, the Fraser Gate Station located on the North Bank of the North Arm of the Fraser River in Vancouver was ranked highest in terms of seismic vulnerability. This gate station serves as a control point for pipelines which form one of the main arteries supplying natural gas to the City of Vancouver. Liquefaction-induced ground deformations were identified as significant hazards to the 508 mm and 610 mm diameter pipelines entering the gate station and the facilities within the gate station. This paper presents the details of a site-specific seismic vulnerability assessment of the gate station and subsequent seismic upgrading work that was undertaken to reduce the risk of gas supply disruption at the station to levels acceptable to BC Gas.

Site description

The Fraser Gate Station is located in the 2700 Block of Kent Avenue South in Vancouver, B.C., Canada, as shown in Fig. 1. The gate station compound is rectangular in plan (~100 m x 75 m), and it is situated on the north bank of the North Arm of the Fraser River. The plan alignments of the 508 mm and 610 mm diameter transmission pressure pipelines entering the gate station are also illustrated in Figure 1.

The site is bounded by Kent Avenue South to the north, part of Riverfront Park owned by the City of Vancouver to the west, and a residential development to the east. A site

Fig. 1. Location plan – BC Gas Fraser Gate Station



KEY PLAN

plan is shown in Fig. 2. The strip of land between the crest of the river bank and the southern fence is presently used as a walkway which is understood to be a part of the City's Riverfront Park.

The site topography within the station compound and also in the east-west direction is generally flat. Prior to treatment, the river bank sloped down towards the south at slopes ranging 1H:1V to 3H:1V (horizontal:vertical) within the rip-rap area which extended to about 6 metres below crest level. The river bed below this level sloped southward at an average gradient of about 8% to the horizontal.

Subsurface soil conditions

A profile illustrating the inferred soil stratigraphy, developed based on a geotechnical field investigation comprising both on-shore and off-shore drilling at the gate station site, is shown in Fig. 3. The field investigation indicated that the upper soils within the station consist of about 1.7 to 2.7 metres of loose to compact sand to sandy silt fill material. The upper fill materials in the northern part of the gate station are underlain by a layer of very soft to soft silt (WL = 38%; IP = 11%; W = 40%) extending to depths in the order of 6 to 8 metres below the ground surface. This silt layer is underlain by a compact to dense sand stratum, which in turn was found to overlie a very dense sand and gravel stratum at a depth of about 8.8 metres below the ground surface. The test holes carried out within the southern shoreline part of the gate station compound indicate that the soils underlying the upper fill materials primarily consist of loose to compact sand extending to depths of up to 11.0 metres below the ground surface. Underlying these soils, compact to dense sand with a trace to some gravel was encountered. These strata are underlain by dense glacial till-like material which was encountered at a depth of about 14.0 metres below the ground surface. Electric cone penetration testing within the river adjacent to the site also indicates the presence of sandy soils, below a 2 metre thickness of silt and clayey silt, and extending down to a depth of about 9 metres below the river bed. These materials were found to be underlain by a compact to dense soil stratum.

The groundwater level within the gate station compound was noted to be at depths of about 1.0 to 3.0 metres below the ground surface during the period of the geotechnical investigation.

Geotechnical performance under earthquake loading

Seismic ground motion response analyses were carried out using the one-dimensional wave propagation program SHAKE (Schnabel, 1972) to compute the induced cyclic stress ratios. Seed et al. (1984) liquefaction resistance charts were used to assess the liquefaction potential of the site soils. The results indicated that the zones of potential liquefaction are primarily located on the south portion of the site. For the seismic risk levels corresponding to 1:100, 1:475, and 1:2,000 year return periods (firm ground accelerations 0.09g, 0.20g, and 0.34g, respectively), the depth of loose potentially liquefiable sandy soils at the site was found to be generally limited to about the upper 12 metres (see Fig. 3). An earthquake magnitude of M7 was used in the liquefaction assessment. Based on the Chinese criteria (Marcuson et al., 1990) for the assessment of liquefaction susceptibility of fine-grained soils, the risk of liquefaction of the silty strata located within the northern portion of the site, was classified as low.

The post-liquefaction stability of the gate station compound was analyzed using the computer code XSTABL. The post-liquefaction shear strength parameters for potentially liquefiable zones were mainly selected based on laboratory post-cyclic monotonic simple shear test data reported by Pillai and Stewart (1994) and Sivathayalan (1991). Based on this information, a post-liquefaction shear strength equivalent to 20% of the initial effective vertical stress was assumed for the stability analyses. Both circular and non-circular failure surfaces were analyzed to investigate the potential for a flow slide condition at the site. Potential slip zones with significant encroachment into the station compound (failure zones extending landwards about 30 metres from the river bank) were computed to have a post-liquefaction factor of safety less than 1.0 even without application of any seismic inertia forces. This suggested a high risk of a flow slide as a result of earthquake shaking leading to very large deformations for the southern part of the site for earthquake loading corresponding to all three risk levels.

Limited ground displacement analyses were carried out to obtain a better understanding of the magnitude and patterns of the relative ground movements in the area north of the predicted flow slide zone. The liquefaction-induced free-field ground displacements were calculated using the

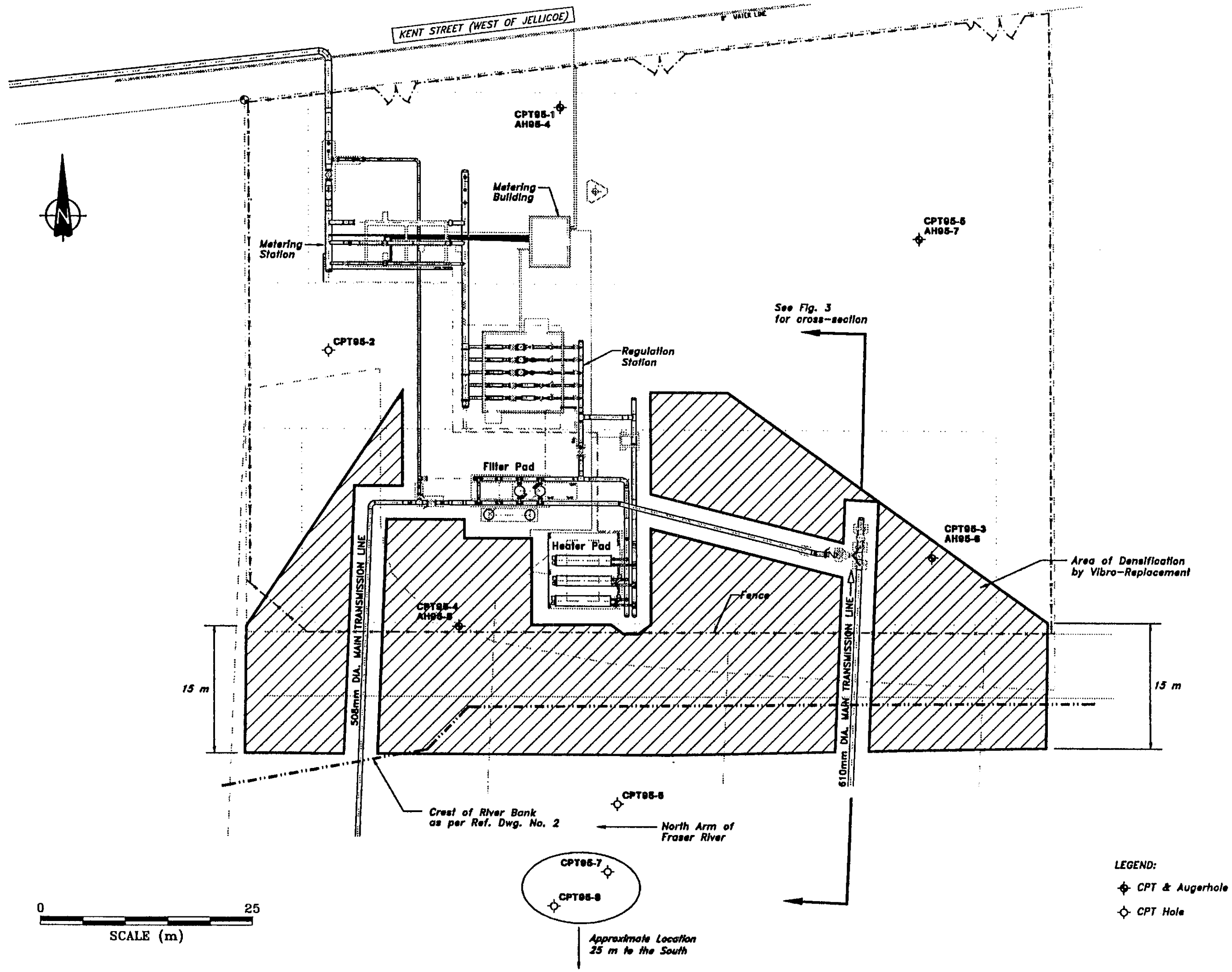


Fig. 2. Site plan showing exsiting structures, pipeline configurations, and geotechnical testhole locations

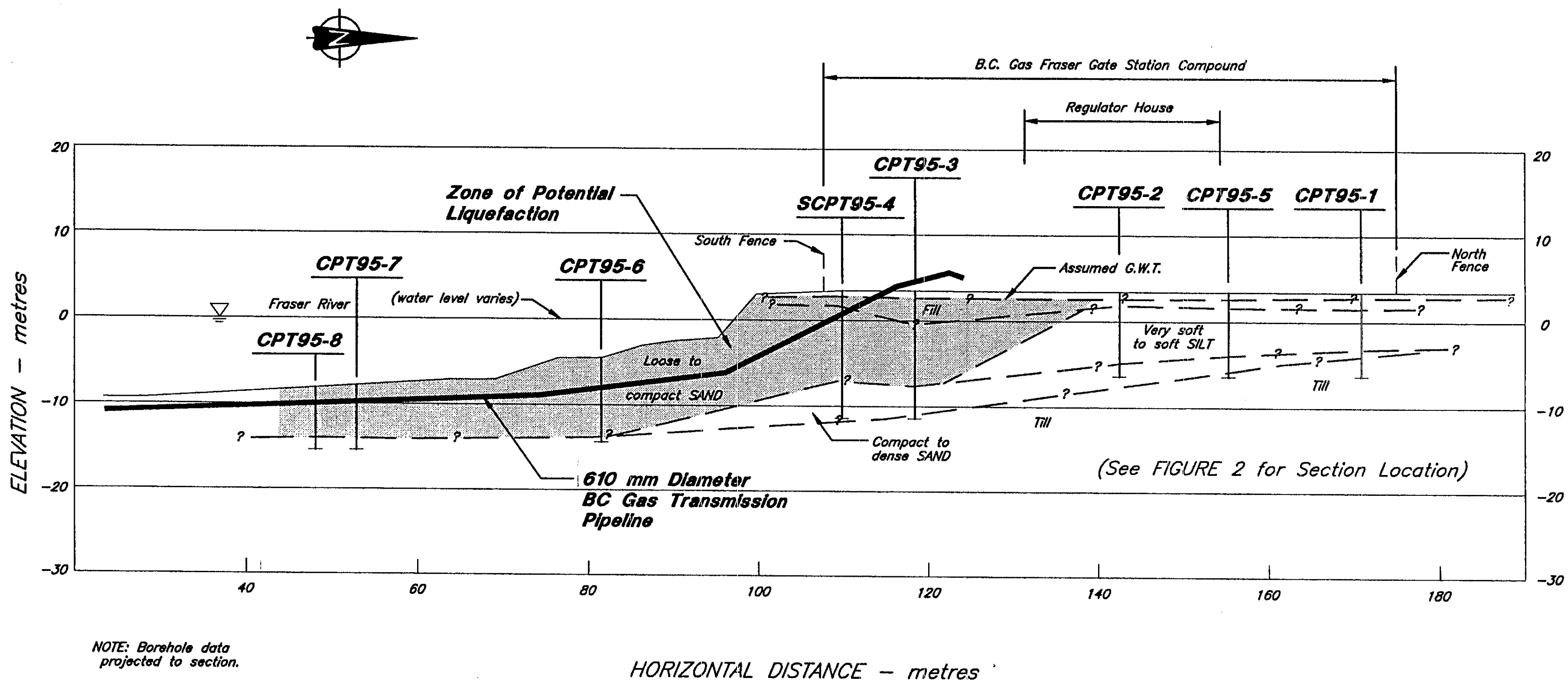


Fig. 3. Profile of soil stratigraphy, predicted zone of potential liquefaction, and alignment of pipeline 610 mm diameter pipeline

computer program DISPLMT developed by Houston et al. (1987) using the Newmark (1965) sliding block method and the empirical MLR method developed by Bartlett and Youd (1992).

The predictions from both analysis techniques indicated that, for the seismic loadings corresponding to all the risk levels considered in the study, large ground displacements (in excess of 3 metres) towards the river were predicted to influence an area extending to about 30 metres north from the crest of the river bank. The ground displacements for the non-liquefiable silty zones within the northern half of the site were computed to be less than about 0.1 metres. It is also noted that, along with lateral ground movements, significant vertical ground movements were expected to occur within the southern area of the site due to translation of the soil mass.

Seismic performance of gate station piping

The vulnerability of the piping at the site was assessed using the results of the regional vulnerability analyses in 1993, site-specific soil strength information, experience with detailed evaluations of similar configurations, and observation of pipeline performance in past earthquakes. Based on the variation of ground deformation capacity with soil type and the soil strength information, approximate ground deformation capacities for assessment of the Fraser Gate Station piping were computed as summarized in Table 1. Although these estimates are very approximate, they serve as a means to judge the relative severity of the site-specific estimates of ground deformation on the pipelines.

Maximum computed ground deformations derived from the geotechnical analysis were compared with the computed pipe structural deformation capacities. The computed large ground displacements and resulting differential displacements at the Fraser Gate Station from earthquake-induced liquefaction were found to exceed the estimated capacity of the pipelines by an order of magnitude. This indicates that the risk of damage to the station piping under earthquake loading was well above the BC Gas acceptance criteria.

Evaluation of remedial options

Given the gross exceedance of the available pipeline capacity, the only remedial measures deemed practical for the Fraser Gate Station involved improving the ground conditions. Provided that ground improvements could reduce the potential earthquake-induced permanent ground deformations to less than about 15 centimeters, no modification of the existing station piping was judged to be necessary.

The effectiveness of ground improvement in reducing the liquefaction-induced ground displacements at the site was assessed using the slope stability program XSTABL and the finite element code SOILSTRESS (Byrne et al., 1992). A design concept assuming the densification of a rectangular area parallel to the dyke alignment was investigated. The width of the densification zone was assumed to be 10 and 20 metres and the treatment was assumed to extend to the predicted full depth of liquefaction.

These results indicated that the introduction of a densified barrier, likely in the order of 15 to 20 metres wide, would reduce the expected large earthquake-induced ground movements in the vicinity of the gate station below the structural deformation capacity of the pipelines. In addition to the ground improvement, the shoreline slope was configured to a gentler slope to improve the river bank slope stability.

Table 1. Approximate ground deformation capacities for structural assessment of piping vulnerability

Pipe Size	**508 mm (20 in)**	**610 mm (24 in)**
Lateral Soil Movement Perpendicular to Straight Pipe (m)	4	2
Lateral Offset at Elbow (m)	0.15	0.25

Ground improvement using vibro-replacement

Several methods were considered to improve the liquefaction resistance of the soils. The selection of the most suitable ground improvement technique was governed by several factors, such as soil conditions, equipment space restrictions, pipeline protection issues, environmental regulatory requirements, land availability etc. Based on an evaluation of these considerations, the method of vibro-replacement was considered to be the most suitable technique of ground densification for use at the Fraser Gate Station site.

A total of 273 stone columns were installed (using the method of vibro-replacement) in a triangular pattern at 3 metre centre-to-centre spacing to cover the plan area shown in Fig. 2. The vibro-replacement was performed so that the bases of all stone columns were installed to the top of the underlying hard stratum. The stone columns were installed to depths of between 8 and 16 metres below the existing ground surface, with an average depth of 13.6 metres. The average amperage output during construction of individual stone columns was about 150 A, with peak outputs ranging from 170 to 260 A.

A total of six columns, from an initial densification pattern comprised of 294 stone columns, had to be deleted due to concern that installation at these locations may result in unacceptable deformations of adjacent gas pipelines. Boulders, concrete and timber obstructions were encountered during column installation at some locations, generally at depths of some 3 to 6 metres below the existing ground surface preventing the installation at a total of 22 stone column locations. Of these 22 locations, 7 columns were successfully installed at alternate locations within 1.5 metres of the design location. In general, attempts were made to relocate stone columns rather than locally excavate the obstruction. This approach was adopted due to concern that some of the timbers encountered could extend within the gas transmission pipeline corridor, and disturbance of these obstructions would present unacceptable risk of damage to the buried pipes.

Post-densification verification cone penetration testing

Field verification testing was performed at selected centroids of the stone column pattern using the method of electric cone penetration testing (CPT) during the progress of the densification program. The purpose of these tests was to measure and review the degree of densification achieved by the vibro-probe pattern and other operational variables. The results of the post-densification testing together with review of the stone column installation details indicated that the cone tip resistance (Q_t) values generally exceeded pre-specified performance Q_t criteria (ranged between 100 and 125 bars for clean sand zones in the zone of potential liquefaction). Some of the initial CPTs, carried out within about 14 days from the time of stone column installation, indicated that the specified Q_t requirement was not satisfied in certain zones of silty fine sand (Note: Q_t requirements were corrected the for silt content); however, repeat testing carried out in the same areas after about five weeks from the installation of the stone columns indicated that the Q_t values had increased significantly from the initial post-densification values, and met the specified criteria for silty sands (see Figure 4). These results demonstrate the importance of considering the effect of aging in the assessment of ground improvement, particularly in relatively fine-grained soils.

Monitoring of BC Gas facilities during construction

BC Gas operational requirements necessitated that the gas supply be maintained throughout the construction work at the site. Although there was some flexibility to reduce the gas pressure in the transmission pipeline that was closest to the vibro-replacement work area (and carry out the work in a sequential manner), a complete shut-down of the pipelines was not feasible. In order to minimize the risk of pipeline distress due to the construction activities, the zone of vibro-densification was kept at least 2 metres away from the existing facilities. These included existing high pressure gas transmission pipelines, pile supported heaters/filters, and regulator building, as well as piping used for monitoring and supplying fuel gas. Field instrumentation, including slope inclinometers, deep settlement gauges and piezometers, were installed prior to construction work to monitor the horizontal and vertical ground movements and groundwater level variations in the vicinity of the buried pipelines during the process of ground improvement treatment.

The lateral ground movements were evaluated from periodic slope indicator probe measurements at four locations. The results indicate lateral ground movements of less than 25 millimeters at depths greater than about 6 metres below the ground surface. Lateral ground movements between about 25 to 60 millimeters were noted within the upper 6 metres. As expected, the predominant lateral movement of these upper soils was in a direction away from the zone of densification and towards the gas transmission pipeline.

In general, ground settlements of less than 25 millimeters were observed during installation of stone columns at lateral distances of greater than 6 metres from the settlement gauge locations. Observed settlements of monitoring points within 5 metres depth below the ground surface increased to between 70 and 230 millimeters during installation of stone columns located between 0.5 to 1.5 metres horizontally from the settlement gauge. The observed settlements remained less than about 70 millimeters for monitoring points at depths greater than 5 metres below ground surface.

During the vibro-replacement work, field measurements were periodically carried out to measure the groundwater level in the piezometers to assess any potential build-up of pore water pressures in the soil mass near the river bank. The measurements indicated that there was no significant build-up of pore water pressures due to vibro-replacement.

The subsurface survey monitoring points on the gas transmission pipes were established by excavating down to expose the top of the pipe, installing a vertical flexible plastic pipe which extended above ground surface, and then backfilling around the gas line and plastic access pipe. The survey crew was on-site during installation of stone columns within 6 metres of sensitive facilities, and monitored a selected group of survey points in the immediate vicinity of the construction activity every 15 to 30 minutes. The site engineer was informed immediately if deformations in excess of 3 millimeters difference from pre-construction measurements were observed. The observed vertical movements are summarized in Table 2.

The allowable movement tolerance limits to meet the operating requirements, shown in Table 2, were determined based on structural evaluation prior to vibro-densification, and the observed vertical movements during construction were all within the defined tolerance limits.

Fig. 4. Results of post-densification electric cone penetration tests at the centroid of adjacent stone column triangluar patterns (each carried out 13 and 34 days after installation)

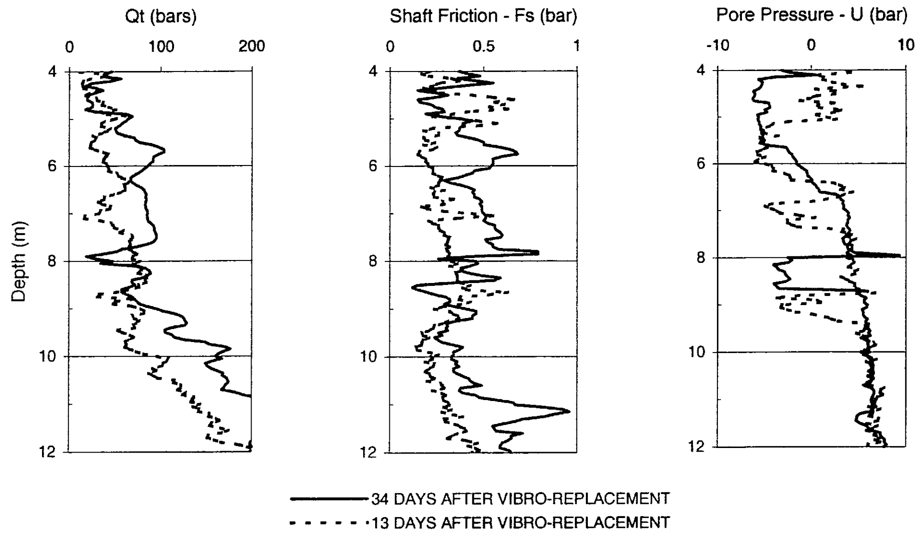


Table 2. Summary of survey monitoring of BC Gas facilities

Monitoring Location	Movement Tolerance Limits	Maximum Observed Movement
Regulator Bldng.	Uniform vert. Mvmt. <50 mm	5 mm (heave)
	Diffrntl. vert. Mvmt. <2.5 mm / m length	6 mm over 10.2 m length of pipe
Regulator Bldng. Inlet Lines	Diffrntl. mvmt. < 40 mm	4 mm (heave)
Heater Pad	Diffrntl. mvmt. between pad and surface pipe <15 mm	11 mm (6 mm max. pad diffrntl.)
Filter Pad	Diffrntl. mvmt. Between pad and surface pipe <15 mm	12 mm (6 mm max. pad diffrntl.)
508 mm Trans. Tie-In	Diffrntl. mvmt. between monitoring points < 40 mm	15 mm (settlmt.) (11 mm max. pad diffrntl.)
610 mm Trans. Tie-In	Diffrntl. mvmt. between monitoring points < 40 mm	14 mm (settlmt.) (3 mm max. pad diffrntl.)

Shoreline restoration and regrading

Restoration and regrading of the rip-rap shoreline slope, including a riparian bench, was carried out and consisted of excavation of the existing rip-rap shoreline to design lines and grades, supply and placement of new filter stone, rip-rap, approved fill materials, geotextile, geogrid, and topsoil in areas south of the Fraser Gate Station compound.

In general, the shoreline was restored in stages such that open excavation along the shoreline was limited to less than 30 metres width. The toe apron and lower portion of the riverbank were excavated using a barge-mounted clamshell derrick and the upper portion was excavated using a land based track-mounted excavator. Timber obstructions were frequently encountered during removal of the existing bank protection materials, resulting in some local zones of over-excavation. In the vicinity of the 610 millimeter gas transmission pipe, only limited excavation in the toe apron area was carried out to minimize the risk of damage to the pipe.

Environmental monitoring

All work on this project was required to comply with the environmental regulations and conditions stipulated by BC Ministry of Environment Lands and Parks (MELP), Department of Fisheries and Oceans (DFO), the Fraser River Estuary Management Program (FREMP), North Fraser Harbour Commission (NFHC), City of Vancouver (CoV), and other agencies having jurisdiction over the area.

As part of the overall construction monitoring, an Environmental Protection Plan (EPP) was incorporated in the vibro-densification and shoreline restoration contract. The EPP identified the nature and magnitude of potential impacts as well as associated risks to the environment and defined the minimum care, procedures and contingency measures to be exercised by the contractor(s) for the protection of the environment during the construction period. Compliance with regulations and conditions was facilitated by the preparation and distribution of an Environmental Emergency Response Card.

An environmental monitoring program was implemented to monitor the contractor's compliance with the requirements of the EPP to ensure no unacceptable amounts of deleterious substances or other disruptions such as excessive construction noise impacted the environment.

Conclusion

The BC Gas Fraser Gate Station is a critical control point for pipelines that supply natural gas to the City of Vancouver. The gate station and associated piping were identified as being at high risk of damage and loss of gas pressure integrity in the event of seismic-induced ground displacements. Site specific seismic vulnerability analysis were performed leading to design and implementation of ground improvement strategies in order to lower the risk of gas supply disruption to tolerable levels. Densification of the soils underlying the gate station compound and the adjacent river bank was successfully carried out, in close proximity to the operating pipelines, using the method of vibro-replacement, and the adjacent shoreline was re-configured to a flatter slope.

This paper illustrates a methodology that can be employed for seismic evaluation of buried pipeline systems and the applicability of pipe/soil interaction analysis in evaluation of the ability of buried piping to resist seismic-induced ground displacements. The approach presented above required consideration of geotechnical, structural, operational, and environmental components, and it demonstrates that geotechnical remedial measures can be successfully implemented to minimize the risk of earthquake-induced damage to pipelines and related facilities, even in close proximity to operating pipelines and sensitive environmental areas.

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