Ground improvement for seismic upgrading of lifeline facilities

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Abstract: The occurrence of recent devastating seismic events around the world has resulted in a remarkable increase in the public interest towards earthquake preparedness in the Pacific Northwest. Particular interest has been directed to the performance of lifeline facilities such as railways, highways, pipelines, and power/telecommunication lines, following a major earthquake event. The threat of potential damage to these lifeline facilities, especially those located in areas with potentially liquefiable soils, has encouraged most of the lifeline facility owners in the Fraser Delta to undertake initiatives to protect their facilities from earthquake-induced damage. Experience from previous seismic events that have occurred in other parts of the world indicates that earthquake-induced permanent ground displacements and/or loss of bearing capacity are the key geotechnical hazards to lifelines located in liquefiable soil deposits. During recent years, ground improvement techniques have been widely used for the seismic retrofit of facilities that are located within, or that have foundations supported on liquefiable soils. This paper presents three case histories where ground improvement techniques have been used to provide lifeline facilities, including a major bridge, a natural gas transmission pipeline, and a key structure at a liquefied natural gas (LNG) plant, protection from liquefaction-induced geotechnical hazards.

Introduction

The Fraser Delta of British Columbia is located in Seismic Zone 4 (on a scale of 0 to 6, as defined in the National Building Code of Canada) indicating that this region is a moderate to high seismic risk zone. Intra-plate earthquakes occurring within the North America tectonic plate underlying the Fraser Delta have been regularly recorded over the last century. There also exists a potential for large-scale subduction earthquakes within the region resulting from the thrusting of the offshore Juan de Fuca Plate beneath the continental North America Plate. Although large inter-plate subduction earthquakes have not occurred in the Fraser Delta in recent times, there is geological evidence that such earthquakes have occurred in the past. The recent devastating seismic events around the world, such as Loma Prieta (1989), Northridge (1994), Kobe (1995), Turkey (1999) and Taiwan (1999), has remarkably increased the public interest towards earthquake preparedness. Particular interest has been directed towards the post-earthquake performance of lifeline facilities, such as transportation systems (railways, highways, etc.), oil/gas pipelines, water distribution systems and power/telecommunication lines, that are essential components of today's society.

The threat of potential damage to lifeline facilities, especially those located in areas with liquefiable soils such as in the Fraser Delta, has encouraged most of the lifeline system owners to undertake initiatives to assess the earthquake performance of their facilities. Based on the results of these seismic vulnerability assessments, components of the lifeline systems that are identified as being susceptible to earthquake-induced damage are typically replaced or retrofitted to improve their performance during and following a major earthquake.

Use of ground improvement in seismic remediation of lifelines

Experience from previous earthquakes indicates that permanent lateral ground displacements and loss of bearing capacity are the most serious geotechnical hazards with respect to lifeline disruption at sites underlain by liquefiable soils (Hamada and O'Rourke, 1992 and O'Rourke and Hamada, 1992). Once components of lifeline facilities are identified as being vulnerable to earthquake-induced damage, there are several remedial alternatives that are available to the lifeline system owners. The lifeline facility can be removed and relocated to areas that are less susceptible to earthquake shaking. However, the costs associated with rebuilding and/or relocating the lifeline system infrastructure almost always makes this option cost prohibitive. Structural retrofitting can also be carried out to protect the lifeline components. Some of the most common structural retrofit methods include strengthening of the weaker structural members, reconfiguring lifeline system components to be less susceptible to earthquake damage, increasing the wall thickness of pipelines, and installing energy dissipaters to reduce the loading imposed on structures by earthquake-induced ground movements. For lifeline systems located in areas of potential liquefaction, the use of structural retrofit measures alone is often insufficient to protect the lifelines.

Inspection of sites/facilities following major earthquake events indicates that ground densification is an effective method to reduce damage to facilities. These inspections and studies have lead to the displacement-based or performance-based design of structures in high seismic areas.

A variety of ground improvement techniques have evolved in the past 2 to 3 decades to permit construction of critical structures on sites with poor soil conditions. Some of these ground improvement methods include dynamic compaction, vibro-replacement using stone columns, compaction piling, preloading, blast densification and compaction grouting. More recently, ground improvement has been used to improve the post-earthquake performance of sites in addition to improving their load bearing and settlement characteristics (Mitchell, 1998).

The design of ground densification to be used to improve the seismic performance of existing lifeline facilities is often more complicated than the design of ground improvement for new developments. Whereas ground densification for new construction projects has historically been carried out at undeveloped sites, seismic retrofit of existing lifelines involves ground improvement in the immediate vicinity of lifeline systems that must often remain operational during construction. These ground improvement works must be designed not only to account for space constraints imposed by the existing facilities but also to limit settlements and vibrations that could damage the lifelines.

Ground improvement is increasingly being used to protect structures from earthquake-related damage by providing non-liquefiable soil barriers to limit lateral ground displacements and by increasing the bearing capacity and settlement characteristics of loose or soft soil deposits.

The choice of the most appropriate ground improvement method for protection of lifeline facilities is site-specific. The selection process typically includes evaluation of estimated costs, site constraints such as access and clearance room for equipment, environmental considerations and the potential for damage to the existing facilities. This paper presents three case histories where ground improvement techniques have been used to provide protection to both existing and new lifeline facilities from liquefaction-induced geotechnical hazards. The case histories also highlight how the selection of the appropriate ground improvement methods were influenced by the presence of existing facilities and by operational constraints imposed by the lifeline owners.

Case histories

Case history No. 1 - Seismic safety retrofit of foundations of a major bridge

This case history describes the geotechnical retrofit measures implemented at a major bridge crossing of the Fraser River in New Westminster, BC. The main objective of the geotechnical retrofit was to provide a "safety level" seismic retrofit for the bridge so that it would not collapse under a 1 in 475 year return period level earthquake.

The bridge consists of three main spans over the river channel and a series of north and south approach spans. To the north of the river, dense glacial till is present at shallow depth. The north approach spans and the three northernmost piers supporting the main bridge spans are founded on dense glacial till. The southernmost pier (Pier S2) supporting the main spans is supported on steel piles driven into the till through deposits of Fraser River silts and sands. The south approach spans are supported on 10 m to 12 m long timber piles end bearing on compact river sand.

The dense glacial till has a very low risk of liquefaction. Site-specific SHAKE (Schnabel et al., 1972) analyses indicated that a considerable portion of the loose river sand deposit located to the south of the river has a high risk of liquefaction corresponding to a 1 in 475 year return period earthquake. The piles supporting Pier S2 and the south approach spans are located within the zone of potentially liquefiable sands (see Fig. 1).

The potentially liquefiable sand deposit raised two key issues that had to be addressed in the design of the geotechnical seismic safety retrofit of the bridge. First, due to the proximity of the zone of potential liquefaction to the sloping river bank, large lateral ground displacements and unacceptable movements of the pile foundations and bridge piers could occur as a result of the design earthquake. Second, liquefaction of the sand deposit could result in loss of bearing capacity and/or unacceptable post-earthquake settlements of the pile foundations supporting the south approach spans.

In order to provide input to the design of the ground treatment program, the magnitude of anticipated lateral ground displacements were evaluated using a combination of empirical and finite element modelling methods (Wijewickreme, Atukorala and Fitzell, 1999). The liquefaction-induced "free-field" ground surface displacements were initially evaluated using the empirical Multiple Linear Regression (MLR) method proposed by Bartlett and Youd (1992). "Free-field" ground displacements were then computed using the finite element computer program SOILSTRESS (Byrne & Janzen, 1981 and Byrne et al., 1992).

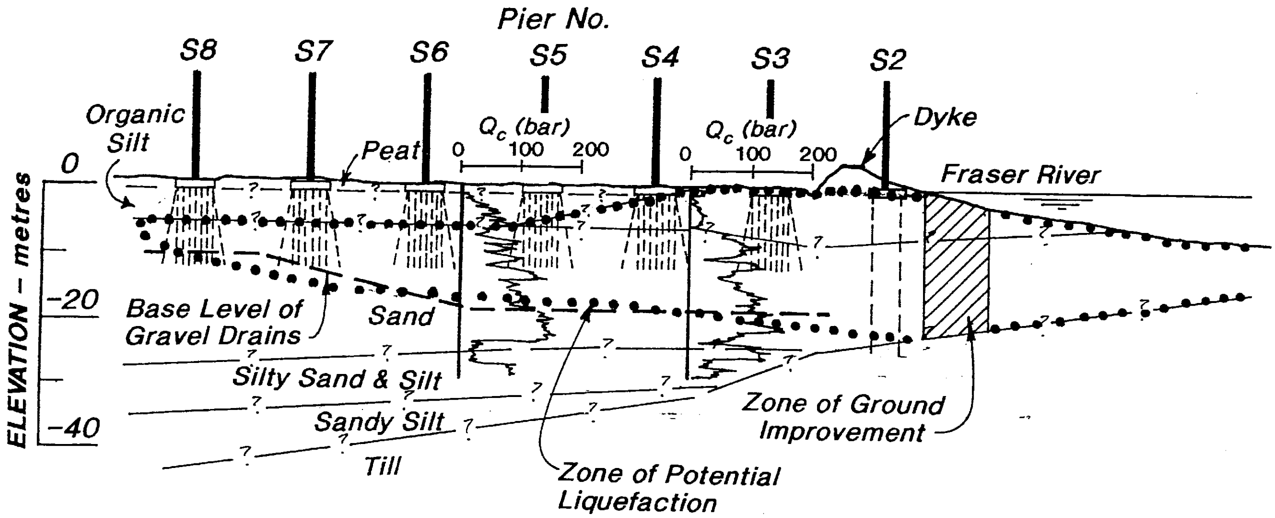

Fig. 1 Zone of Liquefaction and Areas of Ground Improvement – Profile

The results of the initial SOILSTRESS model, which ignored the presence of the bridge foundations, were then calibrated using the displacements predicted by the MLR approach. Finally, the bridge foundations and proposed ground improvement barriers were included in the SOILSTRESS analyses to assess the impact of these man-made features on the earthquake-induced ground displacements. Various ground improvement barrier configurations were modelled to allow the cost-effectiveness of the ground improvement to be optimized.

The modelling indicated that in order to limit relatively shallow slip surface failures in the vicinity of Pier S2, the zone of ground improvement work should be located as close as practically possible to the pier. The selected ground improvement configuration consisted of a single, 40 m long by 10 m wide zone of ground densification or a "seismic dyke" oriented parallel to the river and extending down to the dense non-liquefiable till deposit (see Figs. 1 and 2). The finite element modelling indicated that this barrier, located directly north of Pier S2, was sufficient to reduce lateral ground displacements and prevent excessive movement of the bridge foundations in the vicinity of the riverbank. The results of the SOILSTRESS analyses also indicated that ground densification barriers greater than 10 m in width were not cost-effective in reducing ground displacements.

Pier S2 is located on the river side of the Fraser River flood protection dyke. In order to install a ground densification barrier on the north side of the pier, it was required that ground improvement work be carried out within the river channel. Following review of environmental concerns associated with working within the river channel, vibro-replacement was selected as the method of ground improvement best suited to the site.

A temporary gravel berm was constructed within the river channel to allow construction equipment/personnel access to the work area. The gravel berm also acted as a source of stone backfill for vibro-compaction. Silt curtains were installed to reduce the risk of silt deposition in the adjacent environmentally sensitive areas of the riverbank downstream of the work area. The vibro-replacement stone columns were installed in a grid pattern at 2.75 m centre to centre spacing. The effectiveness of the ground densification was determined by cone penetration testing carried out at the centroid of the triangular zones formed by the stone columns.

Additional remedial measures were undertaken to improve the bearing capacity of the foundation soils of the south approach piers (Piers S3 through S8) which was considered marginal under existing conditions. In this regard, a series of 600 mm diameter vertical gravel drains were installed around each of the piers to expedite dissipation of excess pore water pressures and thereby increase the factor of safety against bearing capacity failure under seismic loading. The drains comprised pea gravel and they were installed at the locations shown in Fig. 2. The material selection for the drains was based upon a review of several filter criteria. The drain locations were based on conventional gravel drain design criteria, accessibility, and engineering judgement.

The gravel drains were installed using vibro-compaction equipment. Migration of the Fraser River sand into the gravel drains during installation was identified as a potential mechanism that could reduce the effectiveness of these drains during an earthquake event. To determine the extent of sand migration into the drains, post-installation sampling of the gravel was conducted using a sonic drill rig for several test stone columns

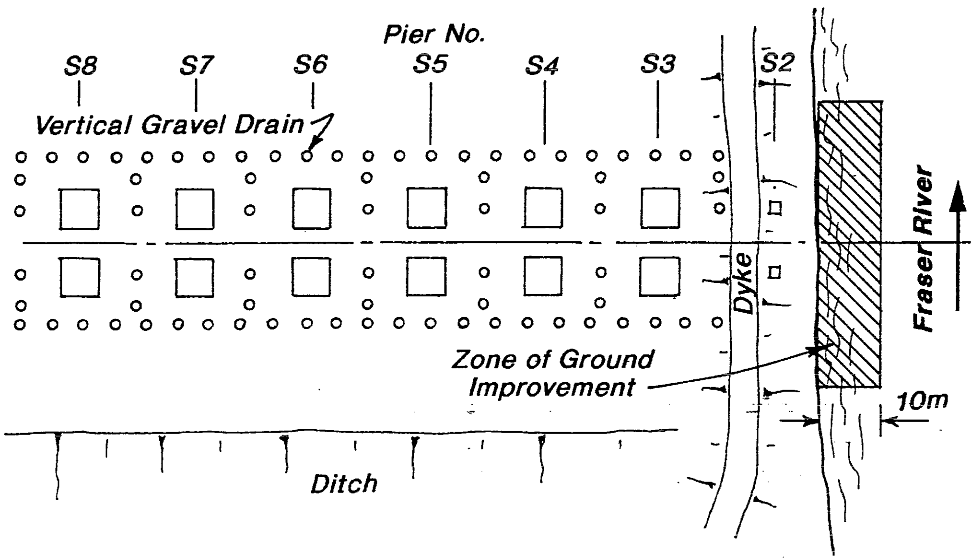

Fig. 2 Area of Ground Improvement and Locations of Gravel Drains – Plan

installed in the vicinity of the bridge. Gradation analyses carried out on vibra-core samples of the stone column backfill obtained before and after installation are shown in Fig. 3. Examination of the pre- and post-installation gradations indicates that gravel in the centre of the vertical drains was virtually free of contamination from the surrounding Fraser River sand deposit.

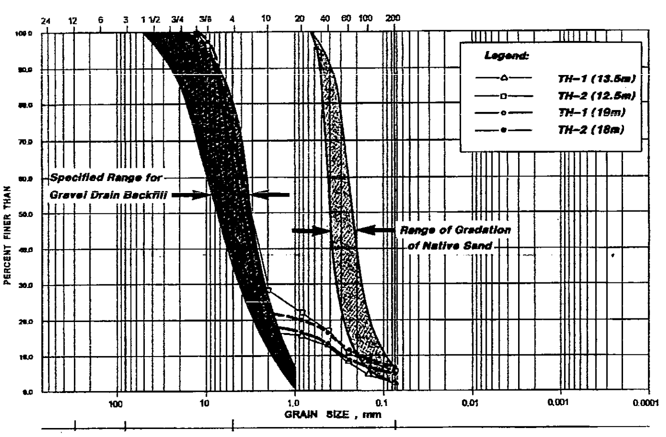

Fig. 3 Gradation of Backfill in Gravel Drains After Installation

Case history No. 2 – Ground improvement underneath and around the Cold Box tower

This case history describes the foundation protection measures implemented at the BC Gas Liquefied Natural Gas Plant (LNG) which is part of their natural gas supply and distribution system for Greater Vancouver. The LNG Plant liquefies and stores natural gas during the summer months for re-gassification and supply to the distribution system during the cold winter months when demand for natural gas is high. During gas liquefaction, natural gas is fed into a liquefier unit which consists of a series of heat exchangers housed inside a 21 m high, 4 m long, 2.5 m wide box-shaped steel-framed tower known as the Cold Box. The Cold Box is considered the "heart" of the LNG plant. It is a lightly-loaded structure with a working bearing pressure of approximately 35 kPa. A series of pipelines, ranging from 50 to 300 mm in diameter, enter and exit the tower at different elevations above its base. The foundation comprises a 0.75 m thick shallow reinforced concrete raft that is about 6 m by 8 m in plan area.

The Cold Box is underlain by 1 m of granular fill over deltaic deposits that comprise 5 to 6 m of silt/silty sand/sandy silt over more than 20 m of Fraser River sand. The sand is underlain by a thick deposit of marine silt extending to depths in excess of 75 m. The groundwater table is influenced by tidal variations and is located at a depth of 1 to 2 m below ground surface. The results of geotechnical engineering analyses indicated that there is a high risk of liquefaction of soils underlying the foundation of the Cold Box to an average depth of 22 m (see Fig. 4). In order to minimize the risk of liquefaction of foundation soils, densification of the soils was considered as a mitigative measure.

Ground improvement methods that could be utilized at this site were limited due to the following site and operational constraints:

(a) only 2 m of headroom was available for equipment;
(b) the Cold Box was very sensitive to both total and differential settlements;
(c) the area of ground improvement was located in close proximity to settlement and vibration-sensitive structures; and,
(d) it was required that BC Gas commences liquefaction of natural gas shortly after completion of ground improvement.

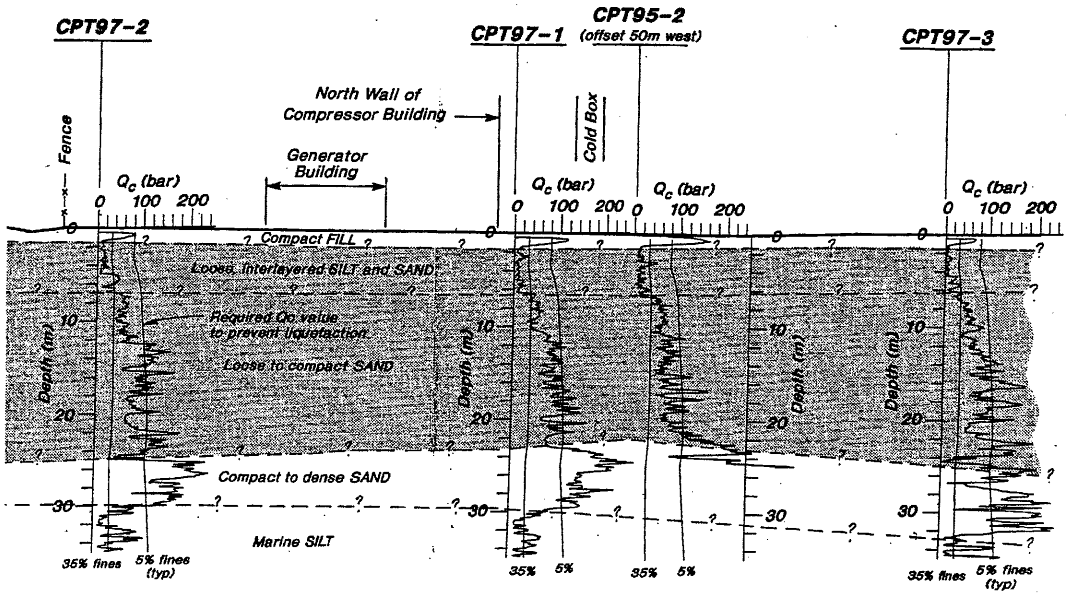

Fig. 4 Soil Stratigraphy and Zone of Liquefaction at Cold Box

Risk of damage to the foundations and equipment was not acceptable to the owner. This was in part due to the fact that there were uncertainties associated with the possible already existing residual stresses in the various pipelines connected to the Cold Box as a result of movements that had already occurred over time. The key pipelines connected to the Cold Box were insulated and visual inspection of their connections involved extensive effort and disruption to the plant processes. It was also determined that a small rotation of the foundation would be amplified by up to 3 times at the pipe connections located at different elevations.

In consideration of the above constraints including the need for strict control of the foundation movements, the method of compaction grouting was considered the most suitable approach. The ground improvement was undertaken to cover a plan area of about 12 m by 12 m as shown in Fig. 5. In compaction grouting, soil is displaced laterally by injection of a low-mobility aggregate grout pumped in stages through 100 mm nominal diameter steel casings. The process involves installation of steel casings with drop-off conical tips into the ground using a percussion hammer. Grouting is carried out in stages with careful monitoring of both the grout take and pressure as the steel casings are pulled out in increments of 0.3 to 0.6 m.

The grout pressure and grout-take were reduced within the upper 10 m to minimize heaving of the ground surface and nearby foundations. During grouting, the foundation of the Cold Box and other nearby settlement-sensitive foundations were monitored using 6 tilt meters connected to a centralized data logger and optical surveying of about 20 locations.

The grouting and monitoring locations are shown in Fig. 5. The grouting pressure, volume of grout pumped with depth, and the consistency of grout were carefully monitored and the sequence of installation of grouting points was selected/adjusted to minimize potential differential foundation movements during compaction grouting. Laser targets were also installed in the vicinity of grouting locations that would trigger an alarm if the movements exceed 3 mm.

Ground improvement commenced by grouting 6 primary holes that were located within the foundation and 2 additional holes located immediately adjacent to the edge of the foundation. The progress of grouting was slow occurring over a period of about 6 days for these initial holes. This was due to the fact that the casings penetrated to the full depth of 24 m and that there was high grout take over each depth increment. Based on grout volumes injected, it was inferred that grout columns ranging in diameter from 0.2 to 0.6 m were formed in-situ. During this period, the tilt meter movements and the measured settlements were generally small although some changes were noted. Grouting of the remaining holes outside the foundation footprint were carried out relatively quickly over the next 2 to 3 days as the ground tightened and the casings could not be advanced to the full depth in these holes. Also, an attempt was made to expedite completion of at least the primary grout columns so that operation of the Cold Box could commence as scheduled. At the end of this period, the tilt meters indicated that the foundation had rotated. Optical survey of the foundation confirmed that the foundation settled differentially with settlements of 13, 18, 9 and 12 mm recorded at the NW (S11), NE (S13), SW (S9) and SE

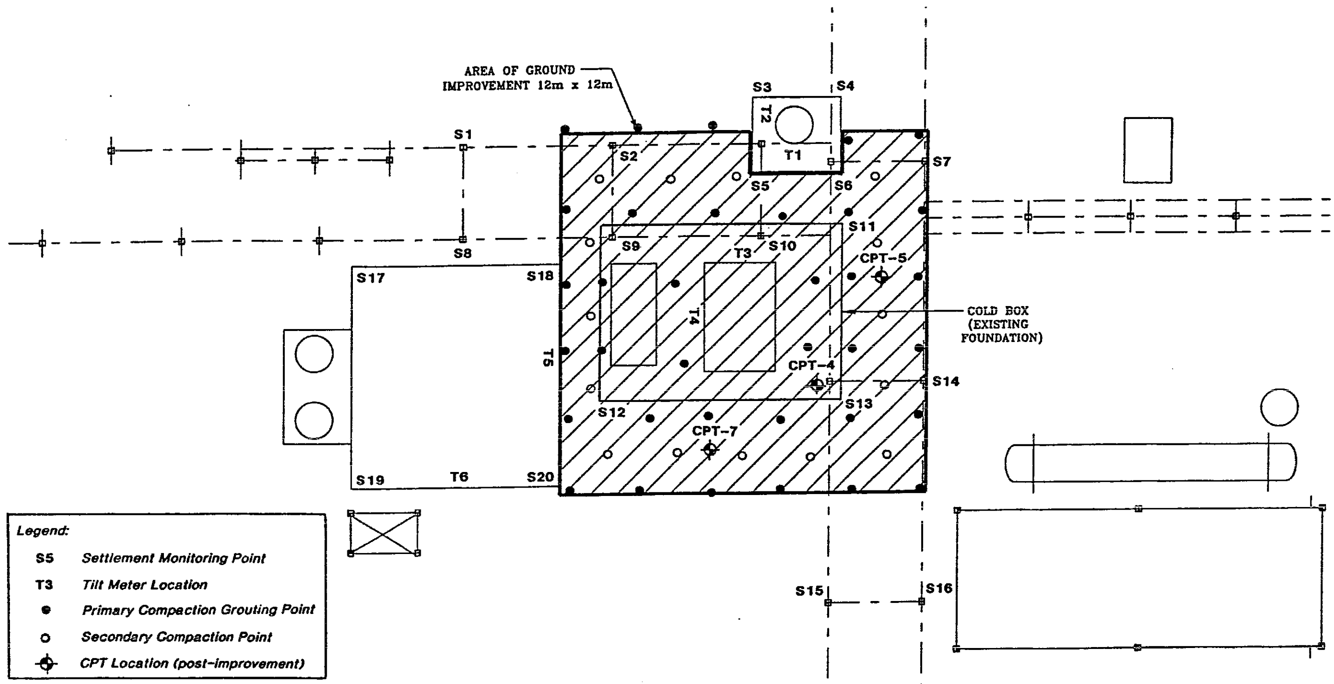

Fig. 5 Area of Ground Improvement and Monitoring Points for Compaction Grouting

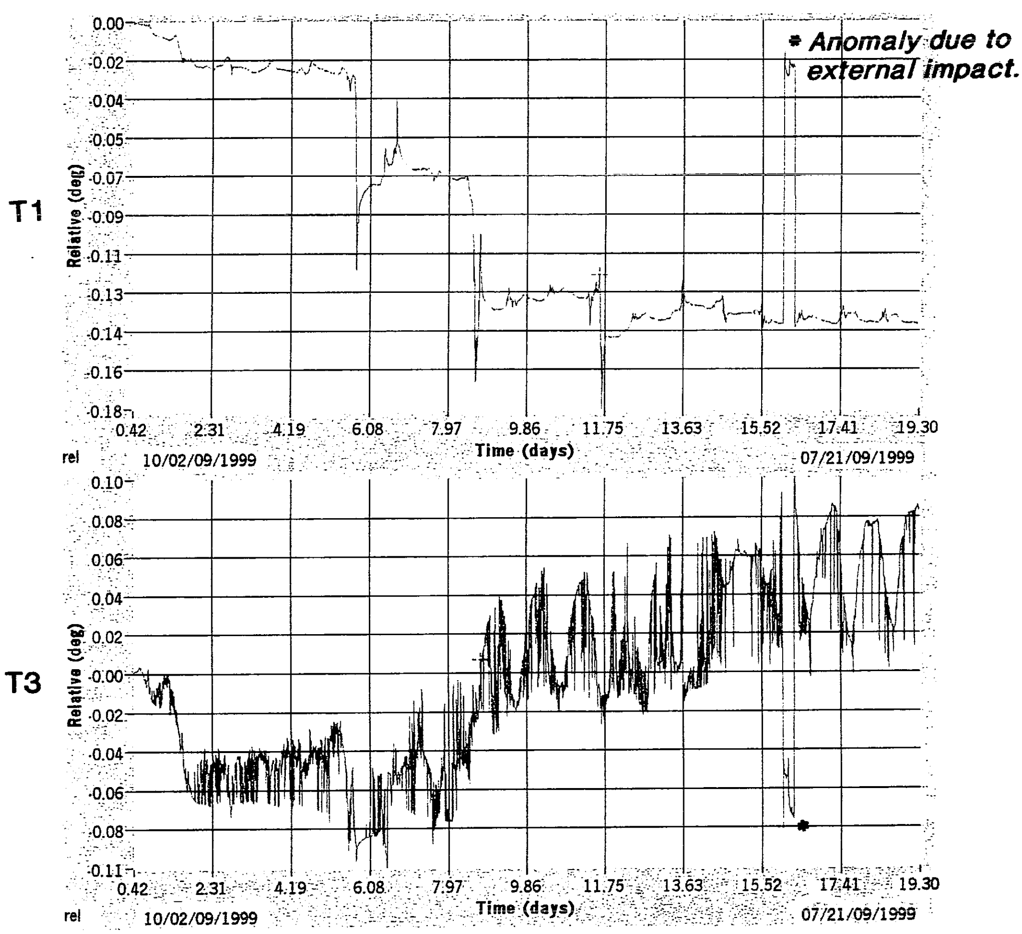

Fig. 6 Tilt Meter Recordings from T1 and T3

(S12) corners, respectively. Typical tilt meter recordings for T1 and T3 are shown in Figs. 6a and 6b, respectively.

It was postulated that the shear-induced excess pore water pressures, that developed as a result of the rapid rate of installation of the grout columns, softened the 5 to 6 m thick zone of silt/sandy silt beneath the foundation and, in turn, may have caused the settlements. The rate of installation of the compaction grout columns and the type of soil treated were determined to be critical factors that contributed to the generation of pore water pressures. Cone penetration tests, including pore pressure dissipation testing, carried out immediately after the settlements were noted were not successful in detecting any significant residual excess pore water pressures in the silt/sandy silt/silty sand layer. These tests however indicated that the grout penetrated beyond the anticipated radius about the point of grouting suggesting that there may have been fracturing of soil due to the high grout pressures.

The ground improvement work was stopped immediately and a structural evaluation was undertaken to estimate the extent of possible damage to the critical pipelines connected to the Cold Box. The structural assessment, which consisted of both visual inspections and engineering analyses of the critical pipelines, concluded that the foundation settlements that have already occurred did not pose a risk for operation of the Cold Box following completion of the ground improvement program.

Subsequent grouting was carried out with a lower slump grout and at a much slower rate with increased level of monitoring of the foundation movements. The ground improvement program was successfully completed on schedule and on budget.

Post-ground improvement analysis of grout volumes indicates that there was close to 6% volume change in the potentially liquefiable sands encountered between 6 and 15 m depth. Below 15 m depth, the average change in volume was about 2%. Within the upper silt/sandy silt and silty sand, the estimated change in volume was about 3%. This relatively low change in volume of the silty soils was anticipated in the original design.

A typical plot showing the comparison of the pre- and post-improvement cone penetration results is shown in Fig. 7. The results indicate considerable improvement of the cone tip bearing resistance in the area of primary interest.

In addition to liquefaction mitigation of foundation soils, engineering analyses indicated that the bearing capacity of the Cold Box foundation under seismic loading needed to be increased. This was achieved by a combination of densification using compaction grouting and installation of reinforcing steel within the upper 9 m of the grout columns that were located outside the foundation footprint.

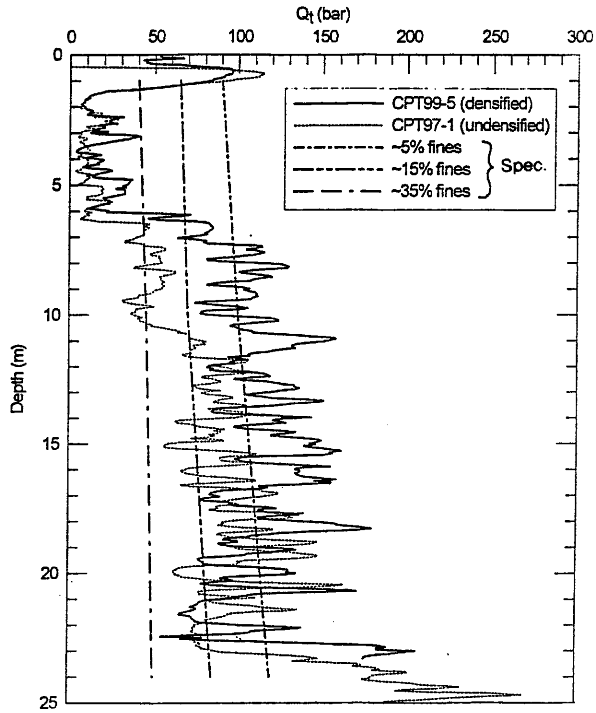

Fig. 7 Pre and Post-Improvement Cone Profiles

Case history No. 3 - Ground improvement for a natural gas transmission pipeline

This case history relates to ground improvement work carried out to minimize the seismic risk to a new natural gas transmission pipeline proposed by BC Gas Utility Ltd. The pipeline owner set an acceptable seismic risk criterion of less than 0.05 percent per annum for loss of gas pressure integrity.

As a part of their seismic upgrading program, BC Gas is in the process of replacing a key segment of an existing 500 mm diameter gas transmission pipeline across the Fraser River using an 914 mm diameter pipeline crossing to be installed using the method of horizontal directional drilling (HDD). As part of the upgrading project, an existing pipeline valve station situated approximately 250 m to the north of the river bank was also to be expanded to include additional piping components to allow for future "pigging" of the new pipeline (see Fig. 8).

Several alignments were evaluated for the segment of the pipeline to be installed using HDD techniques. Due to construction and site constraints, including concerns about working in close proximity to the existing pipeline, the selected alignment for the NPS 36 pipeline was offset about 60 m from the existing NPS 20 pipeline alignment on the north side of the river (see Fig. 8). Therefore, the tie-in connection of the new NPS 36 pipeline to the existing NPS 20 pipeline required an approximately 60 m long segment of pipeline to be oriented in an approximate east-west direction as shown in Fig. 8. Geotechnical

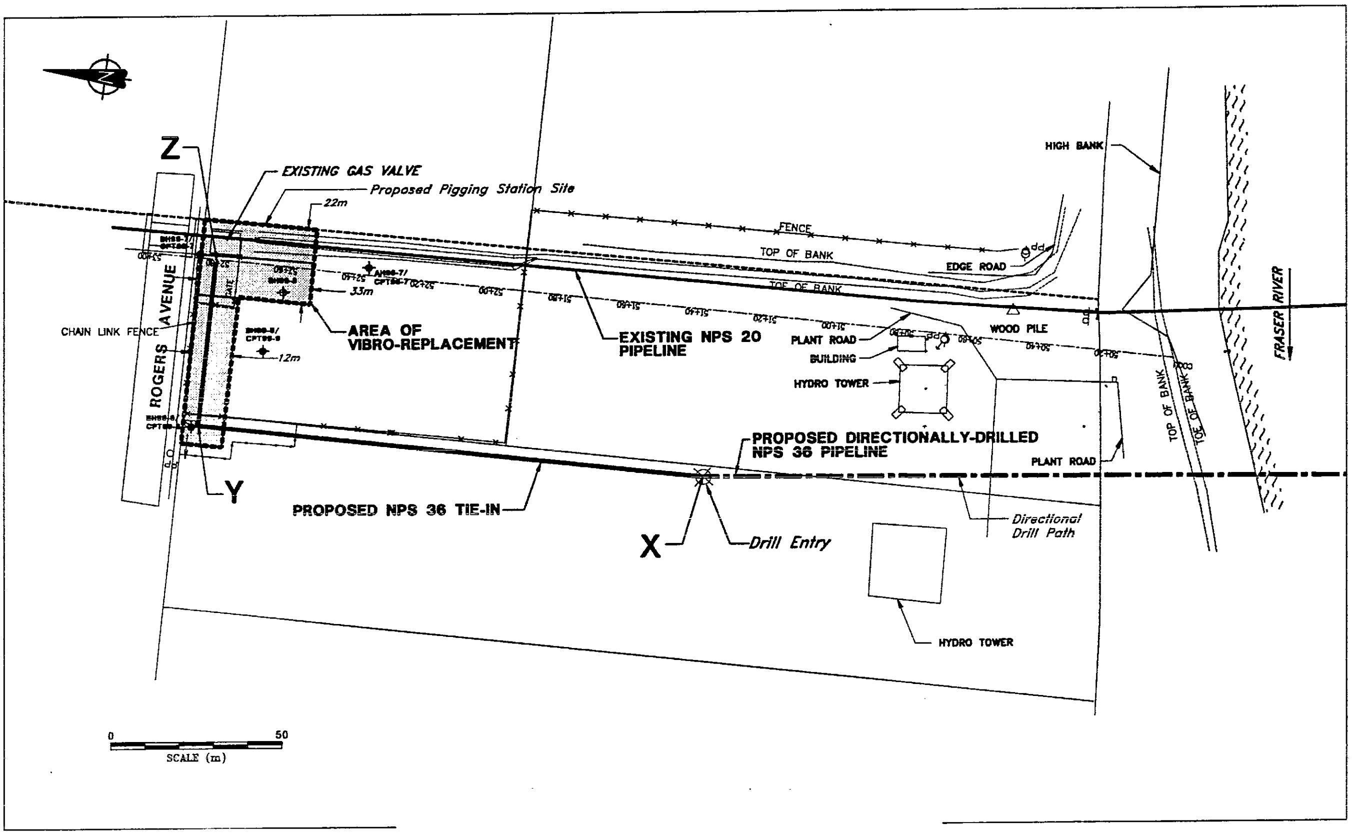

Fig. 8 Area of Vibro-Replacement

investigation, ground response and soil-pipe interaction analyses were carried out in order to determine the anticipated earthquake-induced geotechnical hazard, and the risk of pipeline damage due to the anticipated ground displacements.

The soil stratigraphy to the north of the river was determined based on a field investigation program consisting of electronic cone penetration testing (CPT), auger sampling and mud rotary drilling with Standard Penetration Testing (SPT). The soils at the site consist of relatively thin layers of surficial fill and overbank silts overlying an extensive deposit of Fraser River sand containing occasional silt and/or gravelly layers. Based on SPT and CPT resistance values, the upper portion of the sand deposit, extending to depths of approximately 17 to 18 m below ground surface, was determined to be compact while the remaining lower portion of the deposit is compact to very dense. The sand deposit extends to a depth of approximately 45 m below ground surface and is underlain by a thin deposit of very stiff clayey silt which in turn is underlain by a deposit of very dense gravel. Based on data from mud rotary boreholes put down within the river channel and available geotechnical information from nearby sites, glacial till is anticipated to be located at a depth of approximately 60 m below ground surface at the site.

Site-specific ground response analyses corresponding to a 2,000 year return period level (i.e. predicted peak horizontal firm ground accelerations of 0.36 g, respectively) were carried out using the computer program SHAKE. An earthquake magnitude of M7 was considered suitable for use in the liquefaction assessment. These analyses indicated that the upper portion of the Fraser River sand deposit at the site had a high risk of liquefaction corresponding to the design earthquake event. The zone of potential liquefaction identified by the analyses extended to depths in the order of 17 m below existing ground surface.

The magnitude of liquefaction-induced ground surface displacements at various distances from the river were evaluated using the empirical MLR approach developed by Bartlett and Youd and used as input into the structural evaluation of the seismic performance of the pipeline. These displacements were also cross-checked in relation to the results of detailed finite element analyses carried out to compute earthquake-induced lateral ground displacements for an adjacent site.

The liquefaction-induced ground movements at the site are expected to be oriented predominantly towards the river (i.e. perpendicular to the flow of Fraser River). As such, the north-south segment XY of the pipeline as shown in Fig. 8 would be subjected to primarily axial loading as a result of the anticipated ground movements, and the east-west segment YZ of the pipe connecting to the pigging station would be subjected to ground displacements perpendicular to the pipeline alignment. Structural seismic pipeline evaluation, including soil-pipe interaction analyses carried out using the ANSYS finite element program (a product of Swanson Analysis Systems Inc.), indicated that the east-west segment of the pipeline tie-in connection would be subjected to large lateral loads as a result of the liquefaction-induced ground displacements. The evaluation indicated that the relative movement between the elbow at point X and the anticipated movement of the east-west segment YZ of the pipe elbow due to soil loading would lead to the development of large strains at the pipeline elbow and potential rupture of the pipeline.

Having identified the earthquake vulnerability, several mitigative options were explored. These included:

(a) use of higher grade and thick-walled pipe;
(b) isolation of the pipeline from ground movements using a concrete box culvert and sliding pipe supports; and
(c) local ground improvement to mitigate liquefaction-induced ground displacements.

Structural analyses concluded that Option (a) above was not a cost-effective approach to achieve the owner's risk criterion. Option (b) was considered to be a viable option from an engineering perspective. However, this option was not considered favourable due to operational considerations such as pipeline maintenance and corrosion protection. The Structural pipeline assessment indicated that the pipeline strains would remain in the elastic range if the anticipated lateral ground movements at the east-west segment of the pipeline YZ could be limited to less than about 0.25 m. This level of performance was considered achievable by preventing liquefaction of the upper portion of the sand deposit. As such, densification of the liquefiable soil beneath the east-west segment of the pipeline (Option (c)) to limit ground displacements was selected as the preferred seismic upgrading solution.

The existing NPS 20 pipeline was required to remain in service during the ground densification. Therefore, ground improvement methods such as dynamic compaction and blast densification which produce significant ground deformations and vibrations during construction were ruled out as potential densification techniques. Vibro-replacement was selected as the preferred ground improvement technique for this site. Stone columns were installed in a triangular grid pattern to a depth of about 17 m over a 12 m wide strip area parallel to the proposed east-west pipeline segment YZ as shown in Fig. 8. The densification zone was extended to stabilize the footprint of the proposed "pigging" station to reduce the risk of damage to the piping components resulting from earthquake-induced differential settlements and/or ground movements.

The design of the ground densification work required stone columns to be installed within 3 m of the existing pipeline. Prior to the initiation of the ground improvement work, pipeline markers were installed on the existing NPS 20 pipeline at several locations. Continuous surveying of the pipeline markers was carried out while stone columns were installed in the immediate vicinity of the pipeline to ensure that the ground displacements and settlements were within the tolerable limits of the NPS 20 pipeline.

Electronic cone penetration testing was carried out at several locations within the densification area prior to the commencement of the ground improvement work. Cone penetration (CPT) verification testing was conducted at various times during construction to monitor the effectiveness of the densification. The results of post-densification CPT work indicated that the cone tip resistance (Q_t) values generally exceeded the prescribed performance Q_t criteria (ranging between 100 and 125 bar for clean sand).

Conclusions

Lifeline systems provide basic services that are fundamental to the day-to-day operations of our society. Given the essential services that these systems provide, the protection of these facilities from earthquake-induced damage has become important to both the public and the lifeline system owners.

Ground improvement techniques are increasingly being utilized to improve the post-earthquake performance of lifeline systems by reducing liquefaction-induced ground displacements and improving the bearing capacity and settlement characteristics of soils supporting the lifeline facilities.

This paper demonstrates how ground improvement has been carried out to protect key components of lifeline systems at three sites in the Fraser Delta underlain by liquefiable soils. The case histories also highlight how the selection of the appropriate ground improvement technique is dependent not only on cost but also on site constraints, environmental concerns and considerations related to the operation of the lifeline systems.

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