SEISMIC SAFETY RETROFIT OF THE PORT MANN BRIDGE NORTH APPROACH

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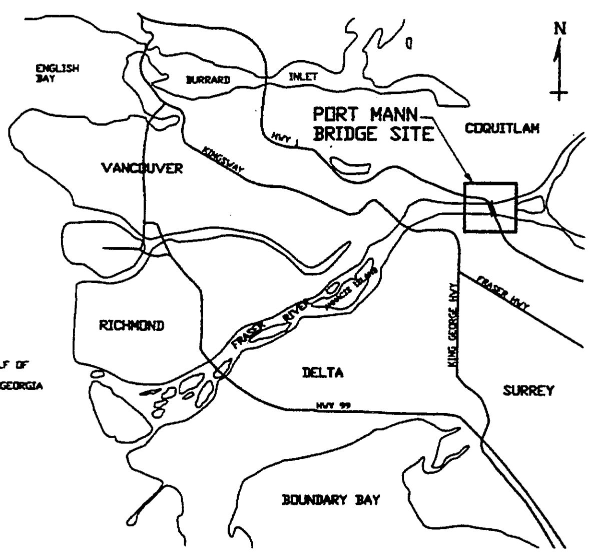
Abstract: Port Mann Bridge is a key structure of the Trans-Canada highway within the Greater Vancouver traffic grid. It is approximately 1.5m long and crosses the Fraser River between Surrey and Coquitlam. The bridge was built in the 1960's over loose and soft deltaic soils, zones of which may liquefy during a major earthquake. The bridge foundations are varied, and include expanded base concrete piles, steel pipe piles, precast concrete piles, and spread footings. The north approach piers have large pile caps founded on vertical and battered expanded-base concrete piles. These piles have inadequate shear capacity and lack ductility. Retrofit schemes to upgrade the foundations with new pipe pile foundations were reviewed, but deemed to be too costly. The proposed retrofit is to allow the piles to break during the earthquake, following which the existing pile caps would act as spread footings. Foundation retrofit include driving of timber compaction piles adjacent to the piers, and installation of seismic drains. With the retrofit, pier displacements during the design earthquake are predicted to be less than 0.3m. This paper describes the analyses method, results of the analyses and proposed retrofit methods.

Introduction

Port Mann Bridge is a key structure of the Trans-Canada highway within the greater Vancouver traffic grid. It is approximately 1.5m long and crosses the Fraser River between Surrey and Coquitlam (Fig. 1). The bridge was built in the 1960's (Hardenberg, 1961; Davie, 1964) over loose and soft deltaic soils, zones of which may liquefy during a major earthquake. The bridge foundations are varied and include expanded base concrete piles, steel pipe piles, pre-cast concrete piles, and spread footings.

Fig. 1. Bridge location plan



The B.C. Ministry of Transportation has recently widened the bridge deck from four to five lanes and a seismic safety retrofit work is in progress (Kirkwood, 2001). The objective of the seismic safety retrofit is to prevent collapse of the bridge during the design earthquake and to allow subsequent repair. A seismic retrofit strategy study was completed in 1995 (Buckland & Taylor Ltd., 1995; Chang et al., 1995), and seismic retrofit final design was carried out between 1997 and 2001.

This paper describes the geotechnical analyses methodology, results, and design recommendations for the seismic retrofit of the thirteen north approach piers (Fig. 2).

Site description and soil profile

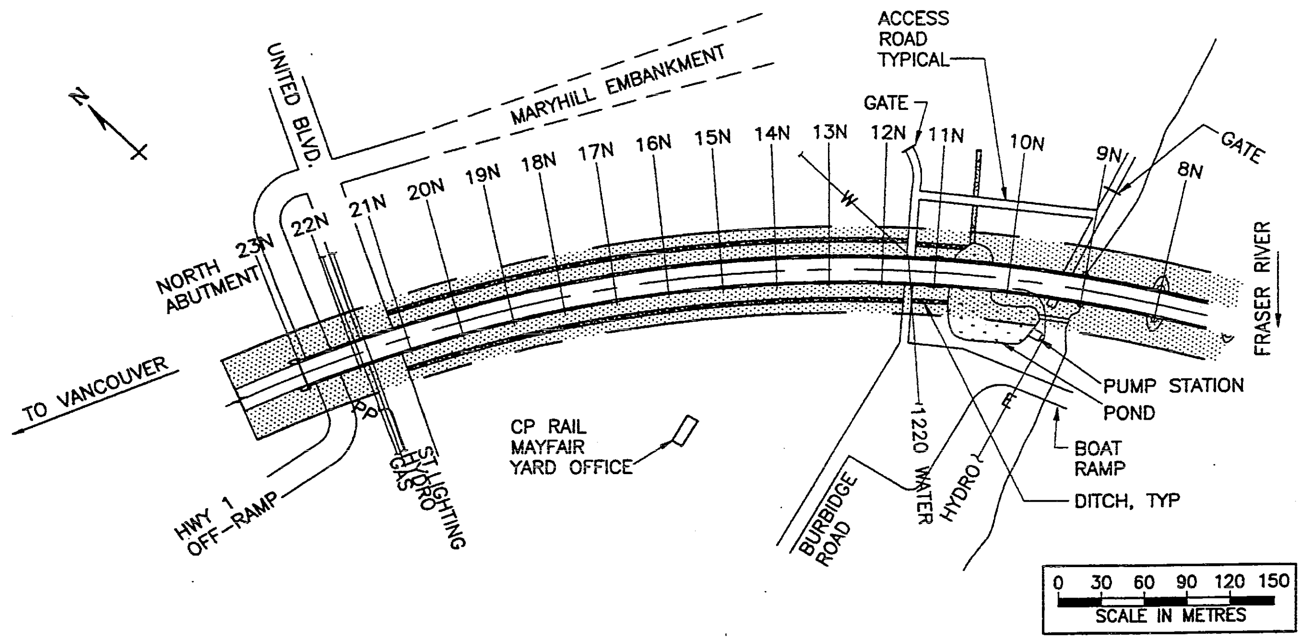
The north approach piers are in the northern flood plain. Formerly this was a marsh and flat farmlands, but it is now used for warehouses, light industrial structures, roadways, and a park. Approximately 2.5 m of fill has been placed over the marsh and farmland. Current grade is approximately elevation 5 m geodetic. At the river edge the flood plain is dyked. Two large ditches of 2.5 to 4 m depth flank the bridge right-of-way. The ditches drain to a pond and pump station near the river edge. An approximately 11m high roadway approach fill for the Mary Hill Bypass is located to the north-east of the right-of-way. The north abutment of the bridge is founded on an earthen approach fill of approximately 14 m in height. A cross-section of the bridge with soil profile is shown in Figure 3.

In the vicinity of the north approach piers the soil profile is typical of the Fraser River delta. As shown in Figure 3, underlying the sand fill (Unit 0) is a silt crust (Unit 1B) of 3 to 5m thickness over loose to dense sands (Unit 3) of 21 to 30m thickness, over 2 to 5 m of clayey gravel (Unit 4B); over firm clays (Unit 5B) of 6 to 15 m thickness, over very dense Pleistocene till-like soils (Unit 6) at 30 to 50m depth. A typical cone penetration test profile with liquefaction thresholds is shown in Figure 4.

North approach super-structure and foundations

The part of the bridge over the northern flood plain is 580 m long and supported by13 piers (Piers 10N to 22N, where 10N

Fig. 2. On-land north approach structure of the bridge



is near the river edge and 22N is near the north abutment). Each pier has two concrete columns on a single pile cap, varying in size from 5.5m by 18m to 9m by 22m. The pile cap is supported on concrete expanded base (Franki) piles.

The central piles in the pile groups are vertical, while the perimeter piles were battered at inclinations between 1:6 and 1:10.

The piles have a 508 mm (20 inch) diameter concrete shaft. Shear reinforcement is nominal, typically 6 mm wire on 250 to 300 mm pitch. Axial reinforcement varies from five 19 mm bars for the vertical piles to three 19 mm bars plus two 25 mm bars for the battered piles.

Pile bases are founded in the upper part of the Unit 3 sand layer at depths of 9 to 13m. The original allowable design load for the piles is 890kN.

The north abutment is a single spread footing of 8m by 13m, founded on a granular embankment over the deltaic deposits.

Seismic upgrade design

The seismic upgrade design is significantly influenced by:

(1) the weak and loose deltaic soils at the bridge site. These soils have the potential for amplifying the earthquake ground motions, liquefaction with its related consequences, foundation bearing failure, and excessive foundation displacement.

(2) the structurally weak and brittle foundation piles. The expanded base concrete piles only have nominal shear reinforcement, and therefore, are weak in shear. Shear failure of the pile would be sudden and non-ductile. Perimeter piles are battered outward, which makes the piers stiff and thus they will attract load, both from structure induced inertial forces and from differential movements of the ground. The load may be sufficient to fail the piles in shear and/or compression.

The scope of work included:

- ground response analysis to assess soil amplification affects, to obtain time histories for dynamic analysis, and to obtain maximum cyclic stresses within the ground for liquefaction assessment.
- liquefaction assessment.
- calculation of pier stiffness for structural dynamic analyses.
- evaluation of pier bearing capacity and potential displacement.
- design of mitigation measures as required.

Ground response analyses

Three sets of firm ground earthquake time histories were provided by MoT for the seismic analyses (Cherry, Anderson and Nathan, CAN 1995). These consisted of:

- two sets of orthogonal time histories which were fitted to match an equal hazard spectrum, representative of a 10% probability of being exceeded in 50 years (or 475 year return period). The peak ground acceleration for the firm ground motion is 0.21g. For liquefaction assessment and ground displacement analyses these are deemed to be magnitude 7 events with an epicentral distance of 84 km.
- one set of orthogonal histories representative of a magnitude 8.25 subduction earthquake with an epicentral distance of 120 km. The subduction event has a peak ground acceleration of 0.17g.

Detailed ground response analyses using 1D equivalent-linear analysis program SHAKE (Schnabel et.al. 1972) (Idriss and Sun, 1992) and non-linear analysis program MARDES (Chang et. al., 1990) were carried out during the

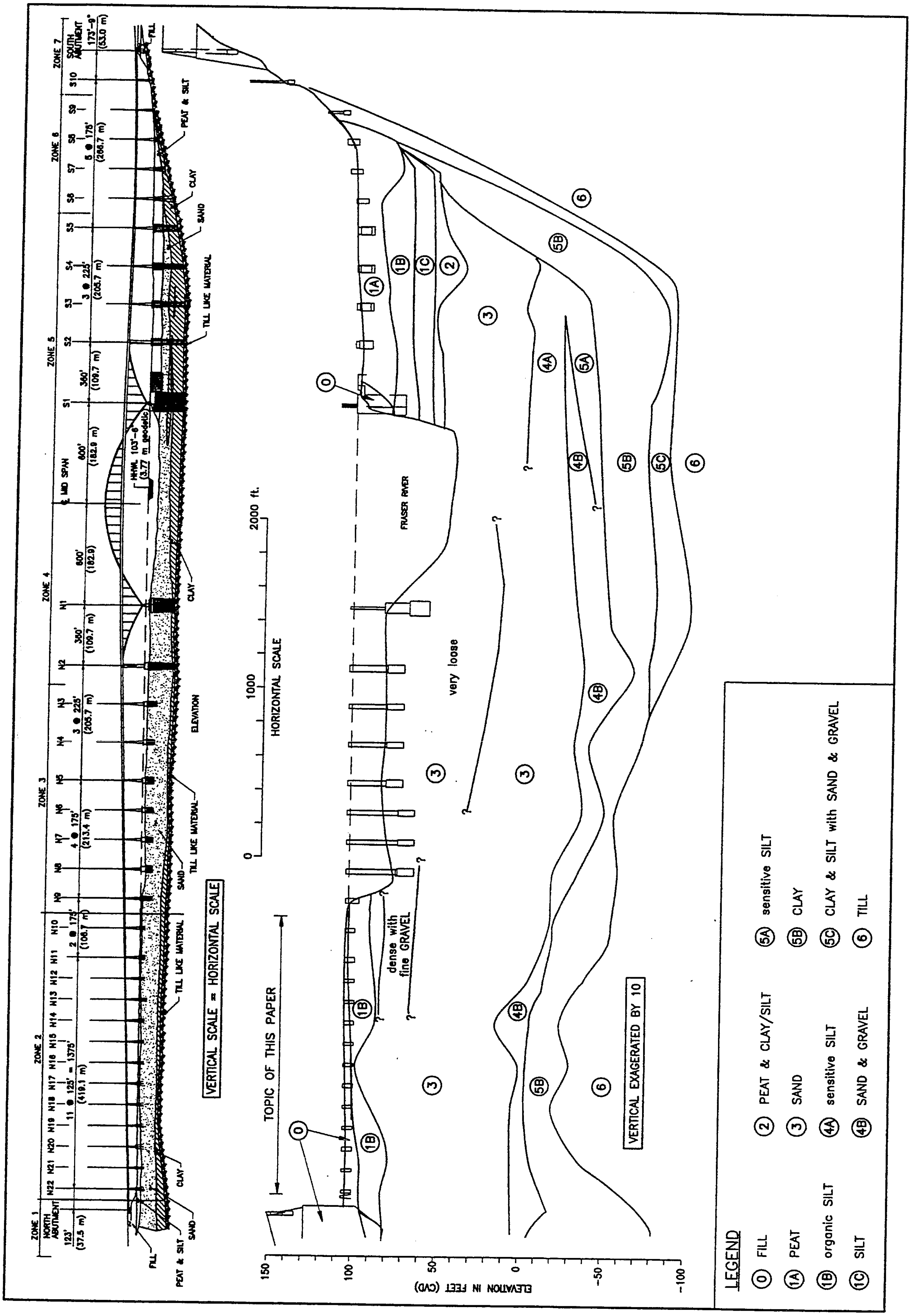


Fig. 3. Bridge and sub-soil profiles with pier arrangement

initial strategy phase of the work. Near surface time histories from the above analyses were obtained for dynamic structural analyses. Cyclic stress ratio profiles were also developed for liquefaction triggering assessment. Vertical ground motions for dynamic structural analyses were taken as 2/3 of the firm ground horizontal motions.

Ground response analyses indicate that in the vicinity of the north approach the firm ground acceleration is amplified by 20 to 30% at the surface.

During the final design phase of the work additional ground response analyses were conducted using SHAKE to obtain time histories at 40m depth. These were used as input motions (Fig. 5) in 2D dynamic soil-structure interaction analyses using the computer program FLAC (ITASCA, 1998).

Fig. 4. Typical Cone Penetration Test (CPT) data

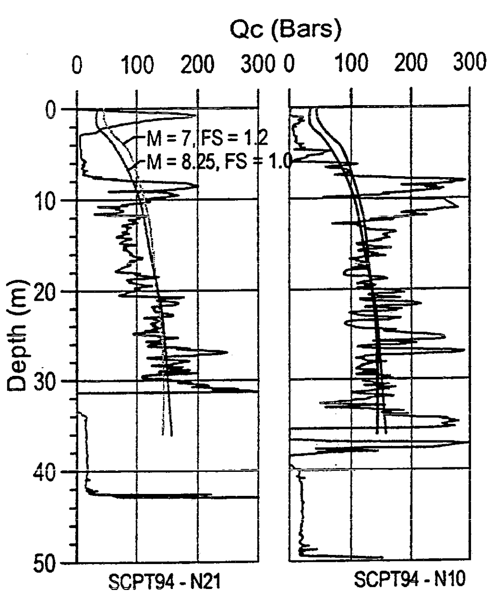
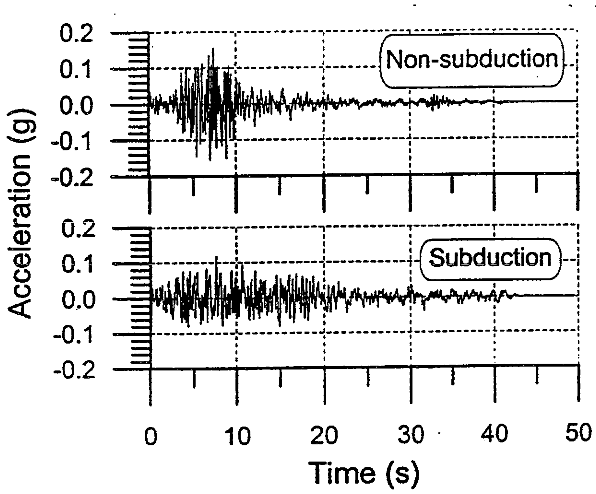


Fig. 5. Time history of input motion for dynamic analyses



Liquefaction assessment

Liquefaction assessment was carried out using the procedures of Seed et al. (1982). Cyclic Resistance Ratio (CRR) was determined using the recent electric cone penetration tests (CPT) data and standard penetration test (SPT) data. Dutch mechanical cone penetration test data from the original bridge investigation was also used in combination with the CPT data.

The recent test holes were drilled by MoT crews. The drilling was of high quality and in strict accordance with the recommendations by Seed et. al.(1985). Hammer energy measurements were made to correct the SPT N-values to N_{60} at intervals in the hole.

Tip bearing resistance (Q_t) from the cone penetration tests were converted to $(N_1)_{60}$ values for use in the liquefaction assessment. The $(N_1)_{60}$ derived from CPT data with $Q_t/(N_1)_{60}$ of 5 agreed reasonably with the data from SPT. Tip bearing resistance from the original Dutch mechanical cone tests correlated well with that of the recent CPT tests.

A factor of safety of 1.2 and 1.0 against liquefaction was used for the non-subduction and subduction events respectively. The assessment indicates that in general, the severity of liquefaction in the vicinity of the north approach piers 10N to 22N is sporadic and marginal. Localized loose zones are present near pier 21N, however densification from the installation of the pile bases is expected to reduce the extent of liquefaction in this area.

Stiffness coefficients for dynamic structural analyses

Elastic spring stiffness coefficients of the pile foundations (for structural analysis) were initially developed using the computer program GROUP (ENSOFT, 1999). GROUP models three-dimensional pile groups as linear elastic structural elements connected with a rigid pile cap. Soil is modelled using non-linear P-Y and T-Z springs.

Vertical, longitudinal, transverse and rotational spring coefficients were obtained using GROUP. In the structural analysis model displacement time histories obtained from ground response analyses were applied to the ends of the vertical, longitudinal and transverse springs.

Pier capacity & displacement analyses

At many of the north approach piers (10N to 22N) the structural demand exceeded the capacity of the relatively brittle concrete expanded-base piles and it was deemed that they would fail. A typical failure envelope of the piles as developed by the structural consultants is shown in Figure 6.

The retrofit concept was to install large diameter pipe piles around the existing piers to provide support in the event of failure of the existing expanded base piles (Fig. 7). This scheme was found to be very costly and an alternative scheme of letting the piles break and using the pile cap as a spread footing was pursued.

Allowing the piles to break would result in sliding, rocking, and settlement of the piers due to ground shear and post-seismic consolidation. Design requirements were that the bridge structure must be able to tolerate both the inertial earthquake loads (which would be worst prior to pile failure) and the displacements, which may occur if the piles do break.

Bearing capacity, stability and displacements following pile breakage were assessed by three methods: (i) limit equilibrium analyses with the pile cap acting as a spread footing and ignoring the reinforcing affect of the pile shafts within the ground, (ii) numerical pseudo-dynamic push-over analyses with soil and structural elements using the program FLAC, and (iii) a dynamic analysis with soil and structural elements using FLAC. The effects of potential soil liquefaction, the adjacent ditches, the adjacent Mary Hill Bypass embankment, and the river edge were included in the analyses.

Fig. 6. Axial load - moment domain failure envelope for expanded base piles

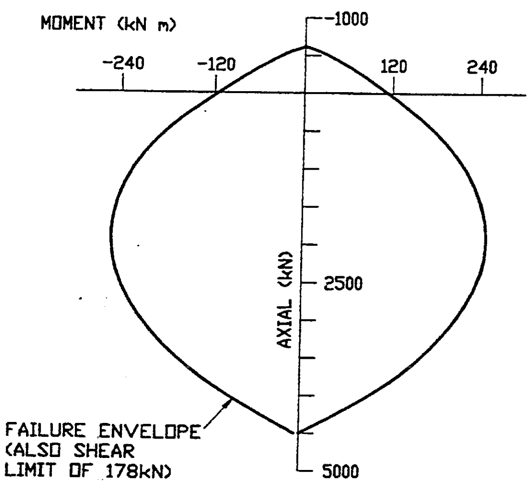
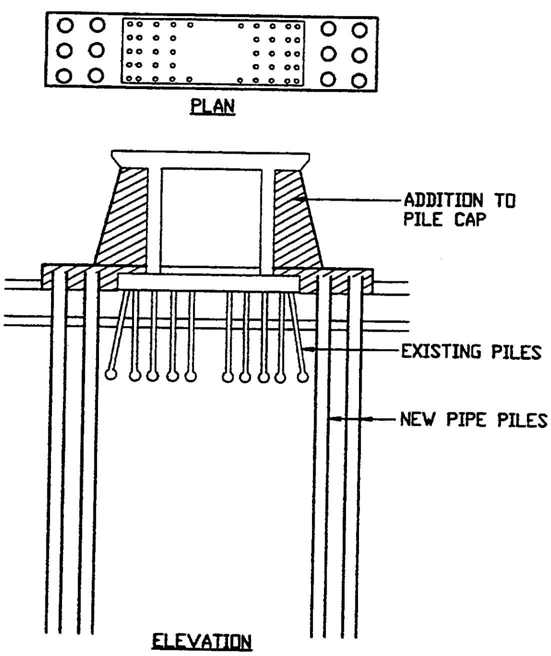


Fig. 7. Mitigation with large diameter pipe piles



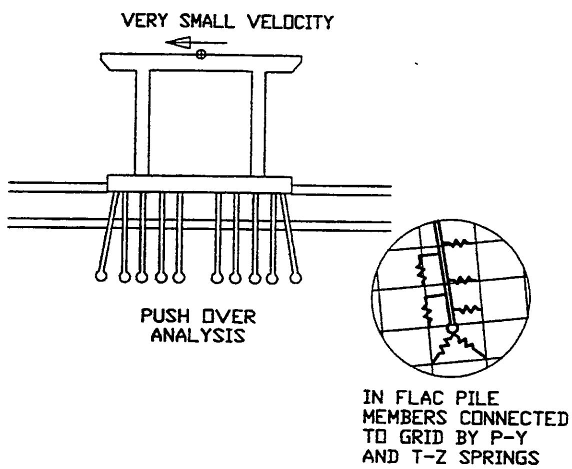
Limit equilibrium stability analyses

Bearing capacity and stability of the foundation soils were first assessed using the Terzaghi general bearing capacity formula and 2-D slope stability program XSTABL (Interactive software designs, 1996). In these analyses the existing pile caps were assumed to act as spread footings bearing on the silt crust and the reinforcing effects of the existing piles were ignored. Shear strength of the silt crust was assumed as 30 to 35 kPa. For the stability analysis vertical loading from the superstructure was applied as pressure over the width of the pile cap. A factor of safety (F.S.) of 1.1 to 1.2 against bearing and slope failure was obtained under static loading with no sub-soil liquefaction. F.S. dropped to 0.6 with the assigned shear strength of zero for the liquefied loose sands below the silt crust. However further numerical analyses with more detailed stratigraphy and with zones of soil liquefaction indicated acceptable deformations and factors of safety above one, as described in the following sections.

Static push-over analyses

Non-linear pier stiffness and capacity relationships were developed with a static push-over analysis using the program FLAC. The top of the pier was pushed back and forth by applying a small velocity at the top of the pier as shown in Figure 8. The three dimensional aspect of the piles was modelled by having p-y and t-z springs between the structural pile elements and the grid. Axial, bending moment, and shear demands in the structural elements and horizontal, vertical and rotational displacements of the pile cap were monitored. If, at any time, the stresses in the pile element exceeded the structural failure envelope (Fig. 6) then that element was deleted.

Fig. 8. Push-over model

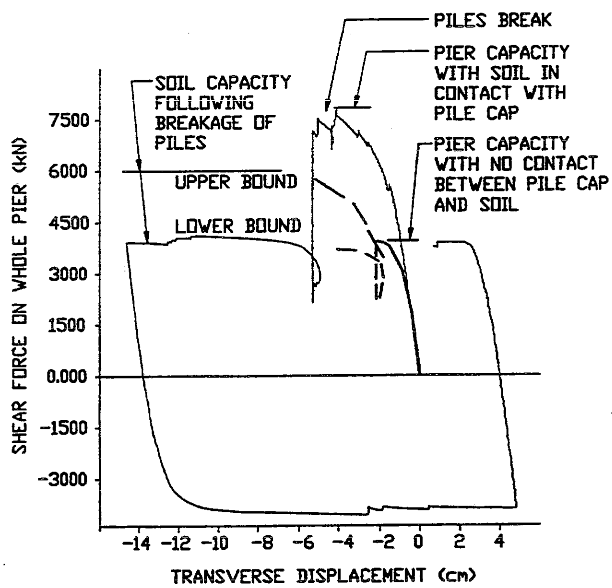


In some of the analyses liquefaction in the layers underlying the silt crust was triggered during the push-over. Liquefaction was triggered by: setting the vertical stress equal to the horizontal stress; softening the shear modulus; and assigning residual shear strength for the liquefied soil. Figure 9 shows the results from push-over analyses in the transverse direction.

Conclusions from the numerical push-over analyses are:

- Pier stiffness calculated from FLAC prior to pile breakage and when the bottom of the pile cap is not in contact with the ground are same as those calculated using the program GROUP. (GROUP cannot model pile breakage or the contact of the bottom of the pile cap with ground).
- Lateral load capacity of the pier prior to pile breakage increases by 25 to 60 percent when the pile cap is in contact with the ground.
- The horizontal load capacity of the pier in the longitudinal direction (narrow direction) is limited by rocking of the pile cap, whereas in the transverse direction it is limited by sliding.
- Liquefaction of layers underlying and adjacent to the pier softens the response but does not reduce capacity.
- After pile breakage there is net vertical deformation of 10 to 50 mm with each cycle of loading.

Fig. 9. Results of push-over analyses



Dynamic numerical analyses

Soil-structure interaction was also studied using dynamic analyses with FLAC. In these analyses liquefaction triggering, the consequences of liquefaction, pier capacity, and pier displacements were assessed simultaneously. The total-stress-liquefaction triggering model UBCTOT (Byrne & Beaty, 1999; Beaty & Byrne, 1999; Beaty, 2001) was incorporated in the FLAC model. This model has been used previously on several projects within the greater Vancouver area (Uthayakumar & Naesgaard, 2000; Naesgaard & Yip, 2001)

Sensitivity analyses on soil properties, structural properties, and boundary conditions were carried out. Structural elements modelling the bridge piers and piles were included in the analyses. Section mass and stiffness of the structural elements were adjusted so that the natural period of the bridge structure and pile loading approximately matched.

The affect of zones of soil densification around the piers and other mitigative measures such as perimeter sheet piles and timber compaction piles were assessed.

A calibration of the FLAC model was done by (a) comparing one dimensional FLAC results to those obtained from SHAKE and by (b) comparing displacements from two dimensional (2D) FLAC analyses to those calculated using the method by Youd (1996).

A 2D dynamic FLAC analyses for the selected design sections was conducted as described in the following:

- Soil elements were first brought to equilibrium under static gravity loading;
- then the bridge structure was added and brought to equilibrium under static gravity loading;
- soil elements were then assigned with undrained dynamic properties and
- the analysis was continued by applying a time history of earthquake loading to the grid base.

Only the horizontal component of the earthquake motion was used in the analyses.

$(N_1)_{60}$ from the various test holes were used as input in the model. Local variation in $(N_1)_{60}$ was modelled by using a random number generator within FLAC with a pre-determined standard deviation. Then layering and smoothing of the random values was done by taking a four value running average in the horizontal plane.

In UBCTOT the number of cycles of shear stress within each element is monitored, converted to equivalent cycles of uniform loading and compared to the liquefaction triggering threshold. When the threshold is reached liquefaction is triggered by changing soil shear strength and moduli to post-liquefaction values. The dynamic analysis is then continued to the end of the earthquake record.

Figure 10 shows one of the design sections developed to assess the behaviour of the north approach piers. This section incorporated a structural model of a typical pier with pile foundation and proposed timber compaction piles. The effects of pile breakage, ditches adjacent to the pile caps, adjacent Mary Hill approach fill, river edge, and soil liquefaction were included in the model. Effectiveness of various alternative remedial measures was also assessed.

In the analyses the demands within the pile elements were checked at each time step in a similar manner to that conducted in the push-over analyses. If the demands in the pile elements exceeded the failure envelope then the element was deleted during the dynamic analysis. Over 20 dynamic

Fig. 10. Design section and FLAC grid

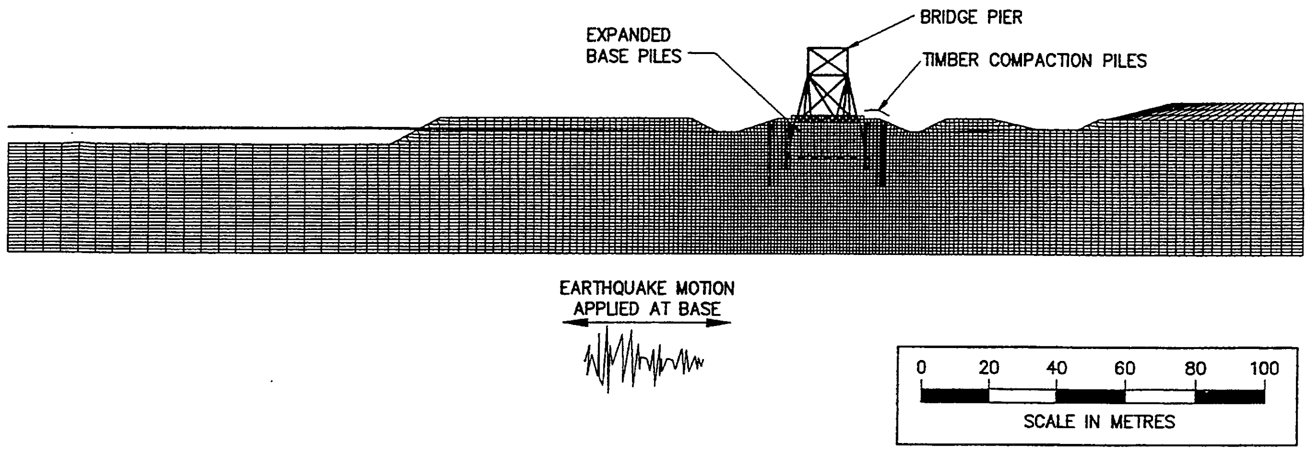


Fig. 11. Typical displacement time history of a pier with gap between pile cap and ground

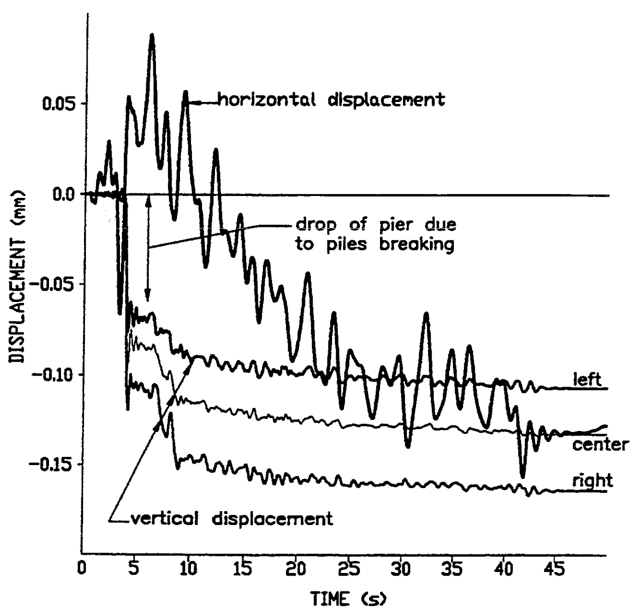
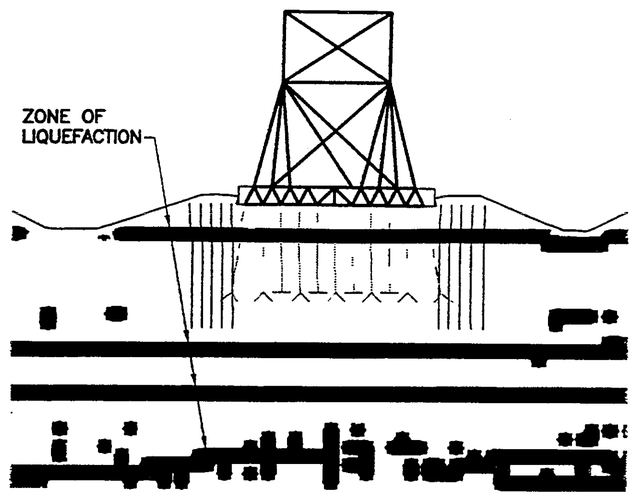


Fig. 12. Typical zones of liquefaction at the end of earthquake motion



time history analyses, each requiring 12 to 20 hours of computer time, were conducted. Figure 11 shows typical displacement time histories of a pier. Figure 12 shows typical zones of liquefaction present near the end of a dynamic analysis and Figure 13 shows the stress strain behaviour of an element which liquefied.

Discussion on the dynamic analyses

Analyses indicate that pile breakage is sensitive to the presence of a gap between the ground and the pile cap. If there is a gap then extensive breakage of the model piles would occur. Often the piles would not break if there was no gap between the pile cap and the ground. Recent test pit excavations at five of the north approach piers indicated no visible gap between the underside of the pile cap and the ground.

When the piles break, plastic soil deformation occurs at the edges of the piers due to pier rocking. However bearing failure of the pier does not occur after the pile break. Confining the soil at the edges of the pier by filling the adjacent ditches, by driving sheet piles around the pier perimeter or by driving timber piles adjacent to the pier reduced the vertical settlement of the pier.

Horizontal movement of the piers is strongly influenced by the continuity of the liquefiable layers below the pier and by ground slope from the river edge and adjacent Mary Hill By-pass embankment. In all analyses the calculated horizontal displacements and vertical settlements were less than 300mm. With the condition of no gap between pile cap and ground, and sub-soil densification (with timber compaction piles adjacent to the piers) calculated vertical settlement of the pier is less than 150 mm - even if the expanded base piles break.

Fig. 13. Shear stress-strain response of a typical soil element in the model showing pre and post liquefaction responses

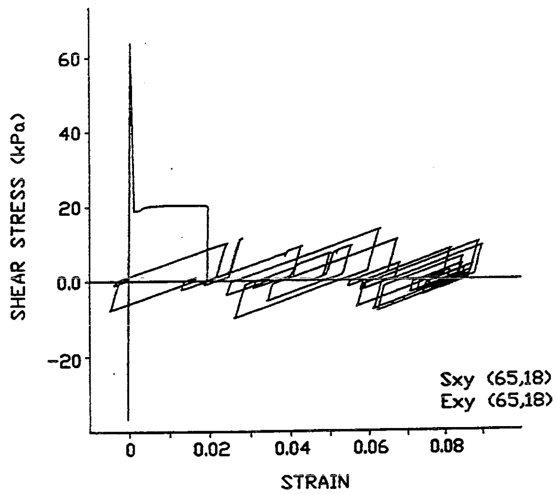
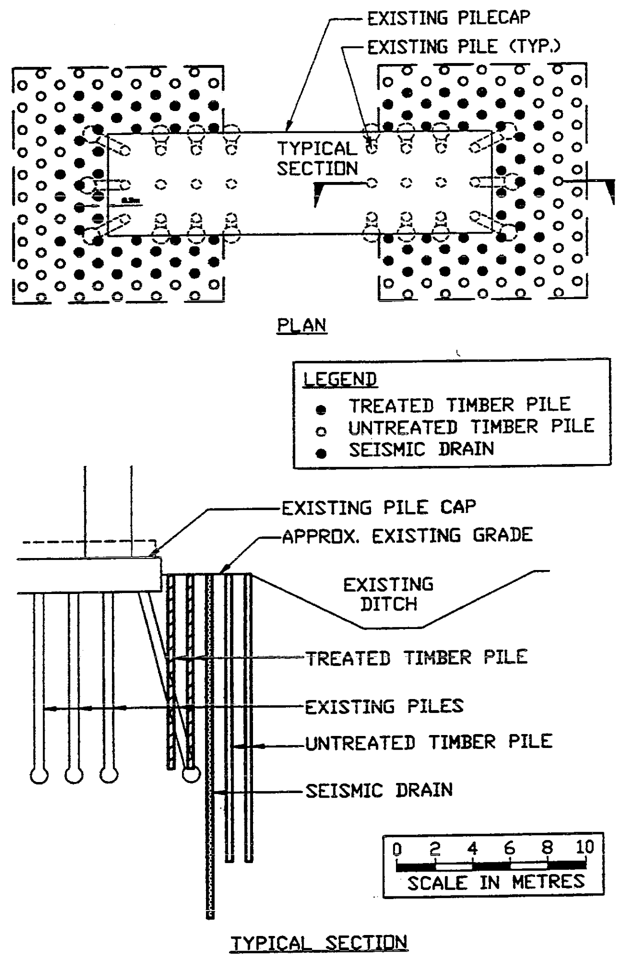


Fig. 14. Proposed retrofit with timber compaction piles and seismic drains



Post-liquefaction settlement

Post-liquefaction consolidation settlement of the ground near the piers was estimated using the procedure by Tokimatsu & Seed (1987). Settlement in the range of 75 to 250 mm was estimated. Post-liquefaction consolidation settlements would be additional to the shear induced settlements obtained from the dynamic FLAC analysis.

Design recommendations

Pier displacements

At piers 10N to 17N it is proposed to install timber compaction piles adjacent to the piers on 1.2m center-to-center spacing (Fig 14). The first two rows of piles adjacent to the piers would be treated piles while the outer rows would be untreated timber piles. The purpose of the timber piles is to densify the soil around the pier perimeter to prevent liquefaction and to act as dowels to confine the soil at the toe of the pile cap/footing. 16m to 20m deep seismic drains are to be installed within the timber compaction pile zone to allow for dissipation of excess pore-water pressure during and following the design earthquake. The seismic drains are to consist of a 250 mm diameter column of 1 to 9 mm sized gravel with a central slotted PVC pipe. At piers 18N to 22N the ditch adjacent to the bridge is shallower and the underlying silt crust is thicker. At these piers only a ring of seismic drains is recommended.

With the proposed ground densification and seismic drains the calculated lateral and vertical pier displacements are less than 300mm. For design, differential displacements between piers have been chosen as less than 200mm horizontally and 250mm vertically. With these displacements the bridge structure should not collapse. However, extensive repair may be required to the pile foundations.

Conclusions

The on-land north approach foundations have large pile caps founded on vertical and battered expanded-base concrete piles, which have inadequate capacity and lack ductility. Retrofit schemes to upgrade the pile foundations with new pipe pile foundations were reviewed, but deemed to be too costly. The proposed retrofit is to allow the piles to break during the earthquake, following which the existing pile caps would act as spread footings. This innovation will not provide the same performance as the alternative scheme of upgrading the pile foundations, but is significantly less costly and meets the intent of the seismic safety design criteria. Analyses of the soil-pile-pier system were conducted using several methods. From the analyses estimates of the earthquake induced pier displacements were made without and with various retrofit options. Foundation retrofits include driving of timber compaction piles adjacent to the piers, and installation of seismic drains. With the retrofit, pier displacements during the design earthquake are predicted to be less than 0.3m.

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