Golden Ears Bridge – Geotechnical seismic design aspects

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ABSTRACT
Golden Ears Bridge is a new six-lane cable-stayed bridge over the Fraser River in south-western British Columbia, Canada. The river crossing (main bridge) is 968 m and total length, including approaches is 2.6 km. Soils consist of soft to stiff clayey silts and silty clays to over 120 m depth, overlain by potentially liquefiable sands with varying thickness up to 36 m in places. The south approach and main span piers are founded on 2.3 to 2.5 m diameter cast-in-place concrete shafts of up to 85 m length. The north approach piers use driven 12 to 36 m long, 0.35 m square pre-cast piles for axial resistance and short, 5 to 6 m long, 0.9 m diameter bored cast-in-place shafts for lateral resistance. The seismic design criteria specifies no collapse for the 2475-year return period earthquake event, repairable damage for the 1000-year, and no significant damage (essentially elastic performance) for the 475-year events. Seismic design included determination of site-specific response spectra, ground motion time histories, assessment of liquefaction susceptibility and consequences, assessment of foundation stiffness and soil-structure interaction analyses and ground improvement design. Analytical procedures varied from the use of simple empirical methods, to the more advanced effective-stress soil-structure interaction, dynamic analyses using the constitutive model UBCSAND within the computer program FLAC.

RÉSUMÉ
Le pont Golden Ears est un pont de type haubanné passant au-dessus de la rivière Fraser, située au sud-ouest de la Colombie-Britannique. La portée principale, au-dessus de la rivière, est de 968 m et la longueur totale du pont, incluant les approches, est de 2,6 km. Le sol en place est constitué d'une argile silteuse à une silt argileux de consistance de molle à raide sur une profondeur de 120 m et reposant sur un sable potentiellement liquéfiable d'une épaisseur pouvant aller jusqu'à 36 m. Le pilier principal de l'approche sud repose sur une fondation profonde constituée d'un pieu en béton coulé en place d'un diamètre de 2,5 m et d'une profondeur de 85 m. Le pilier de l'approche nord est fondé sur plusieurs pieux préfabriqués de 12 à 36 m de longueur ayant 0,35 m^2 de surface pour développer la résistance axiale et des pieux courts, 5 à 6 m de longueur, de 0,9 m de diamètre en béton coulé en place pour résister au cisaillement. Le critère sismique de design pour un tremblement de terre contre la ruine, est d'une période de retour de 2475 ans, contre des dommages réparables, est pour une période de retour de 1 000 ans, et pour aucun dommage significatif la période de retour est de 475 ans. La complexité de l'analyse sismique inclut la détermination de la réponse spectrale du site, l'historique du mouvement du sol, l'évaluation de la susceptibilité à la liquéfaction et ses conséquences et l'évaluation de la rigidité des fondations et de l'interaction sol-structure pour un design optimisé. Les analyses utilisées pour ce projet varient de la simple méthode empirique à une analyse avancée dynamique de l'interaction des contraintes sol-structure utilisant un modèle constitutif de type UBCSAND avec le logiciel FLAC.

1 INTRODUCTION

The Golden Ears Bridge is a new six-lane cable-stayed bridge currently under construction in south-western British Columbia, Canada. The bridge spans across the Fraser River, connecting the cities of Surrey and Langley on the south, to Pitt Meadows and Maple Ridge on the north (Fig. 1). The main spans of the bridge are supported by an extradosed cable-stayed structure, while the approaches have continuous precast concrete girders supported on cap beam and column frames at each bent. There are four main piers in the river over a length of 968m. The total length of the bridge and approaches is 2.6 km. Figs. 2 and 3 show a general arrangement of the main crossing and an artist's rendition respectively. Fig. 4 shows the bridge under construction. Deep deltaic deposits of potentially liquefiable sands and near-normally consolidated clay/silt soils with depths greater than 120 m were found at the site.

The project is being built under a Design-Build Public-Private Partnership, and maintained for 35.5 years by the Golden Crossing Constructors Joint Venture (GCCJV) for the Greater Vancouver Transportation Authority (Translink). Key partners in the GCCJV are Bilfinger Berger and CH2M HILL. Buckland & Taylor Ltd. and Trow Associates Inc. are the structural and geotechnical consultants, respectively, to GCCJV for the major bridge structure. EBA Engineering Consultants Ltd. assisted with the seismic analyses for the South Approach structures. Extensive road-works and overpass structures are also part of this project and were designed by others. These other structures are not discussed in this paper.

The objective of this paper is to give an overview of the seismic design challenges, criteria, and methodology.

Figure 1 Golden Ears Bridge location

2 GEOLOGY, GROUNDWATER AND SEISMICITY

2.1 Geology

Published geological studies (e.g. Clague1998 and personal communication) indicate that the project site was under more than a kilometre thickness of glacial ice during the last glaciation, which ended approximately 10,000 years ago. This ice scoured much of the soil, which existed prior to the glaciations, within the current Fraser lowland and left the underlying sedimentary bedrock basin with local pockets of Pleistocene soils.

Figure 3 Artist's rendition of bridge (from http://www.translink.bc.ca/GoldenEarsBridge/)

Melting of the ice sheets at the end of the glaciation resulted in rebound of the basin and a gradual rise in sea level. The retreating ice sheets deposited a mantle of glacial soil (till, and outwash sands and gravels). A glacio-marine environment with a nearby ice margin followed, which with time became a marine basin. Over 100 m thick near-normally consolidated to lightly overconsolidated silt and clay with occasional sand interlayers occur within this basin at the site.

The Fraser River started its current course and formation of a delta with the retreat of the ice sheets. Between 5,000 and 10,000 years ago the Fraser Delta advanced from the east over the project site. Fluvial sands and gravels and sands, of up to 20 m to 35 m thickness, were deposited within the river channel while floodplain silts and minor organic soils were deposited outside the river channel. Sea level has been fairly constant over the last 5,000 years.

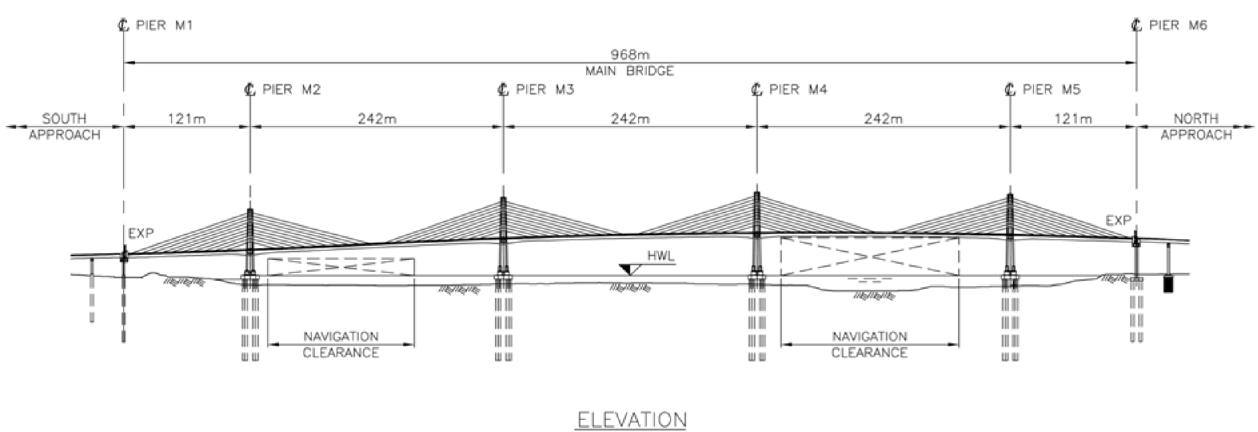

Figure 2 General arrangement of main river crossing

Figure 4 Bridge under construction (from http://www.translink.bc.ca/GoldenEarsBridge/)

The current water table level varies from the ground surface to approximately 1.5 m depth. Water pressures within the fluvial river sands are hydrostatic. Artesian pressures of approximately seven meter of head above hydrostatic, per 100 m depth were measured in the marine clays.

A soil profile at the bridge crossing is shown in Fig. 5.

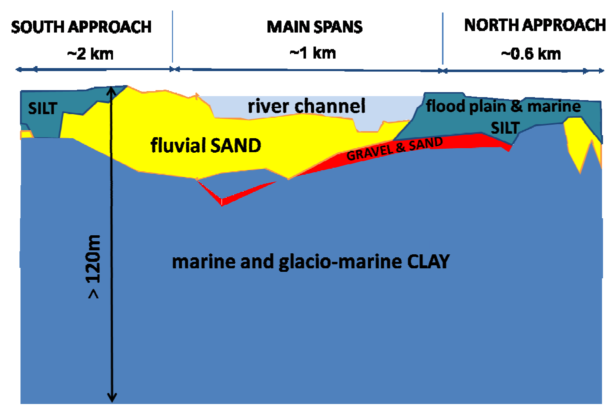

Figure 5 Soil profile at bridge crossing (vertical dimensions exaggerated by 10)

2.2 Seismicity

South-western British Columbia, where the bridge site is located, is an area of active seismicity. The major source of this seismic activity is the oceanic Juan de Fuca plate subducting under and compressing the continental North American plate. This results in three potential earthquake sources: near-surface (0 to 30 km) crustal earthquakes, deep (40 to 50 km) intra-plate earthquakes within the subducting plate, and large inter-plate subduction earthquakes. The first two sources are accounted for in a probabilistic seismic model and are the predominant hazard for the bridge site. The subduction earthquake is typically of larger magnitude but is at a significant distance (≈120 km) from the site and therefore, generally does not control design.

Outcropping firm-ground response spectra for the 475-, 1000- and 2475-year return period events, and for the deterministic subduction earthquake event are shown in Fig. 6. Sets of out-cropping firm-ground earthquake records in three orthogonal directions were fitted to the design response spectra by others and provided to the GCCJV for use in the design.

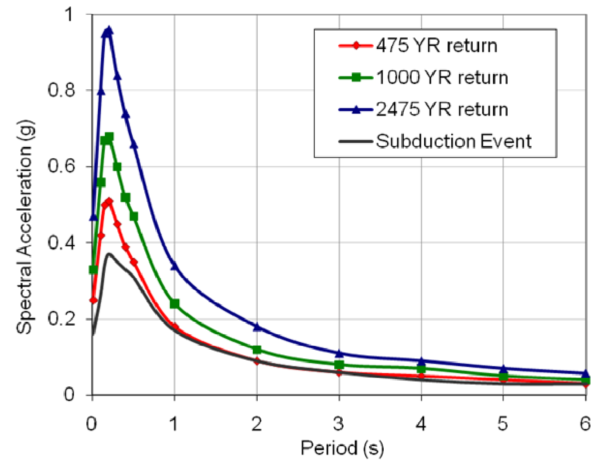

Figure 6 Firm ground (Class 'C' soil) outcropping design response spectra.

3 SEISMIC DESIGN CRITERIA

The seismic design criteria specified for the bridge were as follows:

475-Year Event - The bridge structure must not have significant damage and must be fully functional following the earthquake. Geotechnical resistance factors (ϕ_R) from the Canadian Bridge Code CSA-S6-00 were used for design. For axial pile loading, ϕ_R = 0.5 to 0.6 (with pile load tests) and for lateral loading ϕ_R = 0.5 was used.

1000-Year Event - The bridge structure must be repairable following the earthquake. For axial pile loading ϕ_R = 0.8 was used for pile foundations.

2475-Year Event - The bridge structure must not collapse. For design of axial and lateral capacity of foundations; ϕ_R = 1.0 was used.

Subduction Event - This deterministic design earthquake was only considered when addressing the consequences of soil liquefaction. The bridge must not collapse due to liquefaction consequences.

4 GROUND RESPONSE ANALYSES

Ground response analyses were carried out using three methods: (1) the equivalent-linear method with the program SHAKE91 (Idriss and Sun 1992), (2) a total-stress non-linear hysteretic model (UBCHYST), and (3) a combination of UBCHYST total-stress model for the clay soils and UBCSAND effective-stress model for the granular soils. The ground response analyses were carried out:

- to obtain near-surface design response spectra for structural design,
- to obtain near-surface and in-profile earthquake time histories for structural analyses, soil-structure interaction numerical analyses and Newmark displacement analyses, and,
- to assess triggering and consequences of liquefaction.

4.1 Hysteretic model

The non-linear hysteretic model developed at the University of British Columbia (UBCHYST) was run using the finite difference program FLAC 5.0 (Itasca 2005). A hyperbolic constitutive model with a Mohr Coulomb failure envelope is used. Shear modulus is a function of stress ratio as follows:

$$G = G_{max}\,(1 - R_f\,.\,\eta/\eta_f)^n$$

where

G	=	tangent shear modulus
G_{max}	=	low strain shear modulus
η	=	stress ratio (τ/σ' = shear stress to vertical effective stress)
η_f	=	stress ratio at failure
R_f	=	failure ratio (fitting parameter) set as 0.9[AA1]
n	=	exponent (fitting parameter) set as function of η/η_f (n ranges from 0.75 to 3.2).

Near-surface design spectra used for structural modal analyses are shown in Fig. 7. Fig. 8 compares typical near-surface (5 m depth) response spectra from the hysteretic (UBCHYST) and equivalent linear (SHAKE91) analyses. The equivalent-linear method with the program SHAKE91 was used to obtain the design spectra and time histories for the 475-, and 1000-year return period events while the hysteretic model was used for the 2475-year event due to the larger shear strains associated with this event.

4.2 Coupled Effective Stress UBCSAND model

A coupled effective stress constitutive model (UBCSAND) and analysis procedure developed at the University of British Columbia for modelling earthquake shaking and liquefaction was used. UBCSAND is an elasto-plastic effective-stress model with the mechanical behaviour of the sand skeleton and pore-water flow fully coupled (Beaty & Byrne 1998, Byrne et al. 2004). The model includes a yield surface related to the mobilised friction angle, non-associative flow rule, and definitions for loading, unloading, and strain hardening. Key elastic and plastic parameters were determined by back-analysis of laboratory element tests. Constant-volume simple shear tests were used for this purpose as the loading path and rotation of principal stresses reasonably simulates that which occurs during earthquake loading. The UBCSAND constitutive model was used within the finite difference program FLAC 5.0 (Itasca 2005). In the FLAC program, dynamic analyses are carried out in the time domain with full coupling between groundwater flow and mechanical loading.

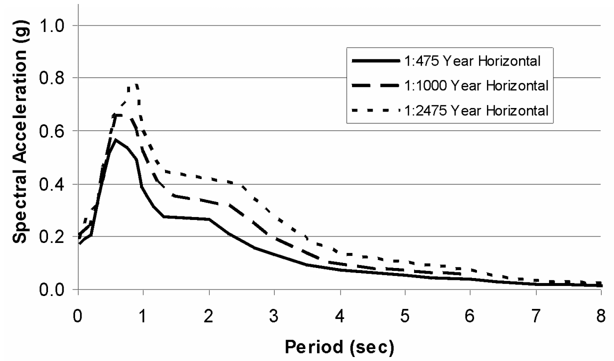

Figure 7 Near-surface design spectra for structural analyses.

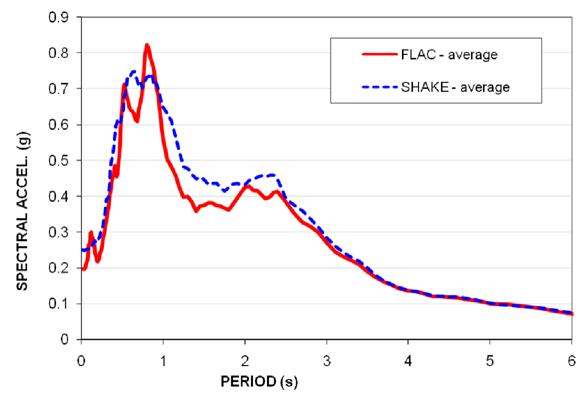

Figure 8 Typical comparison of average 2475-year event spectra from FLAC with non-linear UBCYST model to average calculated with SHAKE91 (Pier M3 profile).

The UBCSAND model has been calibrated against simple shear laboratory tests, centrifuge tests with and without impermeable silt barriers (Yang et al. 2004, Phillips et al. 2005, Seid-Karbasi et al. 2005, Byrne and Park 2005) and the empirical liquefaction triggering charts (Idriss & Boulanger 2007). The model is able to emulate both the observed drained behaviour of loose sand soils (i.e. contractive when sheared below the constant-volume friction angle, ϕ_{cv}, and dilative above it) and the build-up of pore pressure and soil liquefaction that occurs in undrained simple shear tests. The model is also able to emulate the behaviour of a flow failure when a low permeability barrier is present, and no flow failure when the barrier is absent or when drains are installed through the barrier (Byrne et al. 2006, Naesgaard et al. 2005) including emulation of the post-shaking failure of the Lower San Fernando Dam (Naesgaard et al. 2006, Naesgaard & Byrne 2007). Atigh & Byrne (2004) showed that UBCSAND could capture the behaviour of the triaxial tests with fluid inflow carried out by Vaid & Eliadorani (1998).

5 LIQUEFACTION TRIGGERING

Liquefaction triggering was assessed using two methods: (1) Seed's method (Youd et al, 2001) with cyclic stress ratio (CSR) derived from the equivalent-linear response analyses (SHAKE91); and, (2) two dimensional dynamic analysis with UBCSAND effective stress model. The effective stress method has the advantage of assessing liquefaction triggering, the consequences of liquefaction, effects of soil densification and soil-structure interaction all in one analysis. In general, liquefaction was predicted to be sporadic under the 475-year and subduction events; however, significant zones of liquefaction were predicted for the 1000- and 2475-year events.

6 SEISMIC ASPECTS OF FOUNDATION DESIGN

6.1 Introduction

The seismic design aspects generally involved:

- development of a near-surface design spectra and time histories for various pier locations and each of the design earthquakes;
- calculation of foundation stiffness using the P-Y / T-Z method and the programs LPILE Plus 5.0 (Ensoft 2004) or GROUP (Ensoft 2003) ;
- push-over analysis using the program FLAC with UBCSAND model to determine foundation stiffness and ultimate capacity;
- assessment of liquefaction triggering, ground densification requirements, river bank displacements and effects of ground movements on piles using dynamic numerical analysis with the program FLAC and UBCHYST and UBCSAND constitutive models;
- assessment of stresses in piles due to earthquake shaking and inertial loading from the structure (using program LPILE and/or GROUP); and
- assessment of axial capacity of pile to resist earthquake loading.

6.2 South Approach

The south approach is supported on 2.3 to 2.5 m diameter excavated and cast-in-place monopiles (one pile per column and no pile cap) of 30 to 75 m depth (Fig. 9a).

Key challenges of the South Approach design were the assessment of whether or not ground densification was required around piles passing through potentially liquefiable sands, and the determination of design spectrum and time histories at top of the pile for cases with liquefaction (for use in structural analyses). An existing 1.2 m diameter water main in the vicinity of the piers made densification difficult and costly. Detailed dynamic soil-structure interaction analyses were conducted. From these analyses, design response spectrum (Fig. 10), foundation stiffness and stresses in the piles were obtained for cases with and without ground densification. The analyses indicated that ground densification was not required around the large monopiles

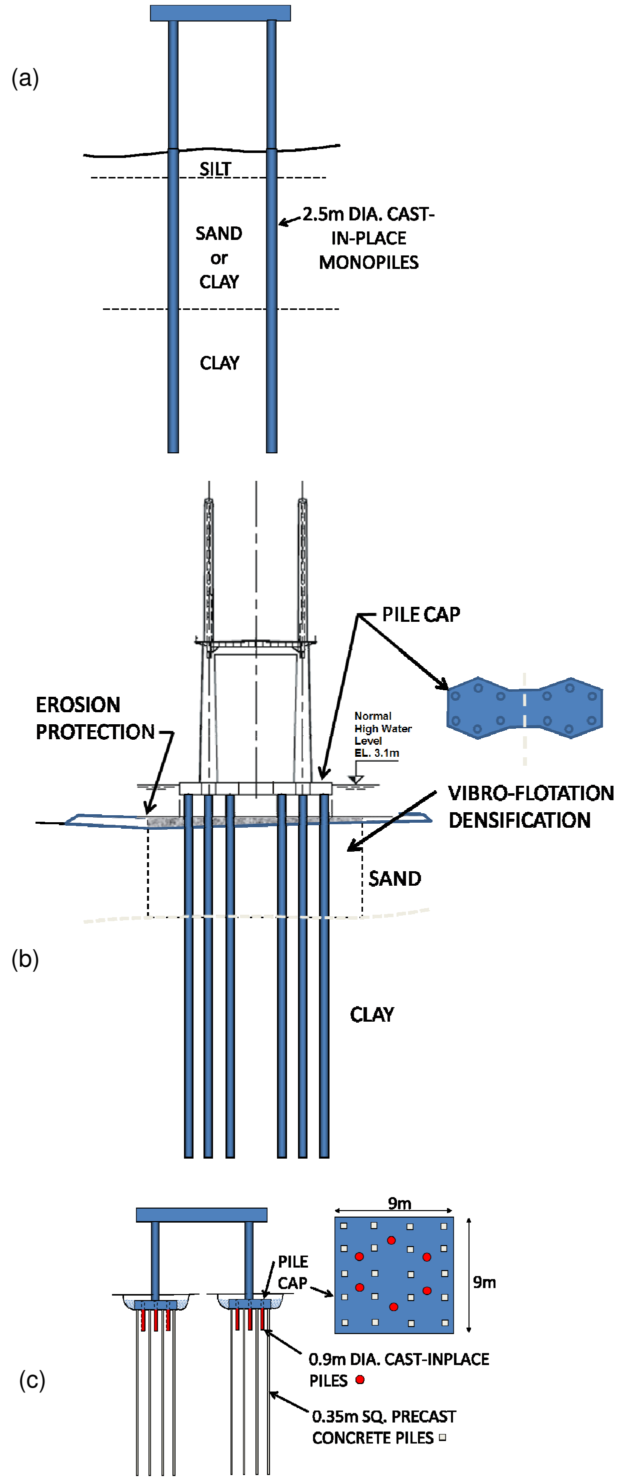

Figure 9 Typical pier bents for South Approach (a) with 30 to 75 m deep 2.5 m diameter monopiles, Main Spans (b) with 75 to 85 m deep 2.5m diameter piles, and North Approach (c) with 5 to 6m deep 0.9 m diameter shear piles and disconnected 12 to 36 m deep 0.35 m sq. precast concrete piles.

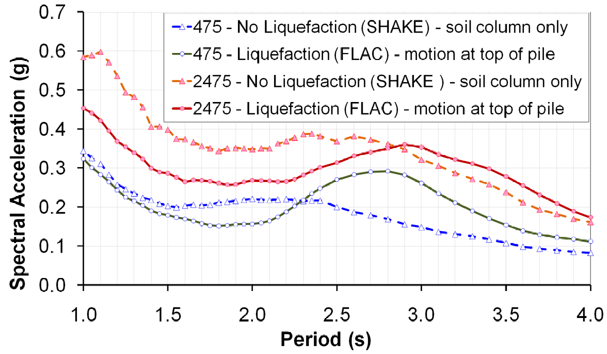

Figure 10 Comparison of near-surface spectrum (for the periods of primary interest) from equivalent-linear SHAKE91 analyses with assumption of no liquefaction to that calculated at top of the monopiles (with liquefaction allowed to occur around the piles) using FLAC dynamic soil-structure analyses.

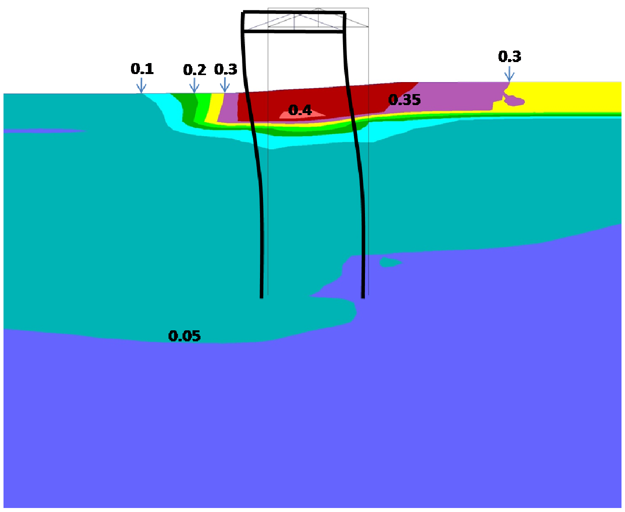

Figure 11 FLAC model of South Approach bent showing horizontal ground displacement contours in meters and exaggerated displaced shape of structure.

except at pier M1 next to the south river bank. Vibro-replacement ground densification with seismic drains adjacent to the 1.2 m water main was used around pier M1. Resistance of soil susceptible to liquefaction was ignored when assessing axial capacity of the piles. Fig. 11 shows typical numerical analyses results for the case with sloping ground and no densification. Moments and shears within the monopiles were within elastic limits.

6.3 Main River Piers

The main river piers (Fig. 9b) consist of a pile cap located above river grade supported on twelve 2.5 m diameter excavated and cast-in-place reinforced concrete piles of 75 to 85 m length.

The 475-year return period loading generally controlled pile lengths. Two dimensional dynamic ground response analyses with simplified structural elements in the model were carried out to assess densification performance and moments and shears induced in the piles by earthquake induced (kinematic) ground displacements. Ground densification by vibro-flotation was carried out around all the river piers in order to stiffen the response of the foundations. Fig. 12 shows a typical longitudinal numerical model profile including much of the south approach, the river channel and banks. Fig. 13 shows a mid-river pier detail from the same model, and Fig. 14 shows typical stress-strain response of loose sand within the model.

6.4 North Approach

The north approach is founded on a unique foundation where the lateral loading from the earthquake is resisted by the pile cap and short 0.9 m diameter bored cast-in-place reinforced concrete piles of 5 to 6 m length and the axial loads are resisted by slender 0.35 m square pre-cast concrete piles of 12 to 36 m length (Fig. 9c). The short shear piles are fixed to the cap, whereas the pre-cast piles are not connected to the pile cap. A 50 to 100 mm thick layer of gravel disconnect was placed between the piles and the caps. The purpose of the disconnect was to reduce moments, shears and tensile loads transferred to the relatively inexpensive and slender pre-

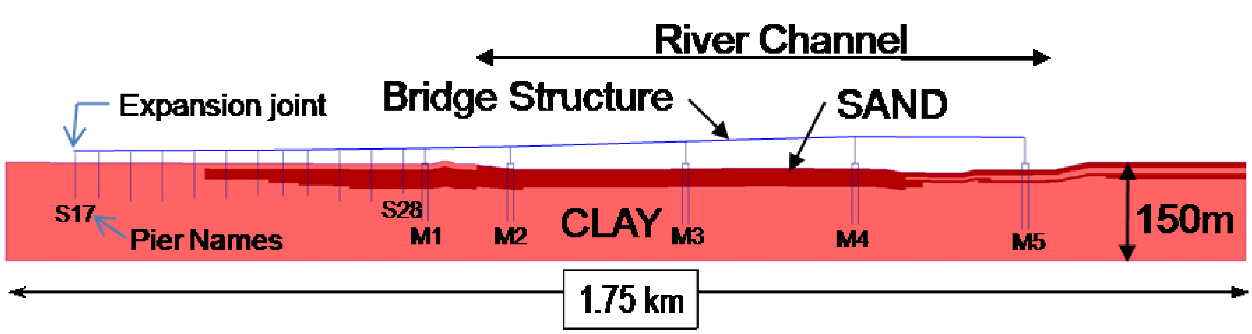

Figure 12 Longitudinal numerical model profile showing sand and clay zones and structure included in model. Model mesh has over 10,000 elements

cast piles. The length of the precast piles varied: 12 m to 24 m long where sand layers exist, and 36 m long friction piles in the deep clay-silt soils.

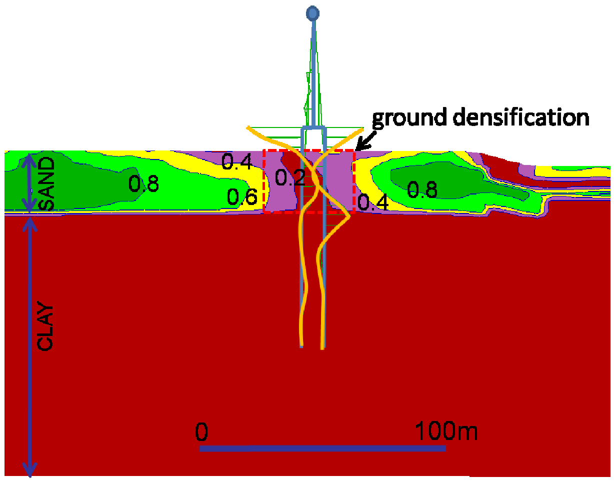

Figure 13 Results from numerical model shown in Fig. 12 showing pore pressure ratio (R_u) contours and moment distribution in pile model

The 475-year event loading generally controlled the number of piles required. Vibro-replacement ground densification and seismic drains were used to mitigate liquefaction at piers where loose sand layers were present.

7 RIVER EDGE AND EMBANKMENT STABILITY

River bank stability and deformation analyses were performed using (1) limit equilibrium slope stability analysis and Newmark method, and (2) dynamic two dimensional numerical analyses with the program FLAC and UBCHYST and UBCSAND constitutive models. Cyclic simple shear tests with and without static bias were carried out to calibrate the UBCHYST model (Fig. 15). Ground displacements of approximately 0.5 m were calculated for the 2475-year return period earthquake. Impact of these displacements on the foundations were assessed by (1) analyzing the pile foundations using a P-Y procedure with the program LPILE and free-field "kinematic" soil displacements, and (2) including simple structure and pile elements in some of the FLAC analyses. These analyses indicated that the pile foundation elements generally remained elastic for all design earthquake loading conditions.

8 CONCLUSIONS

The challenging soil conditions combined with stringent performance-based seismic design criteria led to the use of multiple foundation types and complex soil-structure interaction analyses for design optimization. Foundation types included groups of twelve 2.5 m diameter reinforced concrete shafts for the main river piers, and 2.3 to 2.5 m

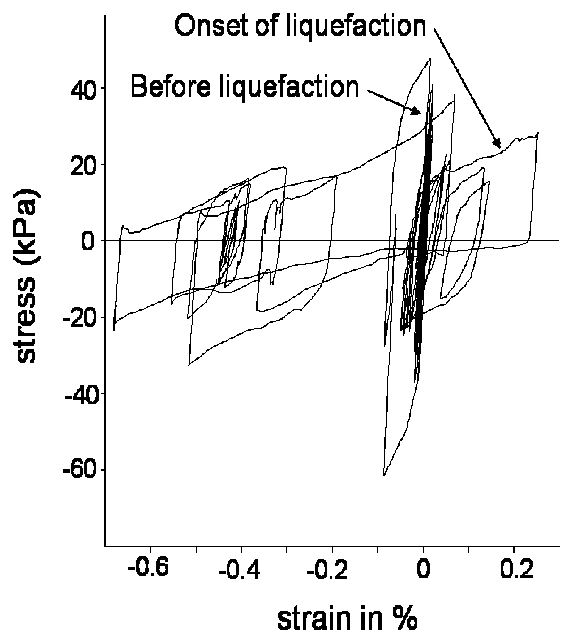

Figure 14 Typical stress vs strain plot of liquefiable sand from UBCSAND model

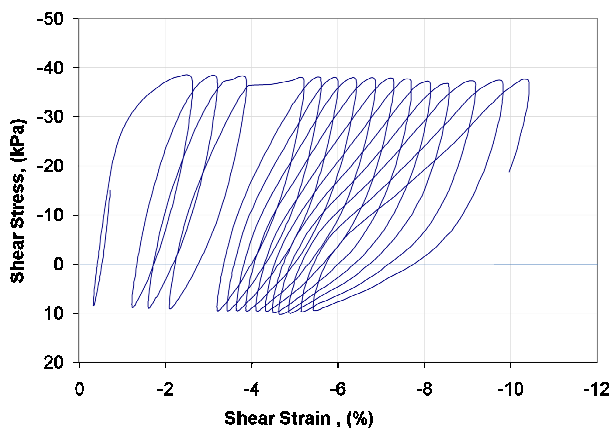

Figure 15 Stress-strain results from cyclic simple shear test on silty clay soil from 12.5 m depth below north river bank (test by Dept. of Civil Engineering, University of British Columbia)

diameter monopiles for the south approach structures.

For the north approach an innovative foundation system consisting of fixed, short, cast-in-place shafts and disconnected slender precast concrete piles were used for lateral and axial resistance, respectively. Numerical push-over and dynamic analyses showed that disconnecting the slender precast piles from the pile cap reduced the earthquake induced moments and shears in the piles.

Seismic design generally governed pile reinforcement and size requirements for the large shafts. Site-specific design response spectra, assessment of earthquake-induced ground movements, and assessment of demands on piles due to both inertial loading from the structure and kinematic loading due to relative displacements within the soil were obtained. Assessment of soil liquefaction, its consequences and mitigation were carried out. Analytical procedures varied from use of simple empirical procedures to more advanced effective-stress soil-structure interaction dynamic analyses using the computer program FLAC with constitutive models UBCHYST and UBCSAND. Soil-structure interaction analyses, with and without soil liquefaction were carried out and confirmed ability of the bridge to meet the specified performance criteria.

ACKNOWLEDGEMENTS

The writers would like to acknowledge the contribution of Hisham Nofal of CH2MHill; Trevor Lumb of Trow Associates Inc.; Keith Robinson of EBA Engineering Consultants Ltd.; and Steve Zhu and Rodger Welch of Buckland & Taylor Ltd. We would also like to thank the Golden Crossing Joint Venture and Translink for the opportunity to work on this project and publish this paper, thank Daniel St Gelais of Trow Associates Inc., Teknika HBA, and Louis Crepeau of Osmos Canada for abstract translation, and Brian Hall of EBA Engineering Consultants for helpful review.

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