Geotechnical Seismic Retrofit of Mission Bridge, British Columbia, Canada

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ABSTRACT
The Mission Bridge, built in 1973, spans the Fraser River in British Columbia and is a critical link in the Province's disaster recovery network. The four-lane bridge is 1,050 m long and is supported by a series of concrete bents or piers founded on piles. The approach piers are supported on short timber piles and the river and riverbank piers are supported on long steel H-piles. The bridge site is underlain by potentially liquefiable Fraser River sand and liquefaction is the key issue affecting the seismic performance of the bridge. Seismic soil-structure and deformation analyses were conducted to evaluate and design seismic retrofit measures to minimize the effects of liquefaction induced displacements. The retrofit measures included ground remediation schemes at the two abutments, the two river banks piers, and the approach piers. Ground remediation used various techniques, including vibro- replacement, compaction piles, compaction grouting, seismic drains, and toe berms. This paper describes the seismic retrofit strategy, geotechnical retrofit construction and some key challenges faced during the design and construction.

RÉSUMÉ
Le pont Mission, construit en 1973, enjambe le fleuve Fraser en Colombie-Britannique et constitue un lien essentiel dans le réseau de rétablissement après sinistre de la province. Le pont à quatre voies a une longueur de 1 050 m et est soutenu par une série de courbures ou de piliers en béton fondés sur des pieux. Les piliers d'approche sont supportés sur de courtes piles de bois et les piliers des rivières et des berges sont supportés sur de longues pieux en H en acier. Le site du pont repose sur du sable potentiellement liquéfiable du Fraser et la liquéfaction est le principal problème qui affecte la performance sismique du pont. Des analyses sismiques de la structure du sol et de la déformation ont été effectuées pour évaluer et concevoir des mesures de réaménagement sismique afin de minimiser les effets des déplacements induits par la liquéfaction. Les mesures de rénovation comprenaient des plans d'assainissement du sol dans les deux culées, les piles des deux rives et les piles d'approche. L'assainissement au sol utilisait diverses techniques, y compris le remplacement par vibration, les pieux de compactage, le coulis de compactage, les drains sismiques et les bermes d'orteils. Cet article décrit la stratégie de rénovation sismique, la construction de réaménagement géotechnique et certains défis clés rencontrés lors de la conception et de la construction.

1 INTRODUCTION

The Mission Bridge was constructed between 1968 and 1973, and spans the Fraser River in British Columbia to connect the District of Mission on the north side to the City of Abbotsford near Matsqui on the south side of the river. Mission is approximately 80 km east of Vancouver. The bridge comprises four traffic lanes and is approximately 1050 m long. The bridge superstructure consists of the following three parts (see Figure 1):

- simply supported concrete I girder approach spans with concrete deck (N5 to N11 and S5 to S10);
- steel I girder river approach spans with concrete deck (N2 to N5 and S2 to S5); and
- stiffened steel box girder main river spans with an orthotropic steel deck (N2 to N1 to S1 to S2).

The bridge uses reinforced concrete piers with an "inverted A" or "V frame" configuration. Most of the bents have two columns. Three bents at the north end (N8, N9 and N10) have three columns to accommodate the widened deck, which carries an additional lane of off ramp traffic. The piers resist lateral loads by shear and flexure in frame action.

The bridge is founded on two main types of foundation systems. The approach piers are founded on cast-in-place reinforced concrete pile caps on treated timber piles. The river spans are supported by cast in place, tall concrete foundations through the water and upper soils on long steel H-piles driven into dense soils.

The north and south abutments are about 11.2 m and 13.3 m high, respectively. The north abutment is wider, 25.3 m compared to 21 m for the south abutment. Both abutments are spread footings measuring 3.6 m wide placed on compacted sand and gravel fill. The abutment fill slopes are 1.5H:1V along the sides of the approach and 1.75H:1V under the deck. The concrete girders rest on neoprene bearing pads, which permit longitudinal movements. Lateral movement of each span end is prevented by two small shear keys.

2 SUBSOIL AND FOUNDATION CONDITIONS

The geological profile at the bridge site shown in Figure 1 consists of the following from top to bottom or from the most recent to oldest deposits: Unit 1 - Surficial Fraser River sediments consisting of clayey sandy silt with a thickness ranging from 3 m to 6 m and this unit is absent in the current river channel; Unit 2 - Post-glacial Fraser River sediments consisting of fine sand with trace to some silt. The upper 6 m to 10 m of this sand stratum is generally loose to medium dense and the density increases to medium dense to dense, except in the southern part of the river channel; Unit 3 - Post-glacial layered fine-grained material consisting of stiff to very stiff silt and clay, with occasional sand layers, deposited in a lacustrine environment; Unit 4 - Hard Glaciomarine deposit consisting of layered silt, clay and gravel with occasional cobbles and boulders; Unit 5 - Pre-glacial deposit consisting of very dense sand and gravel

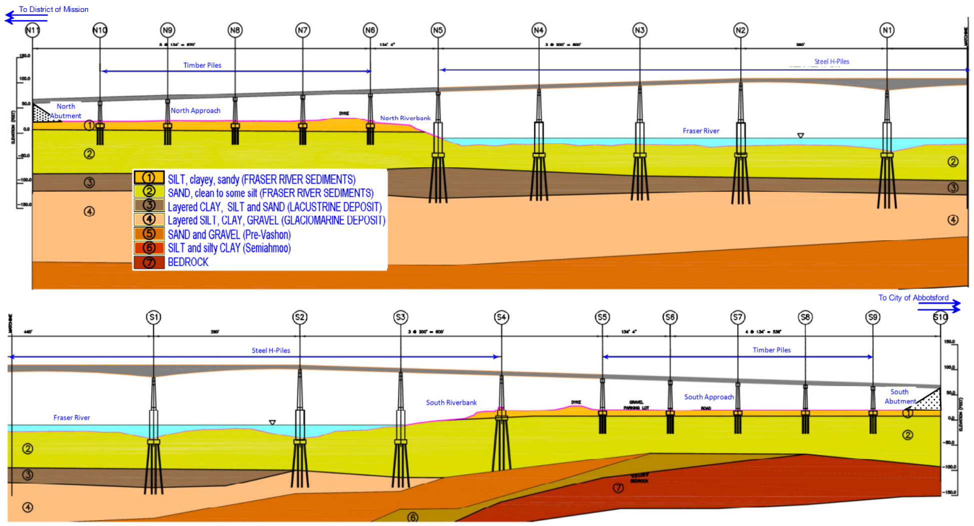

Figure 1: Geological cross-section of the Mission Bridge

from the Pre-Vashon period; Unit 6 - Very old pre-glacial deposit consisting of silt and silty clay; and Unit 7 - Bedrock.

Long-term foundation settlement and scouring of the river were major design issues for the original Mission Bridge foundations. Accordingly, the north and the south approach piers are founded on large timber pile groups. The timber piles transfer the foundation loads through the compressible surficial silt layer (Unit 1) into the underlying sand (Unit 2). Driving of the large number of timber piles at a close spacing of 0.3 m appeared to have compacted the sand and further reduced post-construction foundation settlement. The north approach piers N6 to N10 are supported on pile groups with 78 to 96 timber piles and their length generally varies between 9.2 m to 11 m. South approach piers S6 to S10 are supported on a group of 78 timber piles and their length generally varies between 10 m and 10.9 m.

The river piers are founded on deep HP360x109 (HP14x73) steel H-piles driven primarily into the hard glaciomarine deposit (Unit 4). The number of piles varies between 56 to 96 and their length varies between 21 m to 44 m. It was intended that the steel piles would have adequate load carrying capacity even if the sand were scoured away. The design scour depth was taken as EL – 15.2 m to allow a safety margin. The north and south abutments are typical bank-seat abutments consisting of cast-in-place concrete abutments on spread footings supported on granular approach fill embankments

3 SEISMIC DESIGN CRITERIA

A 'safety' level performance criteria was adopted for the retrofit design, which required collapse prevention under a 475-year return period earthquake. A magnitude M7 event with a 'firm' ground peak ground acceleration (PGA) of 0.245 g was taken to represent the design earthquake. The 'safety' performance criteria require the structure to perform as follows: (1) guard against collapse of any part of the bridge; (2) sustain damage levels which may allow access to the bridge in the days following a major earthquake for emergency vehicles; (3) allow bridge closure for long term post earthquake repairs. Access to emergency vehicles or repairable damage to the bridge following the design earthquake was considered desirable but not required for the adopted safety level retrofit.

4 SEISMIC DEFICIENCY ASSESSMENTS

Amplification of ground motions, liquefaction of foundation soils and liquefaction induced displacements were the key issues affecting the performance of the bridge structure and in the retrofit design of the bridge. Seismic performance of the original bridge structure under the design earthquake was assessed by carrying out the following analyses:

- Site or ground response analyses;
- Liquefaction assessment using insitu CPT and SPT data;
- Limit equilibrium type post-earthquake stability analyses of abutments and riverbank slopes using residual shear strengths for liquefied soils;
- Assessment of lateral spread due to slopes of the river bed and assessment of their impact on the river piers supported on H-pile groups;
- Seismic deformation analyses using FLAC (Itasca, 2000) to assess the liquefaction induced displacements and settlements of abutments, riverbank slopes and riverbed slopes; and
- Seismic soil-structure interaction analyses using FLAC of H-pile supported river and riverbank piers, timber pile group supported approach piers, and abutment spread footings supported on embankment fills. In these analyses, the pile foundations were modeled using pile elements and nonlinear soil-pile interaction springs.

Liquefaction assessment was conducted in general accordance with the Seed's approach as recommended in the Youd et al. (2001). It showed that liquefaction will occur in the loose to medium dense zones of the sand unit throughout the bridge alignment during the design earthquake. In general, the extent of liquefaction increased from the north end of the bridge towards the south end. The results of the liquefaction analyses with reference to the bridge foundations showed:

- At the north approach, liquefaction is limited to looser zones in the upper 5 m to 8 m portion of the sand stratum below the upper silt layer. Liquefaction will occur around the timber pile groups, but not inside and little or no liquefaction is expected below the timber pile groups;
- At the river channel, liquefaction extends from the mud line down to about 10 m to 20 m depth in the sand stratum. Extensive liquefaction is expected around the buried bottom portion of the pier columns, the pile caps, and the upper portion of the pile groups through the Unit 2 sand. The existing H piles extend through Unit 3 and into Unit 4; both of which are not liquefiable;
- At the south approach, extensive liquefaction occurs in the upper 10 m to 20 m of the sand stratum below the surficial silt layer. Liquefaction is expected to occur around the timber pile groups and to about 2 m depth below the pile toes. Liquefaction will not occur within the pile group due to densification effects from installation of the closely spaced timber piles. Potential densification of the sand below the pile toes during installation may limit the anticipated liquefaction zone to less than 2 m thick.
- Liquefaction will affect the stability of the abutment and river bank slopes, and will result in vertical and horizontal permanent ground deformations affecting the pile-supported bridge piers.

Limit equilibrium post-earthquake stability analyses of the north and south abutments showed minimum factors of safety of 1.2 and 0.70, respectively, suggesting that significant displacements will occur at the north abutment and very large or flow slide type of failure will occur at the south abutment. Deformation analyses using FLAC confirmed the post-earthquake stability analyses results. Retrofit measures were required at both abutments and either compacted toe berm or combination of berm and ground densification were considered as viable measures at the abutments.

Post-earthquake stability analyses of the north and south riverbank slopes showed minimum factor of safety of unity for both riverbank slopes indicating that retrofit measures are required for piers located at both banks. The depth of liquefied soils was however greater at the south riverbank which will require treatment to depths exceeding 30 m, if ground densification is used to prevent liquefaction or mitigate its effect. Seismic deformation and seismic soil-pile-structure interaction analyses of south riverbank with Piers S3 and S4 confirmed large displacements will occur at the south riverbank. Retrofit measures were required at both riverbanks and ground densification was considered as an effective measure at both banks. However, feasibility of densifying deep soils at the south riverbank was a concern.

At the south approach piers S5 to S9, liquefaction-induced settlements of up to about 200 mm were predicted with differential settlements of 100 mm to 200 mm. At the north approach piers N6 to N10, the estimated settlements were comparatively less, due to better soil conditions than those at the south approach piers, with total settlement of up to 100 mm and differential of 50 mm to 100 mm. Existing north approach piers can accommodate the predicted settlements. However, retrofit measures were required for south approach piers to reduce settlements, which would occur due to liquefaction of relatively thin layer beneath the timber pile toes. Compaction grouting was considered as a viable technique to densify soils below the pile toe.

Seismic deformation and seismic soil-structure interaction analyses of the riverbed slope and the river piers predicted very small pile cap displacements, which can be accommodated by the original piers. However, horizontal pile cap displacements in the range of 130 mm to 200 mm at pier S1 and 270 mm to 390 mm at pier S2 were predicted. Effectiveness of retrofit measure in the form of annulus ground densification around the pier foundation was assessed. Because of the presence of batter piles, the densification would be placed approximately 4.4 m from the piles, leaving a significant gap of undensified soil. Liquefaction would occur in soils within the gap, making the densification not very effective in reducing the pile cap displacements. Therefore, it was decided to retrofit the river pier columns to increase their ductility to accommodate the predicted displacements. This structural retrofit would also be more cost effective than ground improvement in the river.

5 CONCRETE AND TIMBER COMPACTION TEST PILE PROGRAM

Timber and concrete compactions piles were selected as potentially viable methods to densify shallow and deep potentially liquefiable soils at the north and south riverbank and at the south abutment. During detailed design, a compaction test pile program was conducted at the South Riverbank Pier S4 to evaluate pile driveability and effectiveness in densifying the loose soils. Pre- and post densification Cone Penetration Tests (CPT) were used to assess the densification. Figure 2 shows the concrete and timber test pile areas at Pier S4, and Figure 3 shows pictures taken during the test pile program.

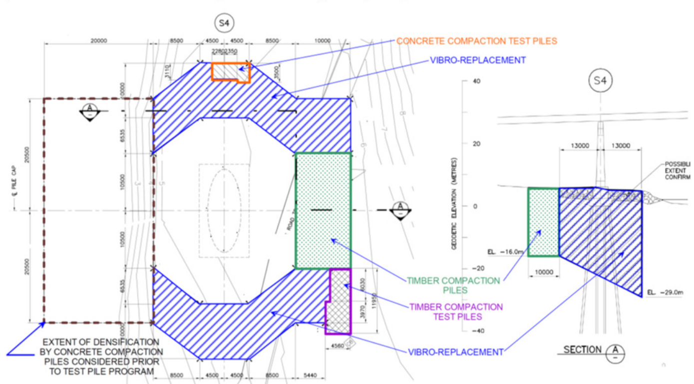

Figure 2: Vibro-replacement and timber compaction piling densification zones at Pier S4, including test pile areas

Figure 3: Concrete and timber compaction test piling at the south riverbank

5.1 Concrete Compaction Piles

ICP Piles (High Performance Pre-tensioned Spun Concrete piles), with outer diameter of 350 mm and pile wall thickness of 70 mm, were installed to the target depth of approximately 35 m in three segments, each 12 m long. The piles were supplied with pre-fitted annulus steel end-plates which facilitated splicing by welding. A full end plate was welded to the bottom of the first length of the pile prior to driving. i.e. piles were driven closed-ended. The first section was driven using a 2445 kg drop hammer with a drop height between 1.5 m and 1.8 m. The second and third lengths were driven using the APE D46-42 single acting diesel hammer at the first or second fuel setting corresponding to rated energies 76 kJ and 103 KJ, respectively. All the piles were driven relatively easy and no pile damage was observed during driving. The pile spacing was varied between 1.22 m and 2.4 m. The monitored data showed no ground heave during pile driving.

Figure 4 shows comparison of comparison of pre- and post-densification cone tip resistances and the required minimum tip resistances corresponding to soils with less than 10% fines and 10% to 20% fines from test areas where piles were installed at 1.67 m and 1.22 m spacings. Significant reduction in tip resistances was evident at 1.67 m spacing or greater. When the spacing was reduced to 1.22 m, no reduction in resistance was observed with some slight increase in resistance. During piling, the surface settlement was monitored at several locations in and round the test area and no noticeable heave was observed. The concrete test pile program showed that, although the piles could be driven to 36 m depth with relative ease and without any damage at close spacing, they would not provide the required densification unless the spacing is reduced to less than 1.22 m. Driving long concrete piles at smaller than 1.22 m will not be economical and may not be practical.

5.2 Timber Compaction Piles

Timber compaction test piles were driven at centre-to-centre spacing 1.52 m and 1.22 m in an equilateral triangular pattern. Piles were driven using a 2445 kg drop hammer and APE D19-42 single acting diesel hammer with a maximum rated energy of 64 kJ. The target depth of the piles was 22 m. The piles were untreated, peeled round wood piles (Douglas Fir) with pile toe diameter in the general range between 255 mm and 330 mm. Selected piles were driven in two lengths with a splice in the form of a cylindrical steel casing with a center plate. Figure 5 shows the comparison of pre- and post-densification CPTs, suggesting that the piles were effective in densifying the soils and piles at 1.22 m spacing generally provide the required densification.

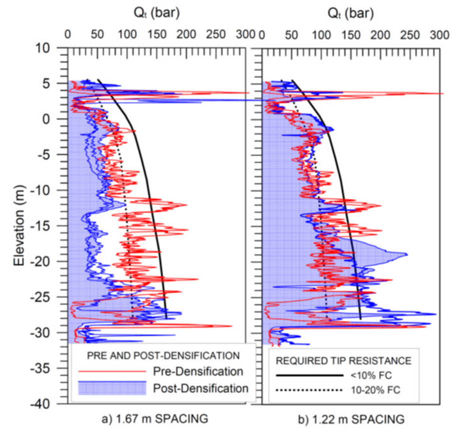

Figure 4: Effectivenss of concrete compaction test piles

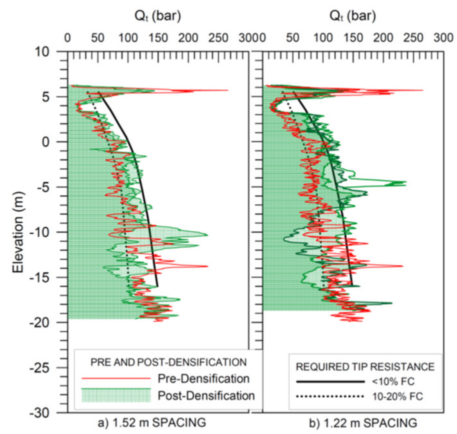

Figure 5: Effectivenss of timber compaction test piles

Following the concrete and timber test pile program, timber compaction piles were proposed for densification of soils at the north riverbank both under and outside the bridge deck, at the south side of the south riverbank under the bridge deck and at the south abutment under the bridge deck. However, use of concrete compaction piles was rejected.

6 SEISMIC RETROFIT STRATEGY

The following retrofit measures were proposed for the abutments and for the river, approach and riverbank piers.

- North Abutment (N11): A horseshoe-shaped toe berm as shown in Figure 6, comprising compacted sand and gravel placed against the existing abutment slopes. The toe berm is 10 m wide and approximately 4 m high;
- North Approach Piers (N5 to N10): A ring of seismic drains around the perimeter of the existing timber pile group. Seismic drains consisting of fine gravel columns 230 mm in diameter installed through the Unit 1 silt and into the Unit 2 sand at 3 m centres to about 3 m below the timber pile toes were proposed;

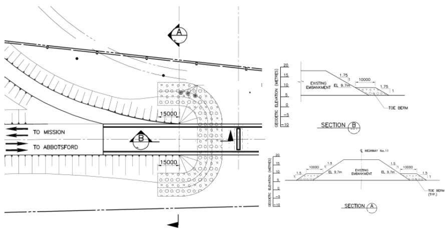

Figure 6: Toe berm at the north abutment

- North River Bank (N5-N6): As shown in Figure 7, a 10 m wide, about 15 m deep and 42 m long densified zone, or a “seismic dyke”, located between Piers N5 and N6. Due to limited head room, timber compaction piles were proposed to densify soils both under and outside the bridge deck;

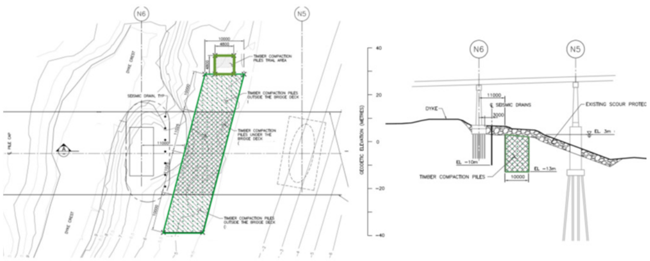

Figure 7: Timber compaction piles at the north riverbank

- River Piers (N4-S3): Structural retrofit of concrete columns to accommodate potential lateral displacements of the piers of the order of 0.3 m, and preferably up to 0.4 to 0.5 m was proposed;
- South River Bank (S3-S4): At the south riverbank, the required depth of ground improvement varies from about 35 m on the river or north side of the pier to 22 m on the south side. Concrete compaction piles were originally proposed for the deep areas as a potential technique to densify deep soils. However, the test pile program showed that the concrete piles would not be a viable or economical technique and was rejected. Following the test pile program, a combined structural retrofit of the pier columns at S4 plus ground densifications were proposed (See Figure 2).

  The strategy was to minimize the ground displacements as much as practical with ground improvements where it is feasible and economical and to enhance the deformation capacity of the pier column by retrofitting it structurally. Ground densifications were proposed on the three sides surrounding Pier S4, except on the river side. Timber compaction piles were chosen to densify soils under the bridge deck on the south side of Pier S4, and vibro-replacement was chosen to densify soils outside the deck on the east and west side of the deck.

  For the structural retrofit of the pier columns, an innovative column jacket made up of ultra-high performance fibre-reinforced concrete was proposed to enhance their ductility and increase their deformation capacity. Details of the design and construction of the structural retrofit of the pier columns is described in Kennedy et al. (2015).
- South Approach Piers (S5 through S9): As shown in Figure 8, a 2 m wide by 2 m deep annulus ground improvement zone beneath the edges of the existing timber pile group. Ground improvement by compaction grouting through inclined drill holes was proposed as viable technique. In addition, a ring of seismic drains around the perimeter, similar to those at north approach piers, was also proposed.

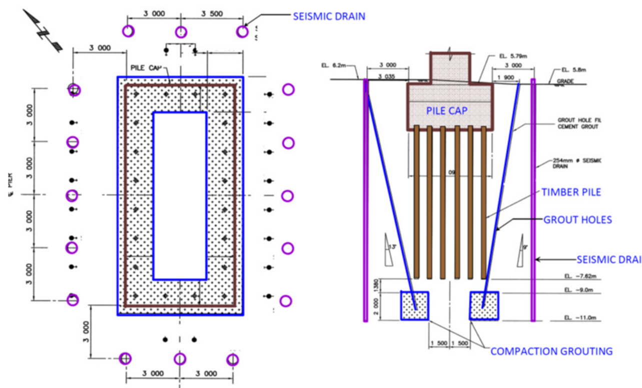

Figure 8: Compaction grouting and seismic drains at south approach Pier S5

- South Abutment: As shown in Figure 9, a horseshoe-shaped zone of ground improvement and a toe berm. Outside the bridge deck, the densified zone is 20 m wide by 17.5 m deep; under the deck, the densified zone is 15 m wide by 17.5 m deep. The toe berm is 10 m wide and 3 m high beneath the bridge deck and 5 m high outside the deck. Timber compaction piles and vibro-replacement were proposed to densify soils beneath and outside the bridge deck, respectively.

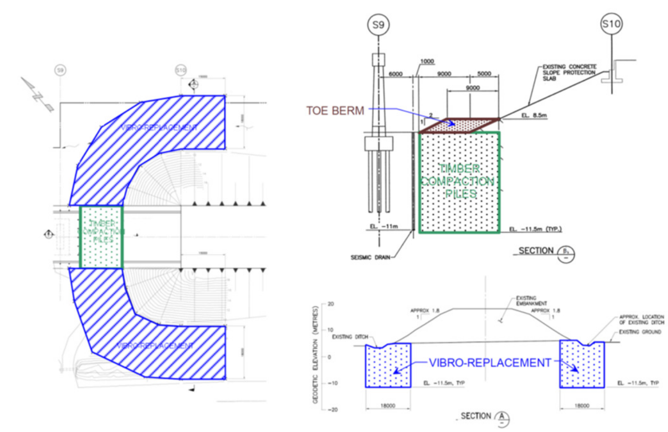

Figure 9: Vibro-replacement, timber compaction piles and toe berm at the south abutement, S10

The post-earthquake stability analyses, seismic deformation and seismic soil-structure interaction analyses were repeated with the proposed retrofit measures as verification and to optimize their extents.

7 RETROFIT CONSTRUCTION

7.1 Toe Berms at North and South Abutments

A horseshoe-shaped toe berm comprising compacted sand and gravel was constructed at both the north and south berms (see Figure 10).

Figure 10: Toe berms at the north and south abutments

7.2 Compaction Grouting at Approach Piers

At the south approach Piers S5 to S9, compaction grouting was performed to create a ring-shaped annulus densification zone beneath the original timber pile group foundation. Grouting was conducted by drilling grout holes at 9° to reach the loose zones beneath the pile toes, and grouting was conducted with target volume and limiting pressure established through trials (see Figure 11). Two trials were performed prior to production grouting and the performance was verified using CPTs. Details of grouting trials, performance verification and production grouting are described in Thavaraj and Sy (2017).

Figure 11: Compaction grouting at south approach piers-casing installation,inclination check and grouting

7.3 Vibro-Replacement at South Riverbank (S4) and South Abutment (S10)

Vibro-replacement using stone columns was carried out at S4 and S10 locations to densify the soils outside the bridge deck (see Figure 12). Trials were conducted both at S4 and at S10 prior to the production to establish pattern, spacing, means and methods for the stone columns and to verify performance.

The pier S4 was supported on 56 H-piles with depth to pile toe of 41m to 49 m. At S4, stone columns were installed using the top feed method in a triangular pattern at 2.75 m spacing. The width of densification was 10 m and the depth varied between 21 m and 31 m. Electrical vibrofloat equipment (175 HP, 1800 rpm and 300 amp) was used. Stone backfill was continuously applied from the top, at each 0.5 m depth, and the probe was vibrated until achieving 200 amps or for duration of 60 seconds, whichever occurred first. In the first unsuccessful trial lower target amperage and duration were used and they were 150 amps 30 seconds, respectively. Increase in the amperage and duration improved the densification. During production, in some deep areas, target densities were not achieved and remedial columns were also installed. A total of 169 stone columns were installed at S4.

At S10, stone columns were installed in 0.5 m increment at the same triangular pattern and at the same 2.75 m spacing to approximately 17 m depth. However, the probe was vibrated until achieving 140-150 amps or for a duration of 30 seconds, whichever occurred first. The target cone tip resistances at S10 is smaller than those at S40. In the unsuccessful first trial, same target amperage and duration were used but a greater spacing of 3 m was used. Reduction in spacing improved the densification. At S10, a total of 369 stone columns were installed. Remedial columns were not required at S10.

Figure 13 shows pre- and post-densification CPTs at S4, indicating that the chosen pattern spacing and procedure could achieve the required densification. The existing bridge structure was monitored during construction and no discernable movements were observed. The measured settlements were less than 10 mm at Piers S4, S5 and S9, and less than 15 mm at the south abutment.

Figure 12: Vibro-densification at the south riverbank and south abutment

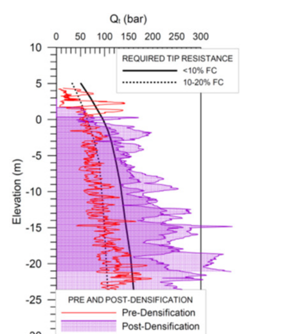

Figure 13: Effectiveness of vibro-replacement at S4

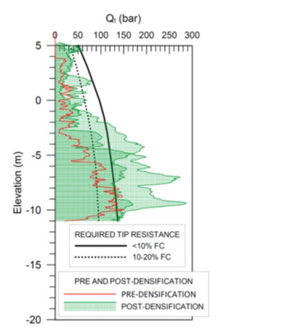

Figure 16: Effectivenes of timber compaction piles at S10

7.4 Timber Compaction Piles at North Riverbank (N6), and South Riverbank (S4) and South Abutment (S10)

Timber compaction piles were used to densify the soils at N6, S4 and S10. At all three locations, the timber piles were driven under the bridge deck where there is headroom constraint, and at N6, they were also driven outside the deck. At all three locations, trials were performed to verify the chosen pattern and spacing, and they were not changed during production. Pre- and post-densification CPTs were used to verify performance and assess achieved densification.

At S4, timber (Douglas Fir) piles with a minimum tip diameter of 225 mm were driven in three sections using two splices (See Figure 14). The approximate first length of the pile was 6 m, and the second and third sections were approximately 7.6 m. 900 mm long steel tube with a 150 mm wide centre plate were used as splices. The diameter of the splices used varied between 254 mm and 356 mm depending upon the pile sizes. The piles directly above and below the splice, and pile tops were banded to minimize or eliminate pile damage during driving. The piles were installed at 1.22 m spacing in an equilateral triangular pattern. The initial lengths were driven using a 2500 kg drop hammer and the subsequent lengths were driven using three single acting diesel hammers mostly between 45 kJ and 63 kJ energies. A total of 179 piles were installed at S4 which included only 9 remedial piles. The required densification was generally achieved with the chosen pattern, spacing and installation procedure. Notably, the use of two splices to drive piles in three segments under the deck to depth of as deep as 21 m was successful.

Figure 14: Compaction pile driving at south riverbank

At S10, timber (Douglas Fir) piles with a minimum tip diameter of 225 mm were driven in three sections using two splices (see Figure 15). The approximate length of each section was 6 m. Same splices as in S4 were used. The piles were installed at slightly greater 1.25 m spacing in an equilateral triangular pattern. Initial driving was conducted using a 2500 kg drop hammer, and near the end of driving most of the piles were driven with APE 7.5a low-headroom hydraulic hammer at the maximum rated energy of 34 kJ. A total of 219 piles were installed and only 4 remedial piles were required at S10. At S10 also, driving piles in three sections using two splices to depth as deep as was 17 m was successful. Figure 16 shows comparison of CPTs for piles at S10.

At N6, timber (Douglas fir) piles with a minimum tip diameter of 225 m were driven at 1.22 m spacing in an equilateral triangular pattern to a depth of approximately 15 m (see Figure 17). The piles outside the deck were driven in single section, and the piles under the deck were driven in two sections with a splice. The length of each section driven under the deck was approximately 7.5 m. 914 mm long by 279 mm outer diameter steel tubes with a center plate were used as splice. All piles were driven using an APE D19-42 diesel hammer with maximum rated energy of 64 kJ. No drop hammer was used during pile driving even for initial driving. Driving the entire length of pile in two sections with a splice using a diesel hammer under the deck was successful. A total of 362 piles were installed, which included 12 remedial piles.

Figure 15: Compaction pile driving at south abutment using low headroom hydraulic hammer

Figure 17: Compaction pile driving at north riverbank using an open-ended diesel hammer and splicing of pile

The existing bridge structure and the adjacent dykes were monitored during pile driving at all three locations during construction and no discernable movements were observed. The observed settlements were less than 10 mm at S4 and S10, and less than 5 mm at N6. The post-densification CPTs showed that the specified tip resistances were generally met and the design intent of the compaction piling was achieved.

7.5 Seismic Drains at North and South Approach Piers

Seismic drains with 255 mm diameter were installed at the approach piers in a ring shape, approximately 3 m from the edge of the pile cap, at a spacing of 3 m and to approximately 3 m below the toe of original timber piles. The drains had a 38 mm slotted Schedule 80 PVC at the center and the annulus space between the PVC and the drill hole was filled with pea gravel. The top of 1 m of the drain was widened to approximately 1 m diameter and filled with sand and gravel. All the drains, except those at N6, were installed using a hollow stem auger. At N6, a sonic rig was used for the installation of seismic drains (see Figure 18).

8 SUMMARY AND CONCLUSIONS

The Mission Bridge, built in 1973, was upgraded to withstand the design earthquake. The existing approach piers are supported on relatively short treated timber piles, while the existing river piers are supported on long steel H-piles. The bridge site is underlain by deep post-glacial river sediments consisting of fine sand with trace to some silt, the upper part of which is loose and was found to be liquefiable under the design earthquake. Liquefaction-induced displacements were the key issue affecting the seismic performance of the original bridge. Site response, liquefaction assessment, seismic stability, seismic soil-structure and deformation analyses were conducted to evaluate the seismic performance and design seismic retrofit measures to minimize the effects of potential large liquefaction-induced ground displacements due to the design earthquake.

Figure 18: Seismic drain installation using auger and sonic rig

Figure 19: Retrofitted pier S4 columns with ultra-high performance fibre-reinforced concrete jackets (Kennedy et al., 2015)

The retrofit strategy included both structural and geotechnical retrofit measures. Ground improvements were conducted where feasible and economical to either minimize or mitigate the effect of ground displacements on the structures. At the river piers where ground improvement was not feasible or economical, and at south river bank pier S4 where fully mitigating the effect of displacement by ground densification was not feasible, structural retrofit measures were adopted. At S4, a column jacket was added to improve its ductility and deformation capacity (see Figure 19).

Geotechnical retrofit measures included: (1) compaction grouting to densify loose soils beneath the pile toes at the south approach piers; (2) vibro-replacement to densify loose soils outside the bridge deck at the south riverbank and south abutment; (3) timber compaction piles to densify loose soils at the north and south riverbanks and south abutment; (4) seismic drains at north and south approach piers to enhance drainage; and (5) compacted toe berms at north and south abutments.

Vibro-replacement was conducted using the top feed method to depths varying between 17 m and 31 m. Stone columns were installed at 2.75 m spacing in an equilateral triangular pattern using an electric vibroflot.

Timber compaction piles were driven in an equilateral triangular pattern at 1.22 m and 1.25 m spacings. The piles under the bridge deck at the north and south riverbanks and south abutment, where there is head room restriction, were driven successfully in segments with splices, to depths varying between 15 m and 21 m. At the south riverbank and south abutment, two splices were used and one splice was used at north riverbank. 900mm to 915 mm steel tubes with center plate were used as splices. The piles at the riverbanks were installed using drop hammer and single acting diesel hammers, and the piles at the south abutment were installed using a low-headroom hydraulic hammer.

Pre- and post-densification CPTs were conducted to verify the that the intended densifications by compaction grouting, vibro-replacement and timber compaction piles were achieved. Each of these techniques was started with a trial to establish pattern and spacing for compaction points and procedure for densification.

Prior to implementation of retrofit measures at the riverbanks and abutments, a compaction test pile program consisting of both concrete and timber compaction piles to densify relatively deep and shallow soils, respectively, was conducted. This program confirmed that the timber compaction piles would be feasible, however, the concrete compaction piles would not be feasible and were uneconomical. Following this program, the use of concrete compaction piles was not implemented.

Seismic drains were installed using hollow stem auger and sonic rigs. The integrity of adjacent foundations and structures, underground utilities and the environmental impact on the Fraser River and its habitat were key concerns during ground densifications. They were monitored and were not adversely impacted by the ground improvement construction. The seismic upgrade of the bridge was successfully completed in November 2015.

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