IMPACTS OF DEEP SOFT SOILS AND LIGHTWEIGHT FILL APPROACH EMBANKMENTS ON THE SEISMIC DESIGN OF THE HWY. 15 NORTH SERPENTINE RIVER BRIDGES, SURREY, B.C.

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ABSTRACT
The North Serpentine River Bridges will be constructed on deep soft soils and founded on frictional piles. Lightweight fills will be used to increase the stiffness and strength but reduce the weight of the abutment fills. Geotechnical seismic design issues included the development of ground-surface response spectrum for structural analyses, determination of bending moments and shear forces in the piles under the design earthquake, and most importantly assessment of the seismic stability of the abutments and approach embankments. This paper first describes the methodology and results of response analysis for piles of this bridge on soft soils subjected to earthquake loads using the conventional uncoupled method of analysis. The response of piles and the approach embankments are then computed from a dynamic soil-structure interaction analysis using the computer program VERSAT-D2D. Results of the soil-structure interaction analysis illustrate the effect of permanent ground displacements on pile bending moments and shear forces.

RÉSUMÉ
Les ponts de la rivière Serpentine Nord seront construits sur d'épais sols mous dont les assises seront appuyées sur des pieux en friction. Des matériaux de remblai légers seront utilisés et auront pour effet d'augmenter la rigidité et la force et de réduire le poids des culées. Les éléments de conception sismiques géotechniques considérés ont inclus le développement de spectre de réponse à la surface du sol pour les analyses structurelles, la détermination des moments de flexion et des forces de cisaillement dans les pieux pour un séisme de dimensionnement et, le plus important, l'évaluation de la stabilité sismique des culées et des talus. Cet article décrit premièrement la méthodologie et les résultats des spectres d'analyses pour les pieux de ce pont sur des sols mous sujets à des charges sismiques utilisant la méthode d'analyse conventionnelle La réponse des pieux et des talus ont alors été calculées suivant une analyse d'interaction dynamique sol-structure en utilisant le logiciel VERSAT-D2D. Les résultats d'analyse d'interaction sol-structure illustre l'effet des déplacements permanents du sol sur les moments de flexion et les forces de cisaillement dans les pieux.

1. INTRODUCTION

Piles are often used to support bridges on soft soils. Under static loads, the piles are designed to satisfy requirements on axial load capacity and axial pile settlement. Under earthquake loads, potential damage to the piles may be caused by horizontal forces acting on the pile heads from seismic inertia forces on the bridge, or lateral ground displacements of soils around the piles either in soft soils or in liquefied soils.

This paper first describes the methodology and results of response analysis for the piles using the conventional uncoupled method of analysis. The response of the piles and the approach embankments are then computed from a dynamic soil-structure interaction analysis. The stiffness and strength of the lightweight fill materials proved to be of critical importance to the structural design and performance of the piles and the approach embankments.

2. THE BRIDGE SITE AND SOIL CONDITIONS

As part of the Highway 15 Improvements under the Border Infrastructure Program in the Lower Mainland of British Columbia, the existing North Serpentine River Crossing (Figure 1) will be decommissioned and replaced with two new bridges, the northbound bridge to be built new and the southbound bridge to replace the existing bridge. The two bridges will be side-by-side and identical in design. Each bridge will have a single-span, 24-m-long, 14-m-wide, and will be a two-lane reinforced concrete structure. The two abutments of each bridge will be supported on four 914-mm diameter steel pipe piles with the approaches comprising both lightweight structural fill and conventional mineral fill.

The twin bridges will be founded on very thick soft soils. Figure 2 shows soil data obtained from a deep borehole (BH04-521) and two Cone Penetration Tests (CPT03-520 and SCPT03-521) conducted at the north and south riverbanks. The soil stratigraphy at the site consists of peat, organic silts to inorganic clayey silts at shallow depths (Salish bog and lacustrine lake deposits to Fraser River deposits) overlying Capilano marine silty clay to clay loam to a depth up to 65 m below the ground surface. Below that depth, a glacial deposit of dense to very dense sand and gravel, inferred to be a glaciofluvial deposit within the Vashon Drift, was encountered. The stiff silty clay layer at approximately El. -9.0 was inferred to be the desiccated crust of the underlying Capilano marine sediments. Measured shear wave velocities were about 50 m/s in the peat and organic silts, 76 m/s in the clayey silts, 132 m/s in the stiff crust, 88 m/s at El. -10.5 m, increasing to130 m/s at El. -31 m and 170 m/s at El. -33.5 m. The marine sediments have an average water content of 66% and liquidity index of 1.0, and are inferred to be close to normally consolidated with an average compression index of 0.62. The very dense till-like sand and gravel has a SPT blow count of 100 over 130 mm.

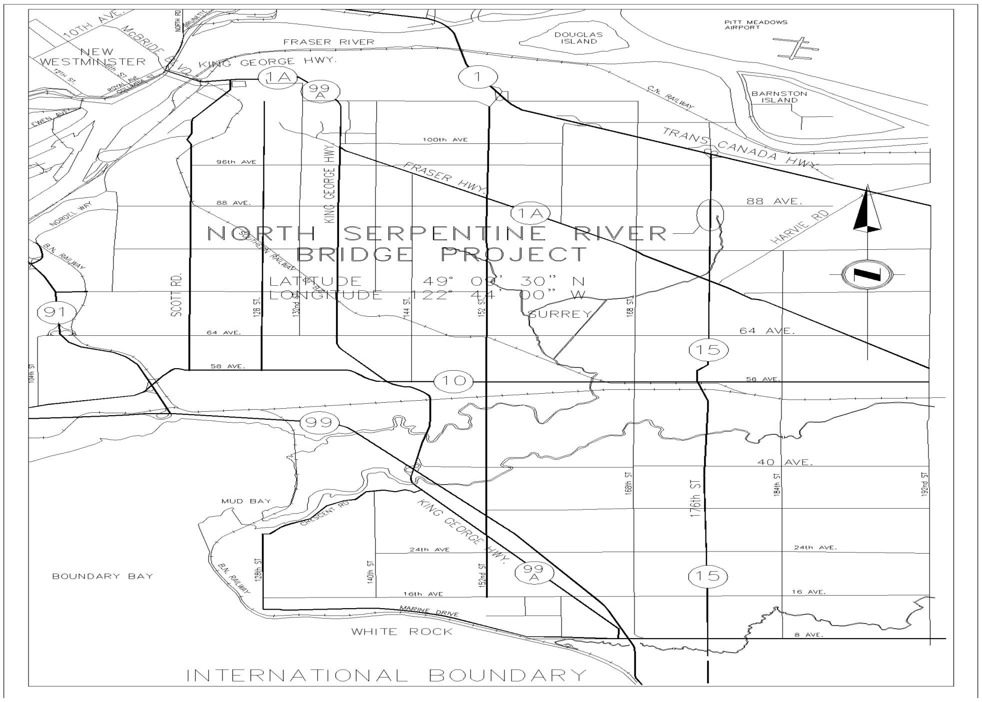

Figure 1. Key Plan Showing Location of the North Serpentine Bridge (NOT TO SCALE)

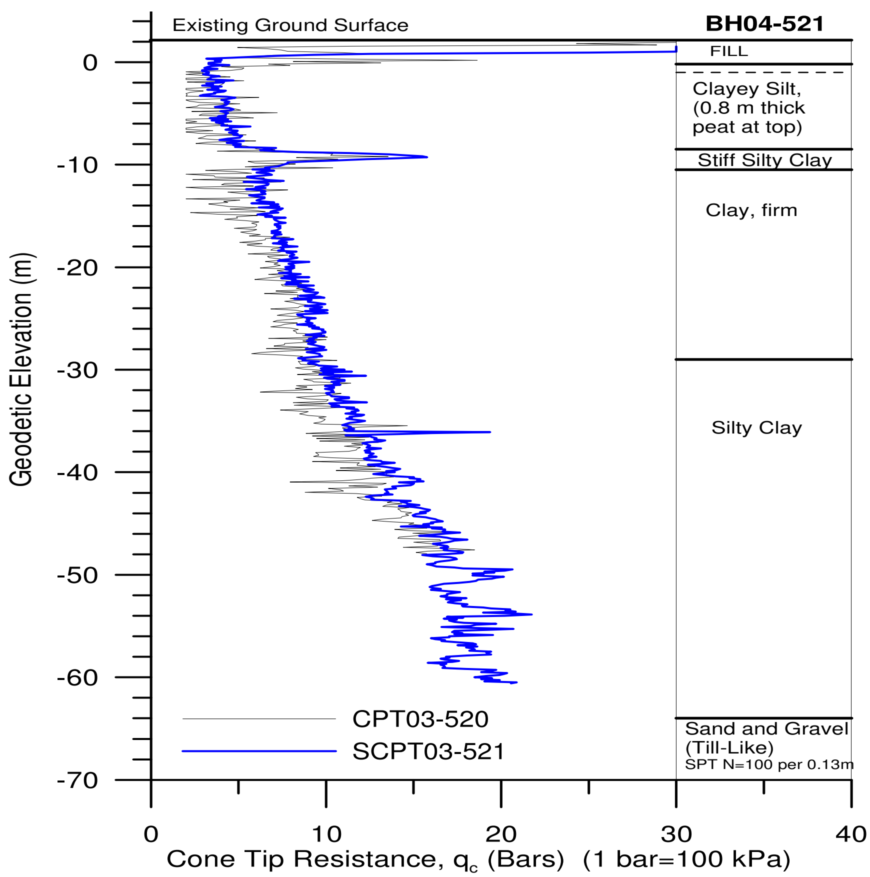

Figure 2. Borehole and Cone Penetration Test (CPT) Data at the Site

3. SEISMIC DESIGN REQUIREMENTS

The bridge site is located in an area identified as having relatively high seismic risk. The bridge and its foundation were designed for earthquakes having a 10% probability of exceedance in 50 years (a return period of 475 years), with a firm-ground horizontal peak ground acceleration (PGA) and velocity (PGV) of 0.23 g and 0.21 m/s, respectively. These values were derived from a site-specific seismic hazard analysis conducted by the Pacific Geosciences Centre (PGC) in accordance with the fourth generation seismic hazard maps of Canada (Adams and Halchuk, 2003).

4. BRIDGE FOUNDATION AND ABUTMENT FILLS

The selected foundation consists of four steel pipe piles, arranged in a row with a centre-to-centre (c/c) spacing of 3.3 m, to support each abutment wall. All piles are of 914 mm outside diameter and 12.5 mm wall thickness. The piles will be driven open-ended to a design elevation of -49 m to achieve a design load capacity of 5525 kN for each abutment. The lengths of piles were determined by the vertical load capacity requirement under the static loads.

Due to river bank stability constraints, the mineral preload fills were constructed with a setback of approximately 17 m between the crest of the Serpentine River bank and the top of the preload. Lightweight fill is required to safely infill the area between the bridge abutment and the edge of the preload. The lightweight fill options considered included hogfuel, Expanded Polystyrene (EPS) blocks, and Lightweight Cellular Concrete (LCC). LCC was selected over EPS primarily due to its high stiffness and less compressibility that are needed to provide the required lateral resistance to the abutment walls under earthquake loads. The use of LCC can increase the stiffness and strength of the abutment fill and also reduce the weight of the fill, and thus increase the stability of the abutment embankment under static loads and reduce the potential lateral ground displacements of the slope under earthquake loads.

The thickness of the LCC varies from 2.4 m along most of the northbound lanes to about 2.9 m behind the abutment wall. LCC was assumed in the design to have cast densities ranging from 450 to 500 kg/m^3, a 28-day compressive strength of 0.84 to 1.14 MPa and a modulus of elasticity around 400 MPa.

5. PILE RESPONSE FROM THE CONVENTIONAL UNCOUPLED ANALYSIS

5.1 Ground-surface Response Spectrum

A seismic site response analysis was first carried out to compute the horizontal acceleration time histories at the ground level of the soft soil site (Idriss, 1990). The analyses were conducted using the equivalent linear method of analysis and the computer program SHAKE (Schnabel et al., 1972). A soil column extending to the dense to very dense glacial deposit, assumed to be a half-space with a shear wave velocity of 760 m/s, was analyzed. The earthquake motions were applied at the firm-ground level as “outcrop” motions. Three earthquake records were selected to represent the firm-ground accelerations for the 1 in 475 year design earthquake.

The results of SHAKE analyses indicated that the ground motions are slightly de-amplified to an average peak ground acceleration of 0.20 g at the ground surface due to the presence of very thick marine silty clay to clay. As a result, the response is strong for long periods between 1.0 to 2.0 seconds.

The envelope values of the 5% damped spectral accelerations of the acceleration time histories were estimated to be 0.3g, 0.45g, 0.55g, 0.55g, and 0.10g for periods of 0.2s, 0.5s, 1.0s, 2.0s, and 3.0s, respectively. These spectral accelerations were provided to the Structural Engineer (Sargent & Associates Engineering Ltd. In Victoria, BC) for use in their response spectrum structural analysis.

5.2 Stiffness of Piles and Abutment Backfill

Foundation stiffnesses of the 4-pile group and the abutment backfills were calculated and provided to the Structural Engineer as input to the structural model. Equivalent linear soil modulus from SHAKE analyses were used in the calculation considering that the soils had strain-softened under earthquake ground shaking. The lateral stiffness of the 4-pile group was determined to be 1.0×10^5 kN/m for translation, and 2.2×10^6 and 1.0×10^8 kN.m/rad for rotations about the long and the short axes of the pile cap, respectively. The corresponding coupling stiffness values are 3.6×10^5 and 4.1×10^5 kN/rad, respectively.

5.3 Response of Piles to Structural Loads

According to Canadian bridge seismic design requirements, two seismic load combinations were analyzed by the Structural Engineer. Load Case 1 consists of 30% seismic loads in the transverse direction and 100% loads in the longitudinal direction of the bridge alignment. Load Case 2 consists of 100% loads in the transverse direction and 30% loads in the longitudinal direction. Under Load Case 1, the structural loads on the 4-pile group were determined to be 1150 kN in shear force along the short axis of the pile cap and 3025 kN.m in bending moment about the long axis. Under Load Case 2, the loads on the same 4-pile group are 1650 kN in shear force along the long axis and 3600 kN.m in bending moment about the short axis.

Pile response to the above structural loads, including shear forces and bending moments, were then calculated from a lateral response analysis of a single pile using the load-deflection (p-y curve) approach and the computer program LPILE (Ensoft, 2000). In the analysis of pile response to Load Case 1 for loads in the longitudinal direction of the bridge alignment, the loads (shear forces and bending moments) at the pile cap were evenly distributed among the piles in the group because piles are aligned in one row.

However, for analysis of pile response to Load Case 2 for loads in the transverse direction, a fixed (zero) rotation condition was applied at the pile top because of dominate rocking stiffness from the vertical resistance of piles. The moments at the pile cap were converted into axial forces in the piles. Due to the rocking effect, it was noted that the shear forces among piles are not evenly distributed and the outer piles (close to the edge of the cap) tend to take more shear forces than the inner piles (near the centre of the cap). The maximum bending moments and shear forces of the pile computed from the single pile analysis using the average loads were scaled up by approximately 10% to account for the uneven distribution of loads among piles. This procedure was verified to be reasonable by analysis using the computer program VERSAT-P3D (Wu, 2000) for quasi-3D finite element analysis (Wu and Finn, 1997) of the 4-pile group as a whole.

The maximum bending moments along the piles in the 4-pile group were determined to be 1240 kN.m for Load Case 1 and 1159 kN.m for Load Case 2. It is noted that the maximum bending moments are for the outer piles in the 4-pile group. .

6. LIMIT EQUILIBRIUM ANALYSIS OF THE ABUTMENT EMBANKMENT WITH LCC

Limit equilibrium analyses, using the computer program SLOPE/W (Geo-slope International, 2004), were conducted first to provide the screening-level assessment on the seismic stability of the abutment embankments under the design earthquake. The cross-section used in the SLOPE/W analysis contains the proposed composition of the abutment fill including LCC fill.

A minimum factor of safety of 1.50 was obtained for the static condition, based on undrained soil strengths, including the lateral resistance of the piles (1.33 without the lateral resistance of the piles). Under seismic conditions, a yield acceleration of 0.09 g was determined for a potential zone of yielding. The yield acceleration is the acceleration applied to a slip surface that results in a factor of safety of unity (1.0), which indicates that the soil within the slip surface would be subject to lateral movements toward the river with each earthquake pulse having acceleration greater than the yield acceleration. In this case, the estimated peak ground acceleration is about 2.2 times the yield acceleration.

A more sophisticated analysis is required to predict with sufficient accuracy permanent ground displacements that would occur in the abutment embankments under the design earthquakes.

7. PILE AND GROUND RESPONSE FROM A DYNAMIC SOIL-STRUCTURE INTERACTION ANALYSIS

7.1 Dynamic Finite Element Analysis

The conventional uncoupled analysis cannot take into account the effect on pile bending moment and shear force response of the permanent ground displacements of the abutment embankment under earthquake loads.

Dynamic finite element analyses were conducted using the computer program VERSAT-D2D (Wu, 2004) to provide an estimate of the permanent ground displacements of the soils behind and below the abutments under earthquake loads, and the resulting pile bending moment and shear force caused by the soil displacements. VERSAT-D2D is a computer program for dynamic 2D plane-strain finite element analyses of earth structures subjected to dynamic loads from earthquakes, machine vibration, waves or ice actions. The dynamic analyses can be conducted using linear, or nonlinear, or nonlinear effective stress method of analysis. The program can be used to study soil liquefaction, earthquake induced deformation and dynamic soil-structure interaction such as pile-supported bridges (Wu, 2001; Wu and Chan, 2002). As shown in Figure 3, the nonlinear hyperbolic shear stress-strain relationships provide simulations of hysteretic damping (or material damping) of soils subject to cyclic loads.

7.2 VERSAT-D2D Analysis Model

The VERSAT-D2D finite element model used in the analysis of the bridge crossing is shown in Figure 4. The model consists of a total of 4716 nodes, 4572 finite elements and 14 soil and structural material units. The nonlinear dynamic analysis was conducted in the time domain using an acceleration record at Saratoga Aloha Avenue of the 1989 Loma Prieta Earthquake (M=7.1). The accelerations were linearly scaled down to a peak value of 0.27g. This earthquake record was selected because its spectrum matches reasonably with the 475-year Uniform Response Spectrum for the Greater Vancouver Area.

Under each abutment wall, the four piles arranged in one row were modelled as a single beam in the 2D analysis. The equivalent bending moment of inertia of the four piles in the 2D plane strain model was estimated to be 0.0011 m^4/m (0.0036 m^4 divided by the pile spacing of 3.3 m).

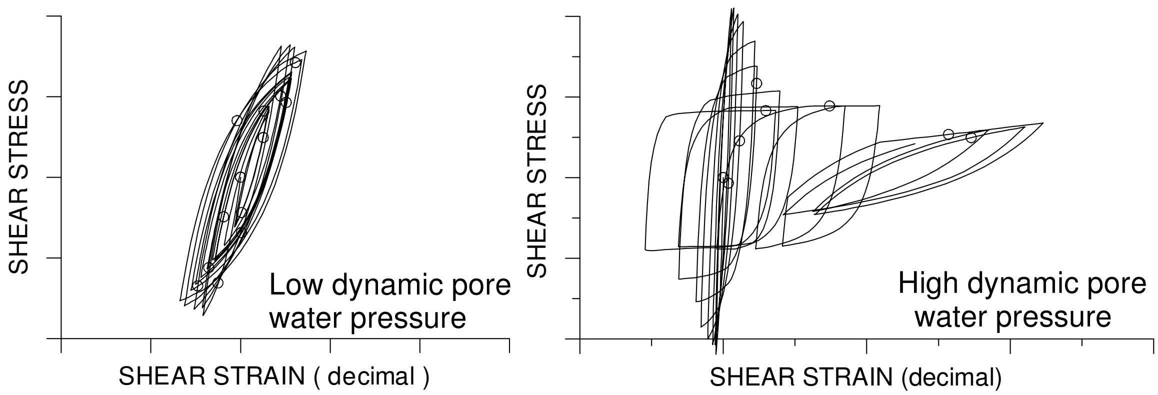

Figure 3. Hyperbolic Shear Stress-Strain Relationships Showing Hysteresis Loops of Soils from VERSAT-D2D Simulations with Harmonic Motions

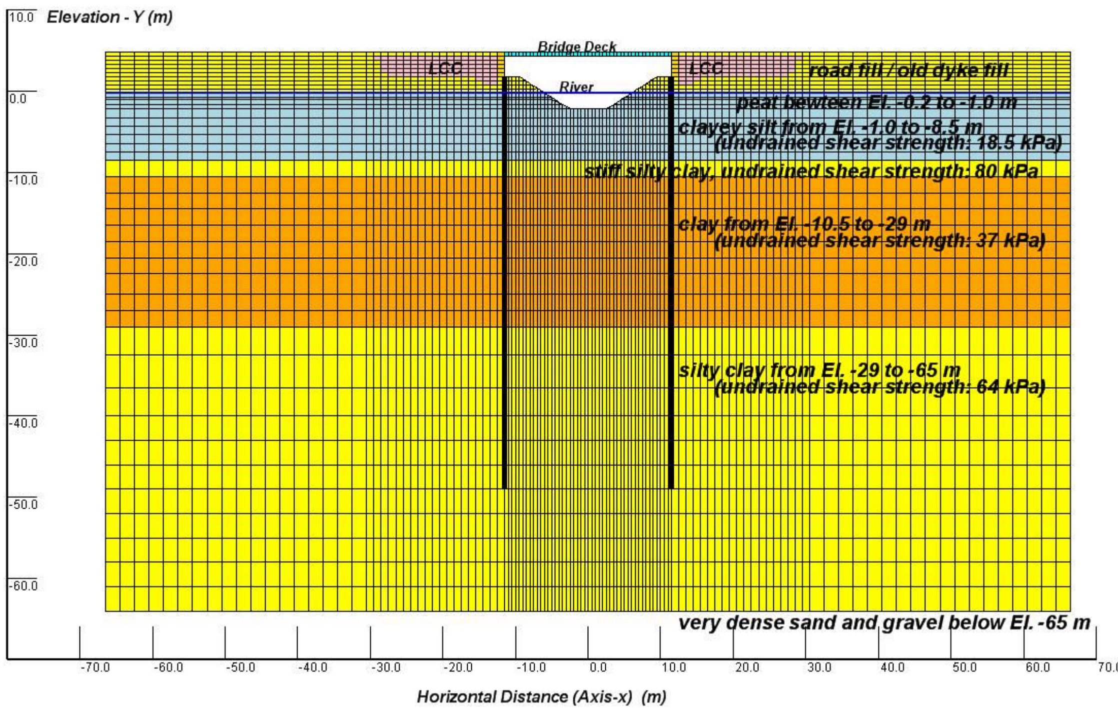

Figure 4. VERSAT-D2D Finite Element Model for Nonlinear Dynamic Analysis of Pile-Soil Interaction

The LCC fill was modeled as elastic because of its much higher strength than soils. The total weight of the bridge deck and the two abutments was estimated to be 9550 kN according to the Structural Engineer. This weight was included in the finite element model.

7.3 Ground Displacements and Pile Response

Selected results of the dynamic finite element analysis are shown in Figure 5. At the end of the earthquake, the entire bridge and its abutments were predicted to be pushed into the approach fill at one end by about 60 mm. Pushed by rotational displacements of soils at both ends of the bridge crossing, piles at two abutments bend in the opposite directions and towards the centre river. Soils and piles at one side of the crossing were estimated to have a maximum horizontal displacement of about 108 mm that results a maximum bending moment of about 2000 kN.m in each pile. The computed soil and pile displacements at the other side of the crossing result in a maximum bending moment of about 2200 kN.m in each pile, compared to 1240 kN.m from the conventional uncoupled analysis.

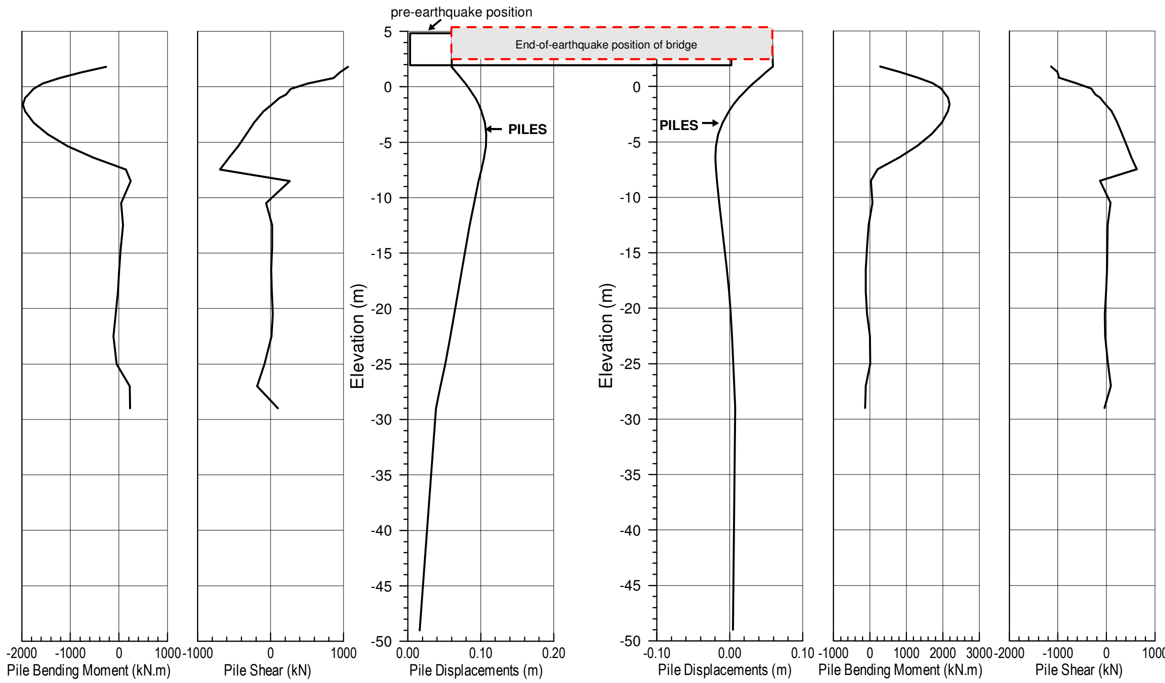

Figure 5. End-of-Earthquake Permanent Horizontal Displacements, Bending Moments and Shear Forces along Piles on the North and the South Abutments

It is noted that the calculated bending moments from the soil-structure interaction analysis are about 80% larger than that from the uncoupled analysis of the piles. This is because soil-structure interaction analysis included the effects on the piles of both the bridge and abutment inertial forces and the ground displacements. The computed bending moments and shear forces in the piles were considered acceptable under the seismic loading condition by the Structural Engineer.

8. ACKNOWLEDGEMENTS

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