Design of Piles for Electric Substation with Soil Liquefaction and Lateral Soil Movement due to Seismic Loading Conditions

H. Kristiansen
Stantec Consulting Ltd., Burnaby, BC, Canada

15 WCEE
LISBOA 2012

SUMMARY:
An existing electrical sub-station required upgrades to address future utility usage demands. The sub-station is located approximately 190 m south of the Fraser River in Richmond, BC, Canada. Subsurface conditions comprise 52 m of Holocene deltaic deposits including soils susceptible to liquefaction in the event of seismic loading conditions. Additionally, there is a possibility of lateral soil movement towards the river due to seismic loading conditions. The existing structures (transformers, bus supports, and control/switchgear buildings) were constructed in 1960 and 1980 with foundations comprising approximately 12 m long timber piles. The proposed upgrade will include similar facilities, most of which impose relatively light loads on the foundation. The evaluation of potential soil movements at the site, and their potential impact on the structure foundations, was evaluated using a 2D numerical analysis (FLAC). Subsequent axial and lateral pile design analyses were completed to address the stringent performance criterion for the proposed post-disaster structures. In summary, these analyses resulted in revisions to the performance threshold originally stipulated for the upgrades.

Keywords: soil liquefaction, lateral soil movement, pile design, soil/structure interaction.

1. INTRODUCTION

Complex analyses are required to design piles in areas with the potential for soil liquefaction and lateral soil movement caused by seismic loading conditions. Suggestions and guidelines to these analyses have been provided in numerous publications, for example (Boulanger et al, 2003; ATC-49, 2003; Brandenberg et al., 2007; Caltrans, 2008; WSDOT, 2012). A brief literature search indicated that most of these references relate to academic research or roadway infrastructure departments at public agencies. A common denominator for most of these references developed in regions along the Pacific Ocean coastline pertains to the use of the p-y curve method. Originally developed in the 1970's to support the lateral pile design for static loading conditions, it is common in North America to also use the p-y method (i.e. displacement based approach) for lateral pile design considering seismic loading conditions.

In later years, the academic research publications often include calibration of numerical models to laboratory centrifuge testing. However, this type of testing is very seldom available to the geotechnical practitioners. State-of-practice analytical guidelines are provided in some publications issued by public transportation agencies in the United States, but are not mentioned in any detail in publically issued Canadian documents. Based on this literature review and discussions with geotechnical practitioners in North America, it appears there is still significant ambiguity in the practice of designing piles for seismic conditions including soil liquefaction and lateral soil movement.

The object of this paper is to present a case history that includes pile design at a site with the potential for both soil liquefaction and lateral soil movement induced by seismic loading conditions. In consideration of the current state of uncertainty associated with design procedures for such conditions, the presented design method is an example of what could be considered yet another state-of-practice design procedure.

2. SITE DESCRIPTION

An existing electrical sub-station facility required upgrades to accommodate future electricity demands at a site located in the municipality of Richmond, BC (see site location on Fig. 1). This 290 m (north-south) by 220 m (east-west) site is located approximately 190 m south of the North Arm of the Fraser River. The topography of the site and surrounding area is relatively flat with generally site grades at about El. 4 m.

Figure 1. Site location

The proposed structures for the site will be similar to the existing structures and will comprise transformers, reactors, bus supports and a control/switchgear building (see photo of existing structures in Fig. 2). Although as-built drawings are not available for the existing structures (originally constructed in 1960 with an expansion in 1980), there are historic documents suggesting they are primarily supported on approximately 12 m long timber piles. Minor lightly loaded structures (including buried pipelines) are grade-supported.

Figure 2. Existing electric sub-station structures (courtesy of Google Earth)

The pile design presented herein pertains to the proposed heavily loaded control/switchgear building with dimensions of 20 m by 35 m. Stringent performance requirements for this lifeline facility were specified as immediate operation subsequent to the design earthquake event with a return period of 1 in 2,475 years. Prior to commencement of the detailed design phase, a vertical and horizontal foundation movement of 25 mm to 50 mm was considered a satisfactory displacement threshold for both static and seismic loading conditions.

A geotechnical site investigation program was implemented to provide supplementary information to the existing limited data. This investigation included solid stem boreholes, electronic Cone Penetration Testing (CPT), shear wave velocity measurements using the SCPT method, standpipe piezometers and laboratory soil testing. The results of deep seismic crosshole testing completed at a nearby site were used to delineate the shear wave velocity below the SCPT termination depth.

Geologically, the site is located within the Fraser River delta with surficial Holocene soils deposited in the last 10,000 years after the most recent glaciation. The thickness of the Holocene soil basin varies from negligible at the south perimeter of Vancouver to a maximum thickness of roughly 300 m at the south end of Richmond with an estimated thickness of approximately 52 m at the subject site. The results of the geotechnical site investigation indicate a thin granular fill zone of less than 1 m thickness over interbedded cohesive clayey silt soils and silt with variable clay and sand contents extending to about 8 m depth. This unit is underlain by a compact sand deposit with a trace to some fines content terminating at depths between 20 m and 22 m. A clayey silt deposit of low plasticity exists below the sand layer to a depth of approximately 50 m, where very dense till-like soil (Pleistocene epoch) overlies bedrock (Tertiary epoch). The interpreted soil stratigraphy is shown on Fig. 3 with an example of recorded CPT data.

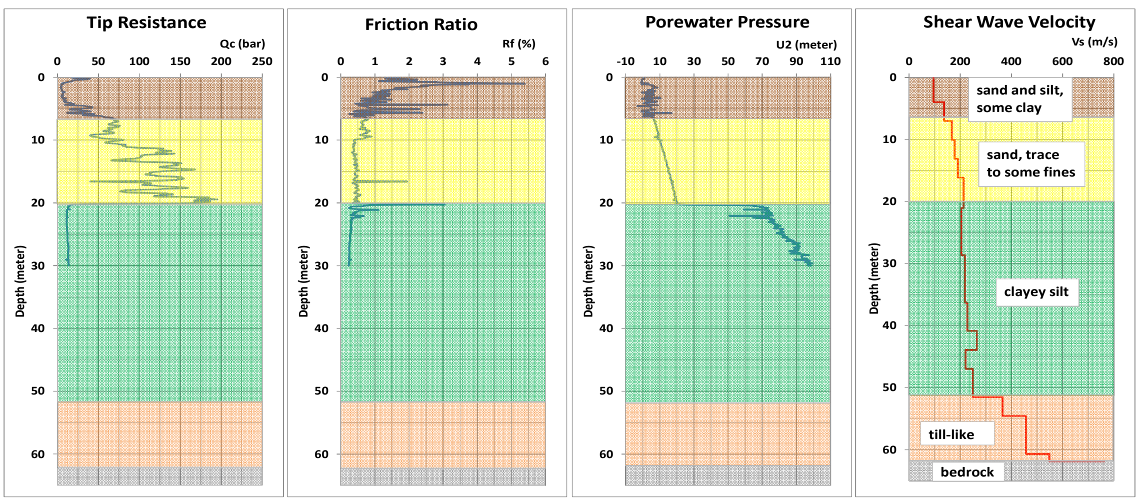

Figure 3. Sub-surface Conditions

The depth to groundwater is 1 m to 2 m with seasonal and tidal fluctuations up to 1 m. Recorded data confirms hydrostatic groundwater conditions within the upper 20 m, whereas no data is available to confirm anticipated hydrostatic conditions in the clayey silt deposit below.

3. SEISMICITY

Post-disaster requirements were stipulated for the proposed structures, which require immediate operation following an earthquake with a return period of 1 in 2,475 years. For the subject site, an earthquake with this return period would have a magnitude of 7.0 and a firm ground (i.e. Vs > 360 m/s in the upper 30 m) PGA of 0.49 g. Due to the high likelihood of soil liquefaction as well as the potential for dampening of the short period ground motion through the 52 m of relatively soft soil, a site specific dynamic evaluation was completed to address these issues. The software program SHAKE2000 (GeoMotions, 2012) was used to complete this analysis, which is based on the equivalent linear analysis method. The Pacific Earthquake Engineering Research (PEER) database of recorded earthquake time histograms was searched to obtain suitable ground motion records. Input to the search included earthquake magnitude, distance to epicentre, site class of firm ground, PGA and spectral accelerations at 0.2 seconds and 0.5 seconds. A total of 8 earthquake ground motions (four orthogonal pairs) were retrieved from the PEER data base and the response spectra for those records are presented in Fig. 4 as well as the average response spectrum and the target firm ground response spectrum. Instead of spectrally-matching the retrieved records to get an exact match to the target response spectrum, the retrieved motions were left unmodified to take into account the potential variability associated with the target response spectrum. The natural period of the proposed structures was estimated to be less than about 0.2 seconds, while the initial fundamental period of the soil column was estimated to be about 1.1 seconds.

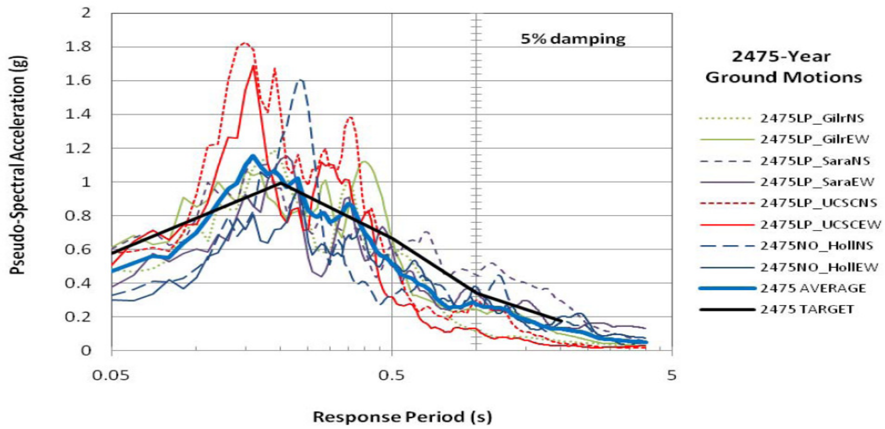

Figure 4. Response Spectra of Selected (Unscaled) Ground Motions (2,475 yr)

With the reasonable match between the average response spectrum and the target response spectrum in the general period of interest, no scaling was completed to the associated eight time histograms. The target response spectrum and the average spectrum for the input ground motions are again shown in Fig. 5, which also includes the response spectrum of the average "within" motion at the base of the Holocene soils (i.e. at 52 m depth) and the average spectrum at the ground surface determined using the SHAKE2000 program. These spectra suggest ground motion amplification factors due to the soil deposit in the range of about 1.5 to 2.0 except for periods generally between 0.1 s and 0.2 s. SHAKE analyses were also performed after scaling the recorded time histograms to the firm ground target PGA of 0.49 g. These results produced an average ground surface response spectrum that was fairly similar to that of the unscaled ground motions. The SHAKE2000 results indicated maximum soil shear strains of about 0.6% occurring a few meters above firm ground at a depth of approximately 46 m, which is within the strain range typically considered acceptable for using the equivalent linear analysis method.

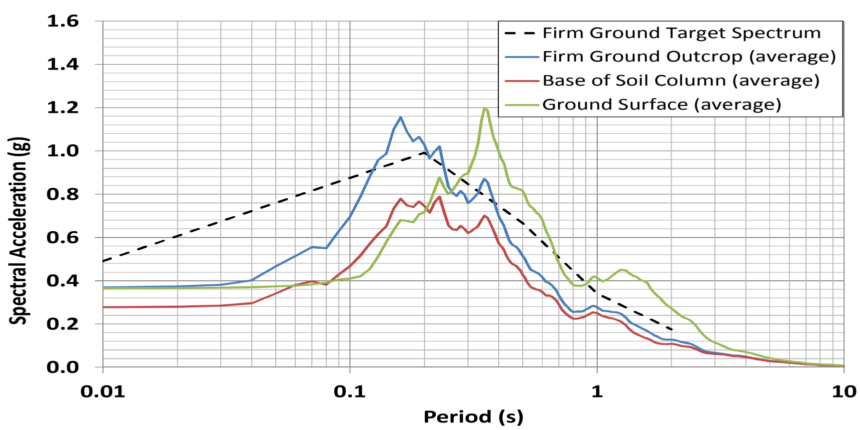

Figure 5. Response Spectra Compilation (2,475 yr)

The results of the SHAKE2000 analysis were used to determine the Cyclic Stress Ratio (CSR) induced by the 1 in 2,475 year earthquake, which indicated CSR-values typically 25% less than those obtained from the simplified Seed-Idriss method (Idriss et al, 2008). The CSR values determined from the SHAKE2000 analysis were compared to the Cyclic Resistance Ratio (CRR) values indicating the soil's resistance to liquefaction. The CRR-values were based on the recorded CPT data using the Robertson and Wride method with correction for fines content (Idriss et al, 2008). The Factor of Safety (FoS) against soil liquefaction was defined as the ratio of the CRR- and CSR-values, which indicated a high risk of liquefaction of all sand deposits to a depth of 22 m.

4. NUMERICAL SOIL DISPLACEMENT ANALYSIS

The site specific dynamic analysis discussed in the previous section indicated the presence of the soil liquefaction hazard in the event of the design earthquake. Further analyses were completed to evaluate the potential consequences of this hazard, which comprised numerical modelling using the software program FLAC (Itasca, 2008). FLAC is a two-dimensional finite difference program capable of non-linear effective stress analyses using an explicit solution scheme. The Mohr-Coulomb constitutive model was adopted for materials considered non-liquefiable and the effective stress model UBCSAND (version 904aR) was used for the liquefiable sand deposits (Beaty et al., 2011). The geometry and soil strata as approximated in the FLAC model are shown on Fig. 6.

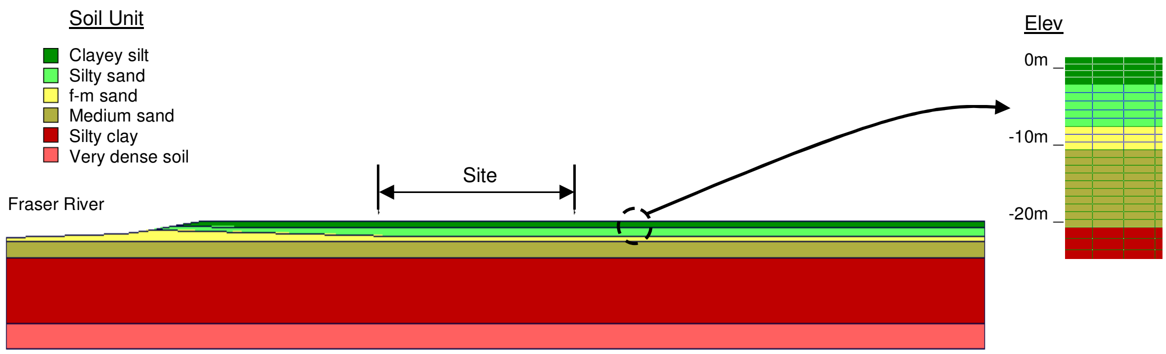

Figure 6. Cross-Section of FLAC model showing soil units

The model parameters required for each soil unit were dependent on the constitutive model and included unit weight, shear wave velocity, friction angle, cohesion, undrained shear strength, ratio of secant shear stiffness and small strain shear modulus, $(N_1)_{60,CS}$, elastic bulk modulus and post-earthquake undrained shear strength. The eight ground motions mentioned in the previous section were used as input ground motion although in one direction only (positive direction towards north).

The FLAC analyses were performed in three stages: initial static conditions, seismic earthquake response and post-seismic stability/deformation analysis. The initial stage was run until the soil stresses and groundwater conditions were in quasi-equilibrium. In the second stage, the constitutive model was changed from Mohr-Coulomb to UBCSAND in liquefiable zones to allow for direct simulation of pore pressure generation and material softening associated with pre-liquefaction and post-liquefaction behaviour. All materials were assumed to behave undrained during the earthquake. The third analysis stage considered post-earthquake stability using the residual soil strength in all liquefied zones. UBCSAND does not directly impose the adopted value of residual soil strength after liquefaction although the model captures the significant softening caused by excess pore pressure generation and soil liquefaction. Residual soil strengths were applied at any location that developed excess pore pressure ratios in excess of 0.7 during the earthquake loading or permanent shear strains exceeding 10% (i.e. it was assumed that soil liquefaction would occurred if these values were exceeded). Residual strengths were incorporated using S_u/σ'_{vo} ratios based on (Idriss et al., 2008).

An example of the pore pressure ratio distribution determined by the FLAC analyses is shown in Fig. 7. This example had final displacements similar to the median of the eight ground motions evaluated. As indicated in this figure, extensive liquefaction (i.e. r_u values exceeding 0.7) was predicted in all three sand units existing between the depths of 3.5 m and 22 m, which is consistent with the results of the simplified Seed-Idriss method discussed in Section 3.

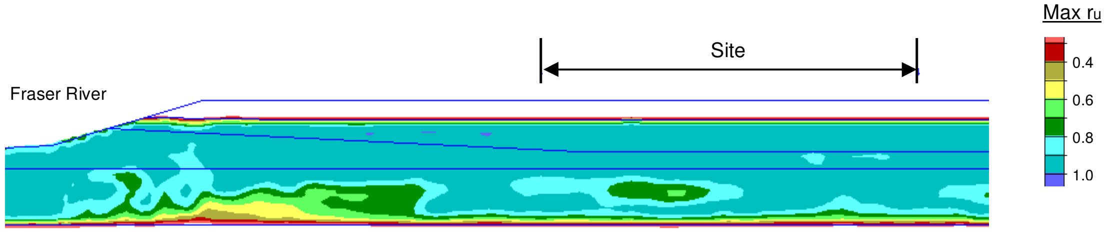

Figure 7. Example of maximum excess pore pressure ratios in liquefiable zones above elevation -20.5 m (2X vertical scale exaggeration)

Selected ground surface time histograms developed by the FLAC and SHAKE2000 analyses were compared. The surface accelerations from the non-linear FLAC analyses showed an expected reduction in magnitude after liquefaction triggering. In contrast, the equivalent linear SHAKE2000 analysis disregards the considerable softening and damping that occurs after triggering of liquefaction resulting in a relatively linear relationship between acceleration at the base of the Holocene soils and the ground surface.

Contours of permanent lateral displacement are shown in Fig. 8 from the North Arm of the Fraser River to south of the subject site. Large lateral movements are localized near the river and relatively limited at the site.

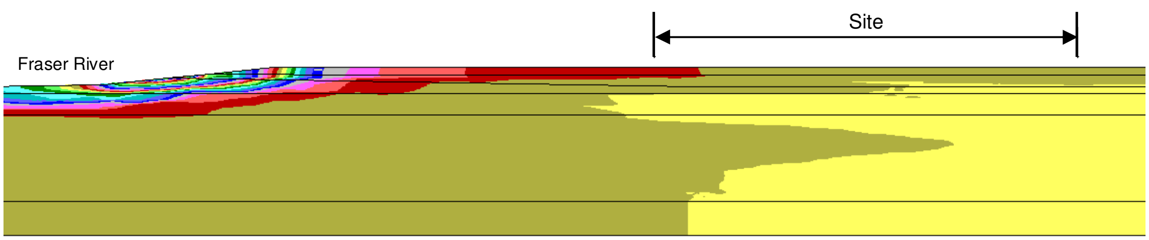

Figure 8. Example of relative horizontal displacement contours (contour interval = 0.1m)

The median permanent lateral displacements are shown in more detail in Fig. 10. The river slope effect on displacement pattern is relatively insignificant near the northern boundary of the subject site. Specifically, the median permanent lateral displacement at the ground surface was estimated at approximately 100 mm and 80 mm at the northern and southern site boundary, respectively, with insignificant lateral movement below a depth of approximately 8 m. The maximum lateral displacement from any of the eight ground motions was 200 mm.

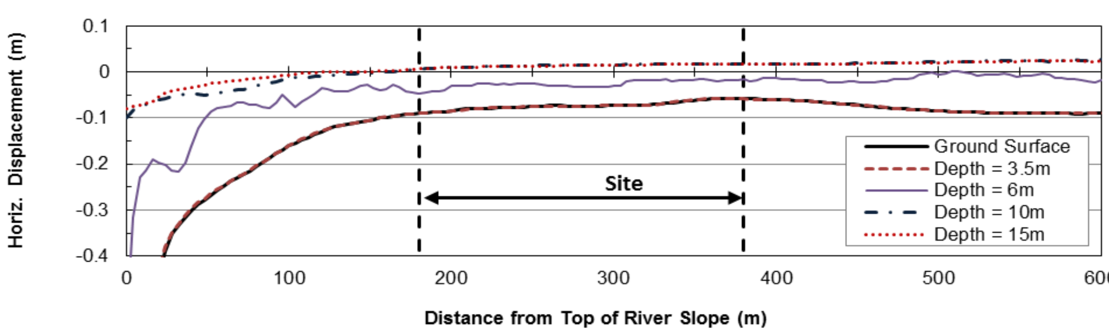

Figure 9. Median estimate of permanent horizontal displacement for selected depths

The results of the FLAC analyses were also used to provide an indication of the peak lateral displacements occurring during the seismic event, specifically differential lateral displacements of the soil along a vertical axis. These estimates suggested the maximum differential lateral displacement of the soil could be on the order of 100 mm over a depth interval of 6 m (1:60).

6. SEISMIC PILE DESIGN ANALYSES

6.1. Axial Pile Design

There appears to be a general consensus in many references that determination of the axial pile settlements for seismic loading conditions with soil liquefaction includes evaluation of the pile neutral plane defined by the depth at which equilibrium exists between downward loads and upward resistances (ATC-49, 2003; Boulanger et al, 2003). Where this equilibrium exists, there will be no differential vertical movement between the pile and the soil (Fellenius, 2004). Specifically, it has been concluded that “liquefaction of soil layers above the static neutral plane (i.e. the neutral plane that exists prior to liquefaction) will have a minor effect on the pile regardless of the magnitude of the liquefaction induced settlement” (Fellenius et al, 2008). A full-scale pile load test completed by (Rollins et al., 2006) with a sustained axial load on a pile penetrating a blast-induced liquefied soil layer and back-analyses of the resulting data agrees well with the above quote. Based on the above conclusion, the axial pile design was implemented to result in the static neutral plane being located below the liquefiable soil with the anticipation of negligible post-earthquake vertical settlements for the proposed post-disaster structure. An example of the pile load and resistance of a 610 mm driven closed-ended steel pipe pile is shown in Fig. 10, which indicates an ultimate axial pile capacity of 7,000 kN for a 40 m long pile. With the neutral plane located below the liquefiable soil, the maximum permanent axial unfactored/working service load for static conditions should be 1,400 kN.

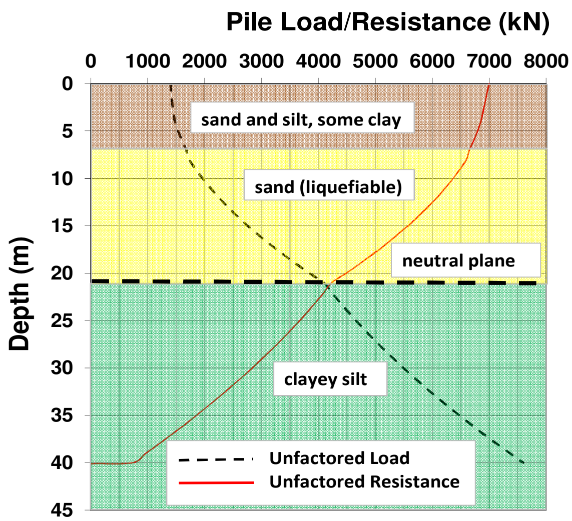

Figure 10. Axial pile design load and resistance curves (note, static neutral plane below liquefiable soils)

6.2. Lateral Pile Design

Many research studies have been completed to advance the understanding of the effect of both lateral soil movement and soil liquefaction on pile foundations. Much focus has in the last decade been on using laboratory centrifuge testing to verify numerical models (Boulanger, 2003; Brandenberg, 2007). Although coupled 3-dimensional numerical analysis models can incorporate both the inertia effects from the structure and the kinematic effects induced by the subsurface on pile foundations, this type of complex analysis is considered state-of-art and seldom used by practitioners. Furthermore, this type of analysis is still in its infancy and there is considerable uncertainty associated with the analytical results. A coupled 2-dimensional analysis method could be used albeit this would require simplifications to model the 3-dimensional pile foundations. Instead, analyses that decouple the inertia effect from the kinematic effect are readily available to the practicing engineer although simplifications are incorporated that lead to some uncertainty about the validity of the results from these simplified models. Several simplified decoupled analytical procedures have been proposed by researchers, but there appears to be an understandable lack of consensus on the analytical procedures recommended to address this complex issue.

Issues of uncertainty associated with the decoupled analyses include the timing of soil liquefaction, lateral ground movement and superstructure inertia. It is unlikely that the peak impact of these effects occur simultaneously. Numerical modelling attempting to simulate the results from centrifuge testing incorporating these three effects indicated that the superstructure inertia increased the pile moments and displacements by about 40% in comparison to only modelling the effects of soil liquefaction and lateral ground movement (Brandenberg et al., 2007). This inertia impact was associated with a structural connection between the piles and pile cap that did not develop plastic hinging and it was estimated that the inertia impact may be as low as 25% of the difference if plastic hinging would develop in the structural connection (WSDOT, 2012).

The analytical procedure to address the effects of soil liquefaction and lateral ground movement presented herein follows the suggestion by (WSDOT, 2012) and (Brandenberg et al., 2007). In summary, this displacement based approach represents these two effects by incorporating p-y curves for liquefiable soil and including the free-field lateral soil displacement profile into a computer software such as L-Pile, version 5 or higher (Ensoft, 2011). The superstructure inertia should also be considered though there is no specific consensus in this regard. There is some uncertainty associated with p-y curves for liquefiable soils as discussed in (WSDOT, 2012). For the subject site, examples of the p-y curves for liquefiable soil using three different methods are shown in Fig. 11 at the depths of 5 m, 7 m and 9 m as well as the p-y curve for non-liquefied soft clay at 3 m depth. Principally, these three methods suggest developing the p-y curves for liquefied soil as follows:

- (ATC-49, 2003): friction angle of 10 degrees and initial stiffness (k-modulus) similar to soft clay.
- (Boulanger et al., 2003; Brandenberg et al., 2007): reduce static p-y curves by the p-multiplier, which generally ranges between 0.05 and 0.5 depending on SPT-$(N_1)_{60\text{-}CS}$ values (p-multiplier = 0.1 assumed in Fig. 11).
- (Rollins et al., 2005): back-analyzed p-y curves from full scale pile load test in liquefied sand with these p-y curves capped at the recommended load intensity of 15 kN/m and a lateral deflection of 150 mm.

It should be noted that the (ATC-49, 2003) method is similar to using a reduced soil friction angle as an alternative method suggested by (WSDOT, 2012). This reduced angle is defined as follows:

$$\varphi_{reduced} = \tan^{-1} (S_r/\sigma'_{vo}) \text{ where } S_r = \text{residual shear strength}$$

In light of the similarity of the p-y curves for liquefiable soils generated by three of the four presented methods, the (Rollins et al., 2005) method was disregarded and the p-y curves for the liquefiable soil at the site was based on the p-multiplier method using a value of 0.1.

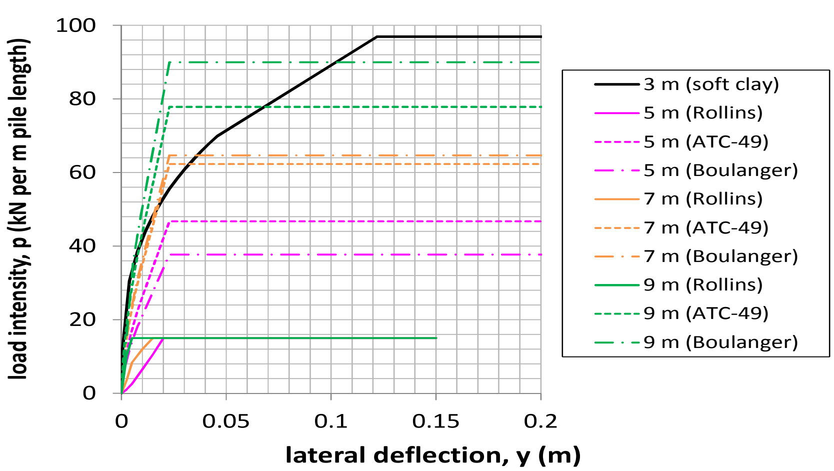

Figure 11. p-y curves at selected depths for different methods

Initial lateral pile analyses were completed for the proposed switchgear/control building assuming fixed conditions between the piles and the pile cap in an attempt to minimize the lateral pile deflections. However, it was recognized that this would result in intolerable differential lateral deflections between this pile supported structure and grade-supported pipe connections required for operation of the structure. Therefore, it was decided to revise the connection between the piles and pile cap to a pinned connection to minimize this differential lateral deflection. The foundation solution for the pinned connection included 75 steel pipe piles (OD = 610 mm, t = 19 mm), driven closed-ended to 40 m depth and concrete infilled with a lateral spacing of 3.0 m c/c. The pile cap/mat structure covered the building footprint (20 m by 35 m) and the thickness was 800 mm with a burial depth of 600 mm. The seismic superstructure loads (incl. inertia) were distributed evenly to the 75 piles resulting in the following individual pile loads: V = 965 kN, H = 160 kN, M = 0 kN-m.

The lateral pile design L-Pile model for the final configuration included the p-y curves for liquefiable soil as discussed above, the kinematic effect represented by a free-field movement distribution of 200 mm at the ground surface linearly decreasing to zero at 8 m depth as indicated by results from the FLAC numerical analyses, and various percentage of the superstructure inertia load. No pile group effect was considered for the liquefied soil p-y curves, but a reduction factor of 0.7 was applied to the p-y curves of non-liquefiable soils to represent the pile interaction for the given lateral pile spacing. The lateral deflection and moment distributions in the pile resulting from the L-Pile analyses are shown in Fig. 12.

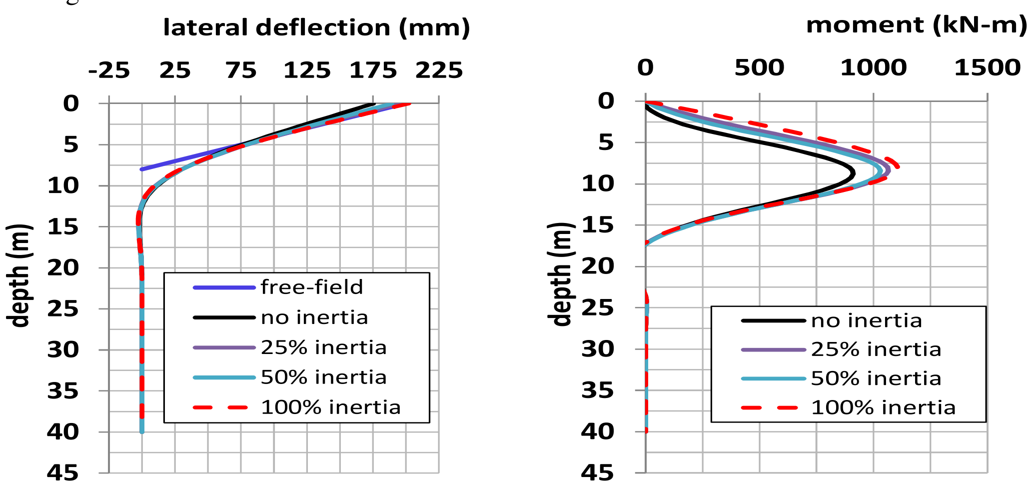

Figure 12. Seismic lateral pile design analyses: a) deflection and b) moment

As indicated in Fig. 12, the deflection and moment distributions are relatively independent of the inertia load without a notable difference as observed in the centrifuge tests presented by (Brandenberg et al., 2007). Hence, it is expected that the proposed pile supported structures will experience lateral movements due to seismic loading conditions that will be similar to that of the existing structures.

7. CONCLUSION

Evaluation of subsurface data for a proposed expansion to an existing electric substation indicated the potential for soil liquefaction to a depth of 22 m and lateral ground movement up to 200 mm in the event of an earthquake with a return period of 2,475 years. Pile foundations would be required for the proposed upgrades due to the presence of compressible cohesive soils, similar to the foundations for the existing structures. Axial and lateral pile design analyses were completed for the proposed structures, which incorporated the effects of soil liquefaction and lateral ground movement. Existing structures supported on 12 m long timber piles as well as existing/proposed buried pipelines will perform similar to the estimated free-field conditions in the event of the design earthquake (i.e. lateral movements up to 200 mm). To achieve immediate operation of this post-disaster facility after the design earthquake, the structural design of the connection between the piles and the pile cap/mat for the proposed switchgear/control building was revised from the initially assumed fixed connection to a more flexible pinned connection reducing the anticipated differential lateral deflections to less than 25 mm. Vertically, the existing structures will settle due to the underlying soil liquefaction, whereas vertical settlements of the proposed pile-supported structures will be minor.

ACKNOWLEDGEMENT

The analytical support provided by Mr. Adam McIntyre of Stantec Consulting Ltd. (Burnaby, BC) for this case-history project and the diligently preparation of many figures presented herein are greatly appreciated. Furthermore, the numerical FLAC analyses completed by Dr. Michael Beaty of Beaty Engineering LLC (Beaverton, OR) are valued as well as the constructive comments provided during preparation of the paper.

REFERENCES

ATC-49 (2003). Rcommended LRFD guidelines for the seismic design of high bridges. *Applied Technology Council (ATC) and Multidisciplinary Center for Earthquake Engineering Research (MCEER).*

Beaty, M.H. and Byrne, P.M. (2011). "UBCSAND Constitutive Model Version 904aR: Documentation Report," *http://www.itasca-udm.com/media/download/UBCSand/UBCSAND_UDM_Documentation.pdf.*

Boulanger, R.W.; Kutter, B.L.; Brandenberg, S.J; Singh, P., Chang, D. (2003). Pile foundations in liquefied and laterally spreading ground during earthquakes: Centrifuge experiments & analyses. *Report No UCD/CGM-03/01, Dept. of Civil & Enviro. Eng., U. of Davis, California, US.*

Brandenberg, S.J; Boulanger, R.W.; Kutter, B.L.; Chang, D. (2007). Static pushover analyses of pile groups in liquefied and laterally spreading ground in centrifuge tests. *ASCE Jour. Of Geo. and Geoenviron. Eng, vol. 133, no 9.*

Caltrans (2008). Soil liquefaction and lateral spreading analysis guidelines. *Memos To Designers 20-15, California Department of Transportation.*

Ensoft (2011). A Program for the Analysis of Piles and Drilled Shafts Under Lateral Loads, version 6.0.15. *Ensoft Inc, Austin, Texas.*

Fellenius, B.H. (2004). Unified design of piled foundations with emphasis on settlement analysis. *ASCE Geotechnical Special Publication (GSP 125).*

Fellenius, B.H. and Siegel, T.C (2008). Pile drag load and downdrag considering liquefaction. *ASCE Jour. of Geo. and Geoenviron. Eng, vol. 132, no 9.*

GeoMotions (2012). SHAKE2000 - A Computer Program for Equivalent-Linear Site Response Analysis, version 9.1.0. *GeoMotions LLC.*

Idriss, I.M. and Boulanger, R.W. (2008). Soil liquefaction during earthquakes. *EERI monogram MNO-12, Earthquake Engineering Research Institute, Oakland, California, USA.*

Itasca (2008). FLAC, Fast Lagrangian Analysis of Continua, version 6.0. *Itasca Consulting Group, Minneapolis, Minnesota, USA.*

Rollins, K.M.; Gerber, T.M; Lane, J.D; Ashford, S.A. (2005). Lateral resistance of a full-scale pile group in liquefied sand. *ASCE Jour. of Geo. and Geoenviron. Eng, vol. 131, no 1.*

Rollins, K.M. and Strand, S.R (2006). Downdrag forces due to liquefaction surround a pile. Proc. Of the 8^{th} US Nat. Conf. on Earthquake Eng, San Francisco, US.

WSDOT (2012). Geotechncial design manual, M46-03.07. *Washington State Dept. of Transportation.*