Case Study on the Geotechnical Seismic Design for Lateral Flow Effects on a Single Span Bridge in Pumiceous Deposits

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ABSTRACT

This paper presents a practical application of pseudo-static analysis methods to assess the impact of liquefaction and lateral spreading effects on a single spanned bridge and its foundation. The case study presented includes the assessment of seismic ground response using shear wave velocity testing to inform the kinematic loading of pile foundations. Some limitations encountered with the current guidance are discussed, specifically when flow failure is expected during or immediately following intense earthquake shaking. Insights into the assessment and modelling processes are also presented, including the different soil-structure loading scenarios for the whole bridge analysis and the incorporation of uncertainty in the assessment procedures.

Keywords: soil-structure interaction, pumiceous deposits, lateral spreading, site response analysis

1 INTRODUCTION

Bridges present both significant investments and potential weak points in transportation networks if structures fail to provide functionality following a natural disaster. The design of bridge foundations for the impact of liquefaction and lateral spreading effects is a highly complex problem. This paper presents a case study from a recent project where seismic ground response analysis was used to inform the kinematic loading of piles foundations, a key step in the assessment of the existing bridge performance, and in the design of suitable retrofit solutions. Waka Kotahi (New Zealand Transport Agency) provides relevant guidance in the Bridge Manual and published guidelines to undertake pseudo-static analysis of the bridge structure subjected to liquefaction and lateral spreading effects. Some limitations encountered with the current guidance are discussed, specifically when flow failure is expected during or immediately following intense earthquake shaking. Insights into the assessment and modelling processes are presented, including the incorporation of uncertainty in the assessment procedures, to develop a robust engineering design solution.

2 NZ DESIGN STANDARDS AND GUIDELINES

The engineering design of KiwiRail bridges and foundations is often undertaken in accordance with the Waka Kotahi/ New Zealand Transport Agency (NZTA) Bridge manual (‘Bridge Manual’) as per clause 6.3 of the KiwiRail railway bridge design brief W201 (KiwiRail 2010). The Bridge Manual adopts a performance-based design philosophy with a comprehensive set of requirements for the design and assessment of bridge foundations as well as soil structures supporting bridge abutments and approach embankments. The performance-based design is reflected in a series of seismic design limit states with varying hazard levels and corresponding performance requirements in terms of tolerable damage and operational continuity requirements.

The assessment of bridge performance when subjected to liquefaction-induced lateral spreading is a complex and technically challenging problem requiring consideration of both the seismic inertial loading of the bridge in response to ground shaking, as well as kinematic loading due to ground movement. Key uncertainties include the nature and intensity of seismic loading that a bridge may experience during its design life; the uncertainty in characterising and predicting the soil response and liquefaction triggering; and the uncertainty in characterising consequent effects – i.e. the loss of strength and stiffness and the resulting ground movements that may occur during or post-shaking; and finally the interaction of the structure to resist these movements with consequent impacts on the bridge performance.

Reflecting the complexity of the problem, NZTA provides, in addition to the requirements stipulated in the Bridge Manual, guidance documents on the analysis of bridges for lateral spreading effects in the form of the NZTA Research Note 553 (‘R-533’) (Murashev et al. 2014), with example design problems also provided (Keepa et al. 2018). These documents help inform suitable assessment procedures for the analysis of bridge foundations subject to liquefaction and lateral spreading effects in New Zealand.

Recognising that the design methods commonly adopted in engineering practice are pseudostatic, and unable to assess the complex time-dependent nature of seismic loads and effects, R-553 (and Bridge Manual clause 6.3.5) propose three load cases to consider, corresponding to different distinct phases of response: (1) peak seismic inertial loads occurring in the absence of liquefaction; (2) degradation of stiffness and strength of the soils due to generation of excess pore water pressure during shaking causing cyclic ground displacement (kinematic loads) to be considered simultaneous with structural inertial loads; and (3) lateral spreading displacement typically occurring post-shaking with minor to no consideration of structural inertial loads. R-553 also provides guidance on the different methods of analysis ranging from the simplified pseudo-static approach (PSA) to dynamic numerical analysis. The PSA method is most commonly used as it is relatively simple to implement and carry out the required sensitivity studies, however it does suffer from significant uncertainties due to the gross approximation of a dynamic problem. The two PSA methods recommended are those of Cubrinovski et al. (2009) ('CEA09'), and Ashford et al. (2011) ('PEER').

The CEA09 method broadly consists of a Winkler-type beam-spring model using simple bilinear *p-y* curves as lateral soil springs connected to vertical beam elements forming the pile, with a conceptual three-layer soil model (i.e. non-liquefied soil crust, over liquefied soil, over deeper competent founding strata). The three-layer model considers the significant difference in the stiffness and yield strength of the soil springs in liquefied and non-liquefied layers, to address the design scenarios (2) and (3) presented earlier. The Winkler-type PSA model can also be used to assess scenario (1) without liquefaction effects. The PEER method is conceptually similar to the CEA09 method but considers non-linear *p-y* curves and provides guidance on accounting for pile-pinning effects for restraining seismically induced embankment movements (see also Boulanger et al. 2007; Caltrans 2012).

The complementary report to R-553 by Keepa et al. (2018) presents worked examples for liquefaction assessment and PSA of piled bridges to demonstrate the application of R-553 recommended methods in engineering practice. This report also provides guidance on how to consider varying combinations of kinematic and inertia loading at different piers and abutments on the whole-bridge seismic response.

3 BRIDGE 83 ECMT CASE STUDY

3.1 Background

The existing Bridge 83 is located on the East Coast Main Trunk (ECMT) Line between Tauranga and Te Puke in the Bay of Plenty Region. It is situated on a straight section of track crossing the Kopuaroa Canal immediately east of Te Puke Highway. The site is underlain by Tauranga Group Alluvium which comprises alluvial and colluvial sands and gravels dominated by pumice clasts, silts, and clay with local peat beds (Leonard et al. 2010). A site investigation was conducted to characterise the ground conditions at the bridge, consisting of two machine boreholes to a depth of 40+ metres and a seismic cone penetration test (sCPT) to 25m (Beca 2022). The site was found to be underlain by approximately 10m of weak Holocene alluvium (loose sands and soft silts) overlying ignimbrite deposits of increasing density with depth. The Matua Subgroup, an ignimbrite unit dating from the Pleistocene was encountered at a depth of around 45m below the existing ground level. The ground profiles were relatively similar at both abutments and the surrounding topography is relatively flat.

3.2 Liquefaction and Cyclic Softening Assessment

An initial liquefaction hazard assessment was undertaken using the SPT- and CPT-based empirical liquefaction triggering assessment methods (Boulanger & Idriss 2014) from the results of the site investigation. The findings indicated a risk of extensive liquefaction triggering in response to design-level seismic shaking, followed by wide-spread lateral spreading and associated ground displacements. However in contrast, the shear-wave velocity-based (*Vs*) liquefaction triggering assessment method (Kayen et al. 2013) exhibited a significantly lower extent of liquefaction. Figure 1 presents the *Vs* profile as *measured* by sCPT, compared to the *estimated Vs* using a CPT-based empirical correlation (Robertson, 2009). The discrepancy has been plotted adjacent as a ratio (referred to as the 'measured to estimated velocity ratio' (MEVR) after Andrus et al. 2007), with values above 1 indicating under-estimation, and values below 1 over-estimation compared to direct *Vs* measurement. The plot shows that the CPT-based correlation over-estimates the *Vs* for the soft clay (3 to 6m depth), but more significantly under-estimates the *Vs* for the pumiceous sands and deeper ignimbrite layers.

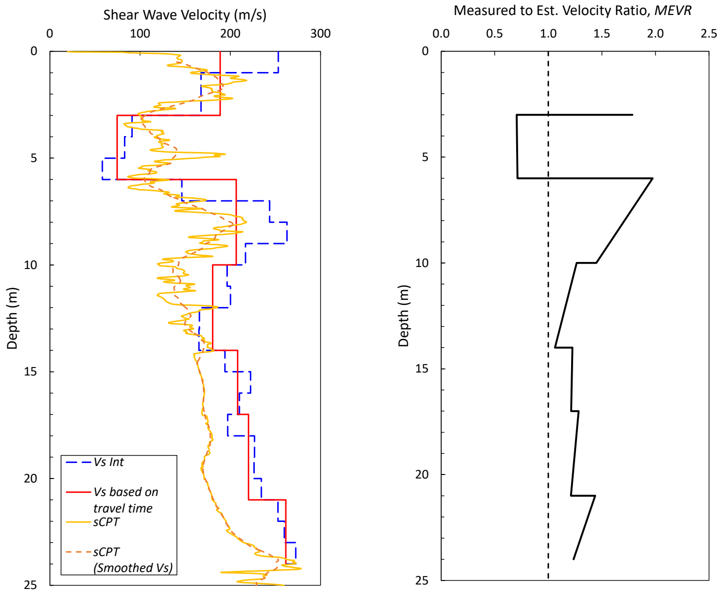

Figure 1. Comparison plots of shear wave velocity (Vs) measurements and CPT-based estimates of Vs (per CPeT-IT after Robertson, 2009). (Beca, 2022)

Problems with assessing the liquefaction hazard of pumiceous sand deposits using penetration-based testing (i.e. CPT, SPT) have been well documented in previous studies (MBIE/ NZGS Module 3, 2021; Orense et al. 2020; Clayton et al. 2019) and have not been discussed in this paper for brevity.

While clays are not susceptible to liquefaction, they may soften temporarily in response to cyclic loading, potentially leading to slope instability. The cyclic softening assessment method of Boulanger et al. (2007) was followed which is based on the results of cyclic testing of soft clays. Cyclic resistance of the clay is defined relative to the development of a specified level of strain, and for simplicity is related to the monotonic shear strength as 0.8 x s_u. Once the cyclic strength has been exceeded, potentially large displacements may occur if slope instability is precipitated, and further strain softening may occur towards residual strength values.

3.3 Bridge Substructure Analysis

The PSA method was adopted in the seismic design of the bridge piles. Non-linear soil springs were derived using commercial software LPile (Ensoft, 2019) for input into the SAP 2000 (CSI, 2021) structural software package. For sensitivity checks, upper and lower bound spring stiffness values were considered at 50% and 200% of the best estimate to account for variability of soil material response. The passive resistance provided by the existing stream banks in front of the piles has been ignored as the banks will not be stabilised. These soil springs were combined with a 3D substructure model in SAP 2000 to undertake a whole bridge analysis. Three different loading conditions/response phases were considered in the design of the piles in accordance with the Bridge Manual (section 6.3.5).

<u>Loading Condition 1 – Prior to liquefaction triggering</u>
In the pre-liquefied phase, permanent ground displacements of abutments were considered in combination with peak inertia loads on the bridge, with the kinematic loads modelled by applying the displacements to the back of the soil springs. The ground movements for a non-liquefied or cyclically softened abutment / embankment were estimated using the co-seismic ground displacement prediction equation of Jibson (2007), which was developed from curve fitting to the results of extensive Newmark's sliding block analyses. The equation required earthquake Peak Ground Acceleration (PGA), Magnitude, and the slope yield acceleration (k_y) as input.

<u>Loading Condition 2 – After liquefaction triggering</u>
After liquefaction is triggered (but prior to any flow failure occurring), cyclic oscillation of the ground surface may impose kinematic demands on the piles simultaneously with inertia loads. An 80% structural inertia load was considered during this interim cyclic phase with reduced soil stiffness and strength. The cyclic ground displacement profile was estimated from the results of 1D non-linear site response

analysis (SRA) summarised in Section 3.4. The kinematic and inertial loads were assessed in a whole bridge setting and assumed to act in the same direction (in-phase) given the short span of the bridge.
Loading Condition 3 – Lateral spreading
Based on a slope stability analysis, with softened strengths applied to units predicted to undergo liquefaction or cyclic softening, the abutments are expected to undergo a flow failure with large lateral spreading displacements following a design seismic event. This condition is identified to occur when a slope with no ground acceleration applied and with softened strength parameters for affected units, exhibits a factor of safety, FS < 1.0. The (free field) ground displacement under flow failure has been estimated to be in the order of 0.5m to 2.0m (Caltrans 2012). Given that large soil movements are expected, an effectively "unrestrained" ground displacement is characterised, and localised failure around the foundation is expected to displace regardless of the presence of piled foundations (Caltrans 2012). As a result of this, pile pinning effects have been ignored.

Different loading combinations as below were considered as part of this scenario, in combination with 25% structural inertia loads as recommended in the Bridge Manual:

- **Symmetrical Abutment Failure:** Flow failure at both abutments with full passive soil pressure mobilised in the non-liquefied soil crust, which acts as a load on the bridge abutment wing walls, and with kinematic loads from soil flowing around piles in the liquefied soil layers. The passive pressure from the non-liquefied crust was determined using the log spiral earth pressure theory.
- **Non-Symmetrical Abutment Failure:** Flow failure at only one abutment occurs, with a reduced / residual soil strength in cyclically softened layers, while reduced soil resistance is considered at the opposing abutment.

3.4 Cyclic Kinematic Loading

For Loading Condition 2, kinematic loads during ground oscillation are required to be characterised. The cyclic ground displacements for a liquefied soil deposit can be estimated from the maximum cyclic shear strains in the liquefied layers (i.e., Tokimatsu and Asaka 1998; Zhang et al. 2004). As highlighted above, liquefaction predicted under large seismic events was limited in a few thin sand horizons with ground failure mostly attributed to cyclic softening of soft clay layers. Consequently, a total stress-based 1D non-linear site response analysis (SRA) was undertaken to characterise the cyclic kinematic loads.

The SRA was undertaken using software DEEPSOIL (Hashash et al. 2020), with the soil column defined using a combination of the Vs measurement from the sCPT and SPT-Vs based correlations by Wair et al. (2012). While the MBIE/ NZGS guidelines do provide detailed requirements for SRA, the requirements in ASCE 7-16 (2017) were followed. This Standard requires that the uncertainty in the Vs profile be considered in the analysis. Therefore, best estimate, upper and lower bound Vs profiles were developed. Due to a lack of statistical test data for the site (1 sCPT, 2 Boreholes), the upper- and lower-bound profiles were characterised by adopting an assumed log-normal standard deviation, $\sigma_{\ln(Vs)}$ of 0.35 (Ulmer et al. 2021).

The non-linear soil model adopted in DEEPSOIL was that of Groholski et al. (2016) which enables calibration of the model to empirical strain-dependent dynamic shear modulus and damping properties, as well as the shear strength of the soil layer at large strains. A selection of seven input ground motions were obtained from the PEER NGA2West database (Ancheta et al. 2014). Ground motion selection and matching were undertaken in accordance with NZS 1170.5:2004, using a site-subsoil Class D target spectrum for Te Puke corresponding to the 1 in 1000yr return period hazard. The ground motions were applied at the top of the stiff ignimbrite layer at the base of the profile.

Figure 2(a) presents the site amplification ratios (i.e., ratio of surface to input motion response spectra) from the three 1D SRA models - i.e., best estimate Vs profile (BE), and upper and lower bounds (UB, LB respectively) compared to the spectral ratio between Site Class E and D spectral shape factors in NZS1170.5. There was relatively good agreement between the 1D SRA all things considered. The non-linear analysis is modelling more soil damping resulting in de-amplification (spectral ratio SR < 1) at low periods, and at the natural period of the soil profile (T ~ 1s) we see amplification (SR > 1), typically to levels expected from the NZS1170.5 spectral ratios. The results suggest the profile is behaving like a Class E profile, which was initially assumed for design. Depending on the period of the bridge structure, the Class E spectra in NZS1170.5 may be conservative (short periods T < 0.3s), about right (T 0.3 – 2s), or a bit unconservative (T 2 - 4s).

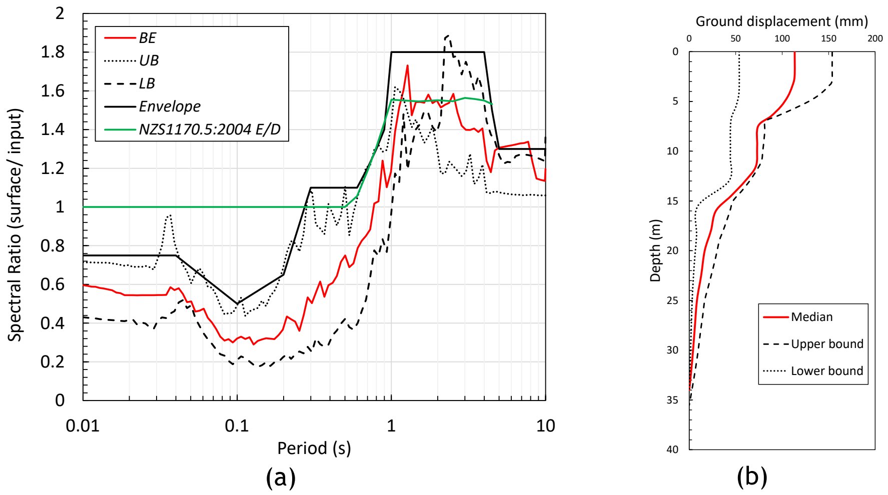

Figure 2. (a) Plot of spectral ratio for the three-site response analysis stiffness profiles considered (BE = best estimate, UB and LB upper and lower bound respectively), compared to NZS1170.5 spectral ratios based on Site Class D. (b) Ground displacement profile derived from the 1D SRA; lower bound displacement profile determined from the upper bound Vs profile and vice-versa.

The 1D SRA analysis also provides the peak ground displacement profile, which was used to characterise the cyclic oscillation loading of the bridge piles for the PSA analysis of Scenario 2 as shown in Figure 2(b). It is worth noting that the ground displacement may extend to the full depth of the foundation depending on the soil stiffness defined and the location of the rigid boundary assumed. Imposed displacements in the non-liquefying layers at depth can often lead to high soil pressures applied to the pile and can result in more onerous loading when compared to Scenario 3. This highlights the importance of considering Scenario 2, especially in soft soil sites where relatively low stiffness soils prevail. More advanced dynamic analyses can help to remove potential conservatisms in a PSA, with the trade-off being more complexity and time to undertake analyses that may not be justified.

4 STRUCTURAL-GEOTECHNICAL COLLABORATION

Good communication and collaboration between the structural and geotechnical engineer is key in modelling the soil-structure interactions of the bridge elements as noted below:

- Have a clear understanding of how the geotechnical inputs are translated in the structural model (i.e., how are the soils springs defined in the structural model).
- Be clear on how the ground displacements should be applied. Relative movement between the pile and the soil is what causes the soil springs to be loaded.
- For whole bridge analyses, consider the direction and magnitude of ground movements (i.e., in-phase or out of phase between the different abutments and piers). There is currently no consensus about what proportion of ground displacements at different bridge supports should be considered together with the inertial loads from the bridge. This requires engineering judgement and can lead to many design cases for multi-span bridges (Keepa, et al. 2018).
- It is often unclear what a conservative loading case is. An upper bound spring stiffness may provide better lateral resistance to the pile in response to inertial loads but will also apply higher soil pressures on the pile in response to kinematic movement.
- Undertake a sensitivity analysis. This can help identify the critical design scenarios / parameters.
- Consider the compatibility of the soil spring stiffness and the imposed displacements. Higher displacements often correspond to a lower-bound stiffness soil considered within sensitivity analyses, while lower displacements correspond to a higher stiffness soil case. It can be tempting to adopt upper bound stiffness with upper bound displacement combination, which would be incompatible and overly conservative.

5 CONCLUSIONS

The authors have presented a practical application of pseudo-static analysis methods to assess the liquefaction and lateral spreading effects on a single spanned bridge. The proposed procedures to assess these effects under the design seismic event together with their limitations are presented. Commonly used tests (i.e., SPTs and CPTs) for liquefaction assessment can lead to an unreliable assessment of liquefaction susceptibility, especially in pumiceous deposits. Shear wave-based testing is often beneficial and should be paired with conventional penetration tests to provide a comparison for assessing the liquefaction potential. The use of shear wave velocity testing also helped characterise the 1D site response analysis, which informed the magnitude of kinematic loads during cyclic ground oscillations for sites undergoing predominantly cyclic softening. The loading conditions listed in the NZTA Bridge Manual require further and careful considerations to define the design scenarios for a whole bridge analysis. This involves assessing different combinations of magnitude and direction of kinematic and inertial loadings on the bridge support elements to address the associated uncertainties and approximation of a dynamic problem in the context of a pseudo-static approach.

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