# NZ Transport Agency's Detailed Design Guidance for Piled Bridges at Sites Prone to Liquefaction and Lateral Spreading

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## ABSTRACT

This paper presents a summary of an example of application of pseudo-static analysis in the design of a bridge on a site prone to liquefaction. The example is one of the outcomes from Stage 2 of a research project commissioned by the NZ Transport Agency (NZTA) to develop a comprehensive design guidance for piled bridges for liquefaction and lateral spreading effects. The Stage 1 report, published on the NZTA website in 2014, provides the key design recommendations for bridges located on sites prone to liquefaction and lateral spreading. The Stage 2 report gives detailed procedures for geotechnical field and laboratory testing, liquefaction evaluation methods and examples for two bridge sites, detailed description of recommended procedures for pseudo-static analysis (including flow charts for the design process), overview of dynamic analysis methods and detailed design examples for two bridges. The design requirements and guidelines given in Stage 1 report are to be incorporated in the NZ Transport Agency's Bridge Manual and disseminated to the wider New Zealand engineering community. A summary of the Stage 2 report is given and the design procedure for the design of two bridges for liquefaction and lateral spreading effects is described in detail.

## 1 INTRODUCTION

There are many case histories worldwide where extensive damage to piled bridges has been observed due to excessive lateral ground displacements and subsidence associated with liquefaction. A variety of methods is available in the literature for the evaluation of the performance of piled bridges on sites susceptible to liquefaction. With a goal of developing a unified approach consistent with the concept of Performance Based Earthquake Engineering (PBEE), the NZ Transport Agency commissioned a research project towards the development of guidelines for the design and assessment of piled bridges at sites prone to liquefaction and lateral spreading in New Zealand. The first stage of this work has culminated in the NZTA Research Report 553 (2014). This report provides recommendations on pseudo-static analysis procedures to assess the effects of liquefaction and lateral spreading in the design of pile foundations for bridges. These recommendations have recently been incorporated in the 3rd edition of the New Zealand Transport Agency's Bridge Manual (2016).

The objective of the second stage is to provide practical examples of the use of the recommended procedures for the investigation and assessment of liquefaction and the assessment and design of piled bridges on sites with liquefiable soils. The project team comprised Opus Consultants, University of Canterbury (Prof. Misko Cubrinovski and Dr Jennifer Haskell) and Dr John Wood. Two practical examples are presented in Stage 2 report to demonstrate the Pseudo-Static Analysis (PSA) procedure. The first example illustrates performance evaluation of an existing bridge, ANZAC bridge, a 4-span reinforced concrete bridge crossing the Avon River in Christchurch. The bridge was severely affected by liquefaction and lateral spreading in the September 2010 M7.1 Darfield Earthquake and the February 2011 M6.2 Christchurch Earthquake. The second example demonstrates design of a new 2-span bridge at a grade separated intersection over the four-lane expressway north of Christchurch. This example is summarised in this paper.

## 2 ANALYSIS FRAMEWORK

The PSA procedures described in NZTA research report 553 (Murashev, 2014) are intended to provide a simplified yet accurate enough analysis tool for routine design or performance evaluation of pile foundations of a bridge. Among various PSA methods described, the displacement based approach developed by Cubrinovski is recommended in the Stage 2 report. In this approach, ground displacements are applied to the piles through bi-linear soil springs. This is consistent with the PBEE framework.

Being a simplified static procedure, three separate sets of analyses are required to capture the peak responses of piles during an earthquake, as recommended in NZTA research report 553 (Murashev et. al., 2014) and in the Bridge Manual (2013). The phases considered in the PSA are:

- Pre-liquefaction.
- Cyclic liquefaction.
- Lateral spreading.

The design is an iterative process that starts with an assumed layout and sizing of the load resisting elements followed by analysis of the piled system using the procedure outlined in Murashev et al. (2014) and then adjustments to the structure and re-analysis until the design meets the performance requirements. The Stage 2 report provides detailed guidance on the analysis procedure and how different parameters are estimated.

## 3 SITE AND STRUCTURE DESCRIPTION

This example demonstrates the use of the recommended analysis procedure in the design of a new bridge constructed as part of a new expressway between Christchurch and Kaiapoi. The bridge is part of a grade separated intersection taking a two lane local road over a new four lane expressway. The site is a relatively flat site. Approaches to the 53 m long bridge are formed with earth embankments up to 8 m high with spill through slopes at the abutments. The site geology consists of deep alluvial and marine deposits with loose sands and soft organic silt deposits in the upper 9.5 m of the ground profile. The site is within 20 km of the faults that caused the M7.1 Darfield and the M6.3 Christchurch Earthquakes in 2010 and 2011 and experienced peak ground accelerations estimated to be between 0.16 g and 0.22 g in these two events.

The superstructure, as shown in Figure 1, comprises two equal spans (2x26.50m) 16 no, 900mm deep single hollow core units seated on elastomeric bearing strips at the pier and connected integrally to the abutment cap. The foundation system comprises 3 no of 1200mm diameter concrete piles at the pier and 5 no of 900mm diameter concrete piles at each abutment. The pier piles are 24m long and the piles at each abutment are 30m long.

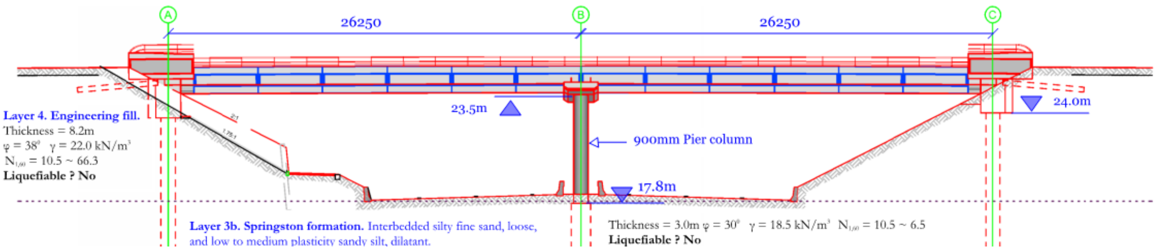


**Figure 1: Longitudinal section of the bridge with idealised soil profile**

## 4 GROUND CONDITIONS AND SEISMICITY

Ground conditions beneath the bridge and embankments have been assessed from 7 cone penetrometers, two including shear wave measurement and 4 fully cored boreholes. The site is underlain by 9.5 m of interbedded and laterally variable alluvial sand and silt over bank and flood channel deposits of the Springston Formation. Underlying the Springston Formation, from a depth of 9.5 m is the Christchurch Formation which extends to a depth of about 23 m and overlies the Riccarton Gravels. The Christchurch Formation comprises beach and dune sand deposits along with undifferentiated estuarine, lagoon and coastal swamp deposits of gravel, sand, silt, clay, shell and peat. The Riccarton Gravels, encountered from a depth of about 23 m comprise sandy and silty coarse gravel glacial outwash deposits interbedded with occasional layers of stiff to hard silt. The Riccarton Gravels are part of a series of Pliocene-Pleistocene marine sediments that are estimated to extend to a depth of about 580 m and are underlain by the Kowhai formation sediments.

As this is a local road bridge, a 1000 y return period was approved by the NZTA for the Ultimate Limit State (ULS) design. The site is classified as a Class D, Deep Soil site in accordance with NZS 1170.5. Using the method recommended by the Bridge Manual, the calculated peak ground acceleration for a 1000 y earthquake is 0.35 g. The effective earthquake magnitude for a 1000 year return period is M6.25 for northern Christchurch.

## 5 LIQUEFACTION AND LATERAL SPREADING

The propensity for significant excess porewater pressures or liquefaction to be generated in a ULS earthquake have been assessed using the empirical method by Boulanger and Idriss (2014). The fines content of each layer was estimated from $i_c$ using the relationship proposed by Boulanger and Idriss calibrated to the measured fines content in sieve analyses. Results of the triggering analysis are shown in Figure 3. Observations following the CES indicated that the simplified methods over-predicted the extent of liquefaction at some sites with thinly layered soils. Possible reasons for this are discussed in the stage 2 report. The level of over-prediction of the thinly interbedded soils were evaluated in a qualitative sense with reference to observations of the site performance in the Canterbury Earthquake Sequence (CES). The layer of sand and low plasticity silt between depths of 3.3 m and 6 m below ground level is anticipated to liquefy at ULS earthquake.

Free field vertical and horizontal ground displacement profiles for the cyclic liquefaction case and the lateral spreading case have been calculated using the methods described in the stage 2 report and are shown in Figure 2. Cyclic ground displacements of non-liquefied soils have been estimated by integrating peak shear strains determined using the cyclic stress ratio and soil's secant shear modulus.

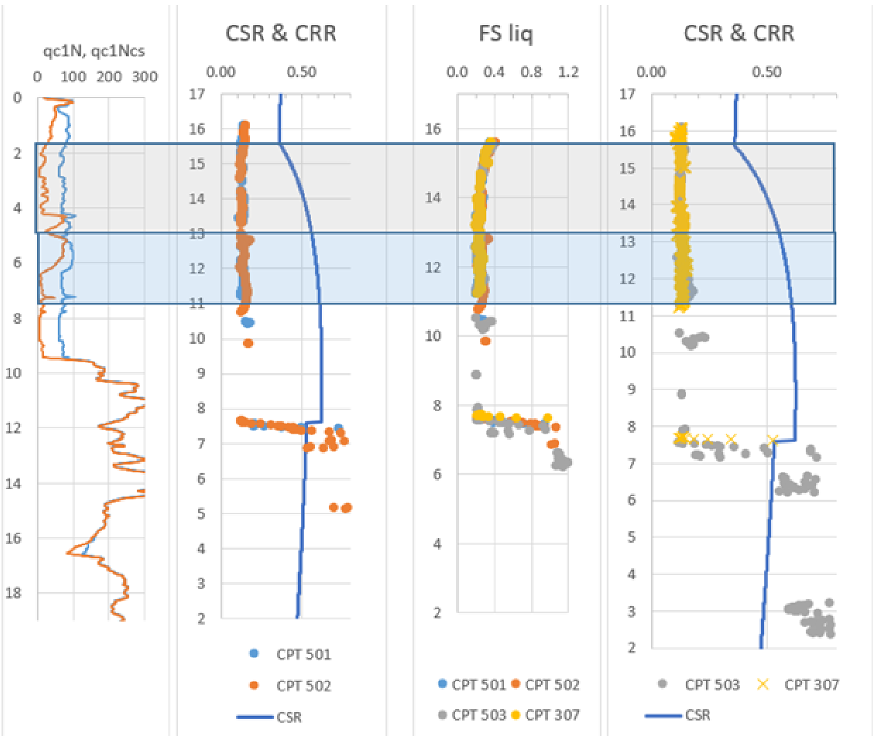


**Figure 2: Results of Liquefaction Assessment for ULS**

In estimating the free field lateral spread displacements, consideration has been given of the continuous crust overlying the liquefiable soils providing some resistance to spreading, the limited width of soil embankment driving lateral spreading and observations of spreading of the Chaneys Road approach embankments in the Canterbury Earthquake Sequence. We have assumed that permanent horizontal ground displacement of the soils below a depth of 9.5 m, (the base of the soft organic soils) will be negligible as a stability analysis shows that the relatively strong and stiff medium dense sands will constrain spreading to the soils above this layer. With the lack of simplified methods available to estimate permanent horizontal displacement of the soft organics, non-liquefied soils and the embankment fill at the abutments, permanent horizontal ground displacements in these layers have been estimated by multiplying the maximum cyclic ground displacement by a factor.

## 6 BRIDGE PILE ANALYSIS

The bridge was designed in accordance with the Displacement Based Design (DBD) method as given in the draft Section 5 of the Bridge Manual (2013). Analysis is carried out using a 3D model of the whole bridge in SAP2000. The piles are modelled using beam elements with nonlinear soil springs, to represent the vertical and lateral response of the surrounding soil.

At the beginning of the seismic design phase, the main lateral load resisting elements of the bridge, namely, the pier columns and piles, and the abutment piles, were modelled using their elastic effective member stiffness accounting for the degradation due to cracking. Geometric nonlinearity was included to capture the effect of large displacement demands on the piles, especially for the lateral spreading phase. Once the location of the plastic hinges are known, further analysis including inelasticity with plastic hinges inserted in the elements at appropriate locations is undertaken. This allows the actual rotation demand in the hinges under a ULS or Major event to be evaluated and the remaining rotational capacity and likely post-earthquake repair requirements to be assessed.

Three types of soil springs were employed to simulate the lateral and vertical responses of the surrounding soil. The lateral soil resistance was modelled using the p-y springs. T-z springs represented the skin friction along the pile shaft and a q-z spring simulated the end bearing response of each pile. All springs were assumed uncoupled, i.e., the response of one spring depends only on the respective soil deformation at the spring location and is not influenced by the response of other springs at the same node. The passive resistance behind the abutment backwall was modelled using a row of soil springs using a simplified bi-linear curve. Soil springs and ultimate pressures were calculated using the methods prescribed in the NZTA research report 553 and the Stage 2 report. The analysis model developed in SAP2000 is presented in Figure 3.

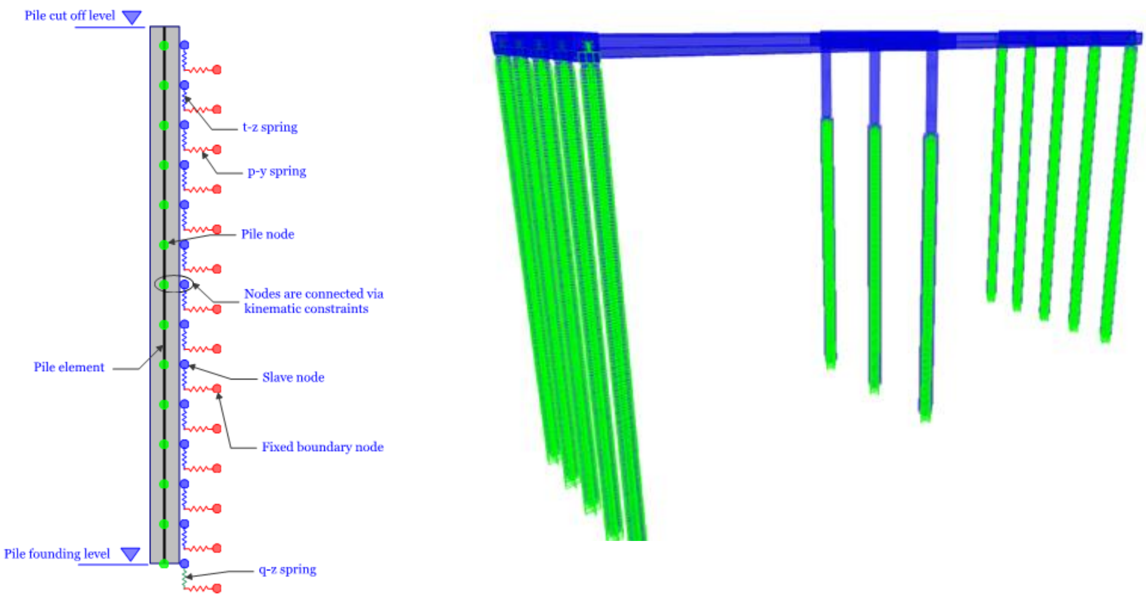


**Figure 3: Idealised pile model and isometric view of complete ridge model in SAP2000**

The peak pile response is then calculated from a pseudo-static analysis for each of the three phases representing the ground response during an earthquake, namely:

- Phase 1, cyclic phase without liquefaction (Pre-liquefaction phase)
- Phase 2, cyclic phase considering liquefaction and cyclic displacement of soils.
- Phase 3, lateral spreading phase including effects of liquefaction and lateral spreading on the pile foundations and bridge structure.

The main issue with the whole bridge model is how the kinematic loads are applied to different supports (e.g. piers and abutments) in each phase. In Phase 2, being a transient phase, the direction of kinematic displacements is not intuitive. Peak ground displacements at the opposing abutments could be in the same direction or opposing directions and may not occur concurrently at each support. In phase 3 there could be some permanent ground displacement at the pier and possibly differing extent of displacement at the abutments from differences in the ground conditions and asymmetry of the earthquake loading. There is no general consensus about how to consider different peak ground displacements at different supports together with the inertial demands from the superstructure in a pseudo-static analysis. Therefore, a significant amount of engineering judgement, as well as parametric analyses of the whole bridge model is required to envelope all the possible design cases. The scenarios considered for the three phases in the analysis of the over-bridge in this example are shown in Figure 4.

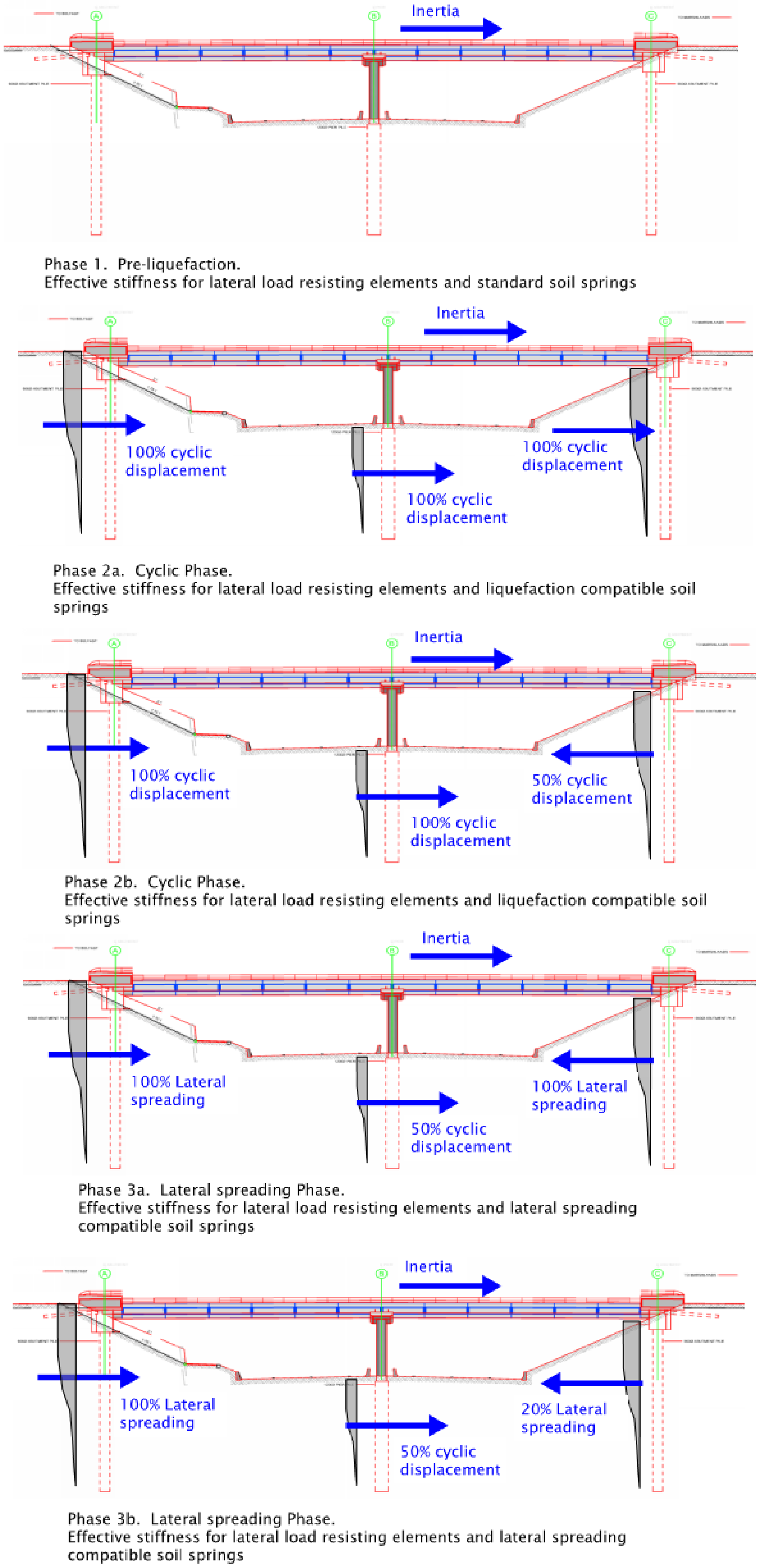


**Figure 4: Kinematic Load Scenarios for each Phase in Pseudo-Static Analysis**

Another issue is how to impose the inertial demands from the superstructure in combination with the kinematic loading. Being a nonlinear analysis, the principle of superposition is not applicable. The sequence of application can affect the location that hinges form and significantly affect the calculated system response. In this example, both loadings are applied to the model together gradually.

Inertial demands can be applied in the model in two ways. The first is via application of a force, which is determined from multiplying the bridge mass by the spectral acceleration at the first fundamental period. The second is by application of the spectral displacement corresponding to the same fundamental period. In a linear elastic system both approaches yield the same results. However, for a nonlinear system, the results can vary significantly. In this example, three cases were considered for each design phase:

- Case 1. No inertial demands from the superstructure.
- Case 2. Inertial demands applied as a displacement.
- Case 3. Inertial demands applied as a force.

The calculated sub-structure displacements and bending moments from the whole bridge analysis in the bridge longitudinal direction for the three phases including the sub-phases are shown in Figure 5 with inertia loads applied as a displacement (Case 2). Bending moments and displacements for Phase 2a with the inertia applied as a force (Case 3) are also shown on Figure 5 to depict the differences that can occur from the different ways that the inertia can be applied in the analysis. Figure 5 shows bending demands on the pier piles are critical in the cyclic liquefaction phase. At the abutment piles, bending is critical in the lateral spreading phase, Phase 3 with hinging occurring at the top of the piles, below the abutment beam.

A parametric sensitivity analysis is an important part of the PSA process. The bridge response was found to be sensitive to the magnitude of free field horizontal ground displacement and the passive resistance of the embankment fill.

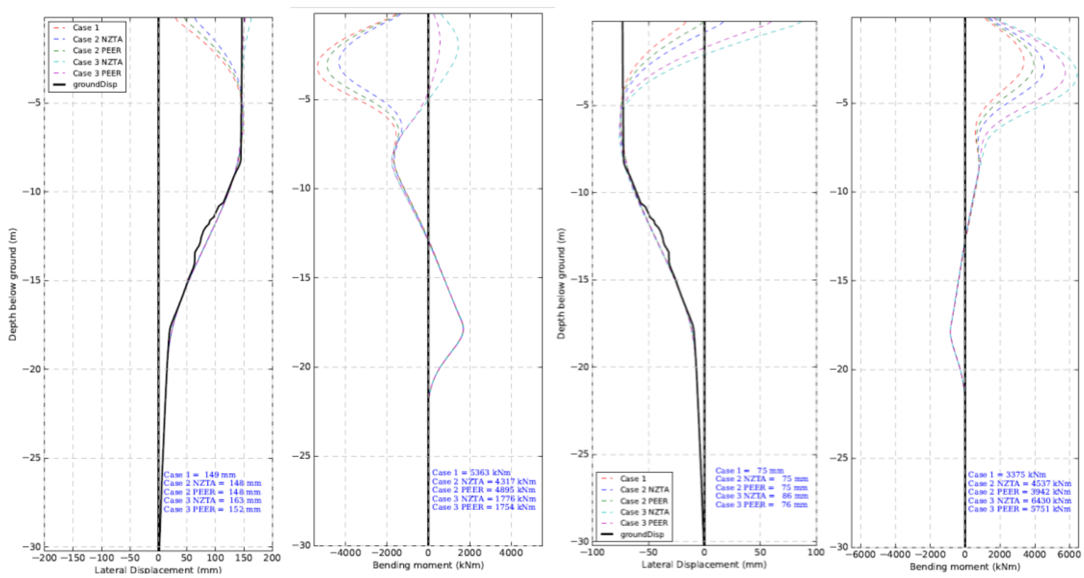


**Figure 5: Displacements and Bending Moments for Abutment Pile (left) and Pier Pile (right) from Pseudo-Static Bridge Analysis**

## 7 CONCLUSIONS

With a goal of developing a consistent approach to the analysis and design of bridges on liquefiable sites, the NZ Transport Agency commissioned a research project towards the development of guidelines for the design and assessment of piled bridges at sites prone to liquefaction and lateral spreading in New Zealand. The report for the second stage of this project provides practical examples of the use of the recommended pseudo-static analysis framework for the design of typical bridges on sites susceptible to liquefaction and lateral spreading in New Zealand.

This paper describes an example of the application of the pseudo-static method in the analysis of an over bridge with approach embankments on a relatively flat liquefiable site. The example highlights some of the challenges when applying the recommended methods, the need for parametric studies and sound engineering judgement to envelope the response and gain a good understanding of the likely seismic performance of the bridge.

## 8 ACKNOWLEDGEMENTS

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