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# Economic seismic hazard mitigation: the Whirokino & Manawatu river bridge replacement project

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## ABSTRACT

The New Zealand Transport Agency (NZTA) has the role of managing the New Zealand state highway network, including ensuring that major highway lifelines remain resilient to natural hazards such as earthquakes, and doing so in a cost-effective way. The Whirokino Trestle & Manawatu River Bridge replacement project, situated in a high seismicity zone in the lower North Island, is provided as an example of how NZTA is developing new lifeline infrastructure with seismic hazard and risk mitigation in mind, whilst also optimising the significant capital expenditure involved. This is achieved through setting economically targeted post-seismic performance requirements, as well as contractual arrangements such as Design and Construct that reward tenderers who can achieve those targets at a competitive cost. This paper presents the project background, the seismic hazard and its implications to the design, NZTA's project specific performance requirements, and how this was achieved on this project, with an emphasis on the optimised ground improvement mitigation measures achieved through performance-based earthquake geotechnical engineering.

## 1 INTRODUCTION

### 1.1 Achieving resilience of the state highway network – an economic challenge

The New Zealand state highway network is essential to maintaining the flow of people, goods and services that enables a functioning modern society. As a lifeline utility, under the Civil Defence and Emergency Management Act 2002, NZTA have a duty of care to ensure the highway remains functional albeit

proportionately at a reduced level, during and after an emergency, and have plans in place for this ([www.civildefence.govt.nz](http://www.civildefence.govt.nz)). Following natural disasters including major earthquakes, the highway network is required for the movement of emergency services and essential supplies during the immediate post-disaster phase and in facilitating the recovery and rebuild phases. The speed and degree to which it can return to full functional service is considered the level of *resilience*.

Major highway structures such as bridges present significant financial investments, but also potential weak-points in the network if the structures fail to provide functionality following a natural disaster, as the consequences of loss of service is more significant to economic and social wellbeing in terms of downtime, repair or replacement costs. The cost of new infrastructure must be weighed carefully against competing demands for government spending, with a requirement to show value for money in a cost-benefit analysis (now with increased emphasis on social wellbeing as a metric) prior to project funding approval being granted.

The NZTA Bridge Manual 3<sup>rd</sup> Edition (Amendment 2) (NZTA, 2016a) (hereafter BM) specifies the performance-based design requirements for highway structures, defined as a function of the importance level of the structure. It includes geotechnical design requirements for earthworks and retaining structures. Table 1 presents the BM earthquake hazard levels for the respective seismic design limit states for major highway bridges and non-bridge geotechnical structures, slopes and embankments. Note that the 2018 issue of the BM (Rev 3 Amendment 3) has altered the hazard levels and limit states for future projects.

*Table 1: Earthquake Hazard Level for Design Limit States for primary lifeline routes (Importance Level 3) and their resilience objectives. AEP = Annual Exceedance Probability*

	SLS 1	Minor Earthquake / SLS 2	Design Level, ULS	Major Earthquake / MCE <sup>2</sup>
Bridges <sup>1</sup> .	1 /25 AEP	0.25* ULS	1 /2500 AEP	1.5 * ULS
Major Retaining Structures <sup>3</sup> .	“ “	1/100 AEP	1/ 2500 AEP	“ “
Major Earth Slopes <sup>4</sup> .	“ “	“ “	1/ 1000 AEP	“ “
Minor Retaining Structures <sup>3</sup> .	“ “	1/ 50 AEP	1/ 500 AEP	“ “
Minor Earth Slopes <sup>4</sup> .	“ “	“ “	1/ 500 AEP	“ “

Design Limit State Resilience Objectives			
	Damage Level	Operation Level	Repairability
SLS1	None	Full	N/A
SLS2	Minor	Full	Full
ULS	Tolerable	Reduced short-term	Full
MCE or repeat ULS event	Extensive, no collapse	Temporary Loss	With reduced capacity

1. Includes major culverts and geotechnical structures and earthworks supporting bridges.
2. MCE is the maximum considered earthquake
3. Major walls have retained height > 5m and face area > 100m<sup>2</sup>. Minor less than this.
4. Major earth slopes have height > 6m. Minor less than this.

In some situations, the BM requirements can prove challenging to achieve economically, e.g. with a combination of high seismic hazard – as occurs in the lower North Island, upper South Island and Westcoast of New Zealand – and weak or unstable ground conditions (not uncommon for bridges crossing rivers). Engineered solutions can call for some measure of costly ground improvement to support the bridge

foundations that involves treating a large volume of supporting ground to increase its strength and stiffness, to either prevent or mitigate against the tendency to lose strength during earthquakes (e.g. soil liquefaction, cyclic softening) contributing to ground failure and loss of support to highway structures.

The Whirokino Trestle & Manawatu River Bridge replacement project (the Project) presented in this paper is an example where high seismicity and weak soils conspired to require significant ground improvement treatment. The key geotechnical engineering features of the project are presented with an emphasis on the cost-effective design measures undertaken to achieve NZTAs project specific performance requirements. The paper authors delivered the geotechnical detailed design of this project as part of the *Fletcher – Novare – Arup* Design & Construct team.

## 1.2 The Whirokino Trestle & Manawatu River Bridge Replacement Project

The project comprises the replacement of two structures, the 1.1 km long, 90 span Whirokino Trestle and the 180 m long, 7 span Manawatu River Bridge, located south of Foxton and north of Levin, on State Highway 1 in the lower North Island of New Zealand. The primary purpose of the project is to improve efficiency, resilience and safety. The Trestle was constructed in the mid 1930's, and the river bridge in the early 1940's and both are near end of life, requiring significant ongoing maintenance for safety. There are other issues (freight efficiency, safety and low seismic resilience) which favoured replacement of both structures. The Project will provide improved route security with stronger, more resilient, safer bridges with wider traffic lanes and shoulders, improved cycling facilities, and reduced delays for the transfer of over-size vehicles. Facilitating the road realignment, new embankments connect the bridges and feature an Agricultural (livestock) Underpass. Cut-slopes through natural dunes to the south complete the earthworks required. These features are shown alongside the alignment in Figure 1. Construction is well underway, and due for completion in early 2020.

The site is situated south of the town of Foxton in the Horowhenua district of the Manawatu-Wanganui Region, on the west coast of the lower North Island of New Zealand. The Manawatu River nears the end of its journey to the Tasman coastline (6km west of the site),

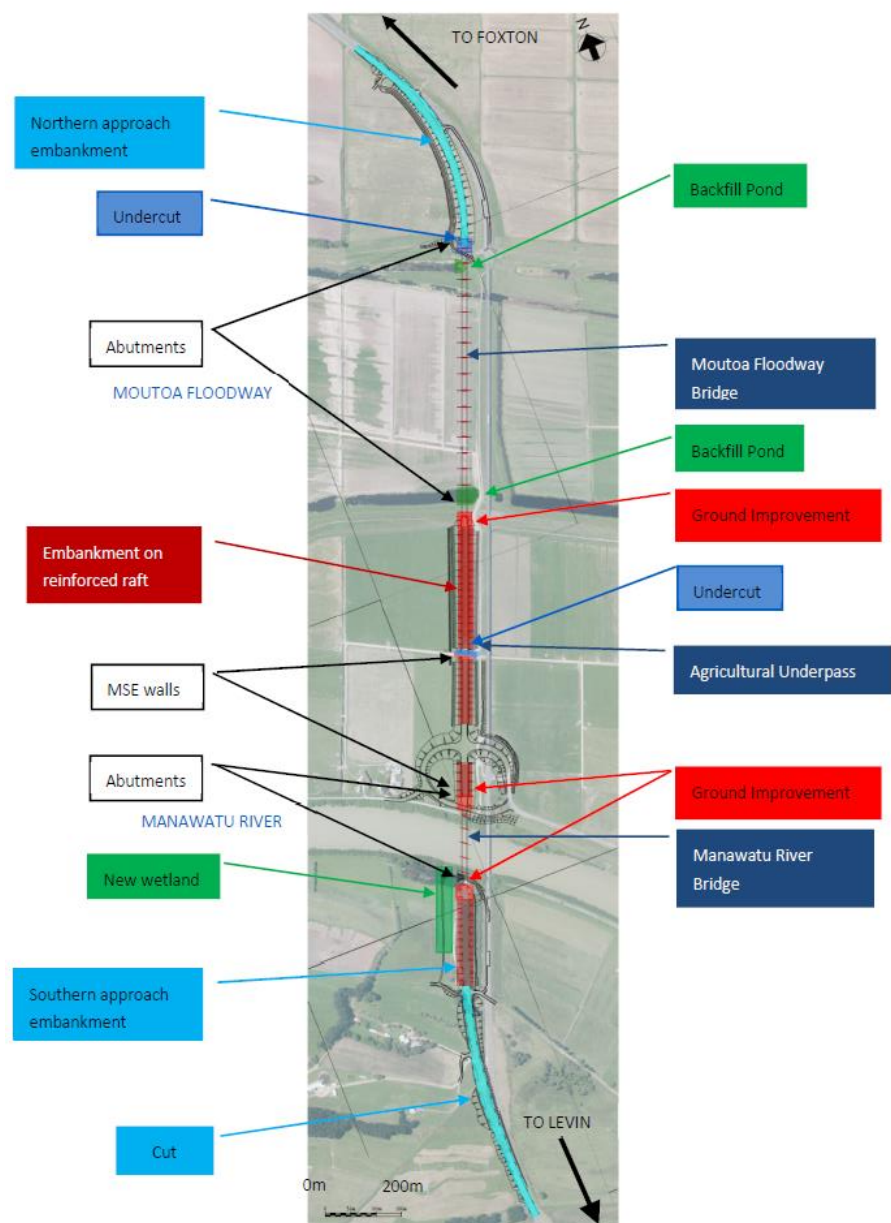


Figure 1: Project layout with annotated engineered features

which began on the eastern side of the Tararua Ranges north east of Palmerston North, before flowing south-west across the Manawatu plains to the Tasman Sea. At the site the floodplain is wide and features the man-made Moutoa Floodway constrained by levees and permitted to flood to relieve the Manawatu River in times of high flow. This was constructed in the early 20<sup>th</sup> century due to problematic flooding brought on by increased runoff owing to rapid deforestation during the late 19<sup>th</sup> Century.

## 2 PROJECT PROCUREMENT

### 2.1 Specimen Design

NZTA's Engineer to the contract, Bloxam Burnett & Olliver (BBO) had been engaged to scope alignment options and requirements for the bridges, with Earthtech Consulting Ltd providing geotechnical design services. BBO were required to manage the contract on behalf of NZTA for the detailed design and construction phases of the project, which was procured under a Design & Construct (D&C) contract.

### 2.2 Design & Construct

D&C is currently a common procurement model for infrastructure projects in New Zealand. The perceived advantages compared to the traditional model (where D and C phases are procured sequentially and separately) are: Increased cost certainty; provides a means to consider alternative and innovative design solutions and construction methodologies that may be tailored to the strengths of a particular contractor, where the D&C contractor takes on the risk for delivering a proposed methodology; benefits in integrating D&C phases in terms of managing one contract for both; and advantages in programme efficiency (such as early construction start). The potential disadvantages are the longer tender period and costs of tender design; the quality outcomes reflect the 'Principal's Project Requirements' – i.e. dependant on carefully specified requirements in the Project Brief; there is less direct control over the design process; and any risks not clearly defined or addressed prior to tender may be priced into the contract, resulting in a higher price to the client (Austrroads, 2014). Due to the contractor and designer carrying the risk of any problems with alternative methods, the degree of innovation will be constrained by their appetite for risk and is necessarily focussed on achieving cost savings rather than better quality of design outcomes for the client and stakeholders; whole-of-life considerations are typically absent.

During a competitive tender, the contractor's appreciation of procurement, programme and cost implications of the construction methodology focuses the tender design on areas where alternatives and innovation would provide a cost-advantage. For the Project, the geotechnical aspects that were targeted for innovation/optimisation during tender were the ground improvement measures (cost and efficacy of method selected, and extent required), as well as the earthworks staging to manage the impact of settlement of underlying weak soils on the construction programme. This paper has focussed on the ground improvement optimisation.

### 2.3 Principal's Requirements and deviation from the Bridge Manual

A means NZTA used to balance the desired quality of design outcomes (in terms of resilience) and manage construction costs were through the project-specific Principal's Requirements (PRs) which formed the project brief (NZTA, 2016b). Notably the PRs for post-seismic performance were relaxed from those in the BM for earth embankments away from/ having no direct impact on major engineered structures (refer Table 2), which were permissible to settle or translate larger than the specified maximum of 150mm in the BM, or even fail completely for ULS and MCE events. This lower performance target was chosen by NZTA to manage costs, recognising that earth embankments can be reinstated relatively quickly by machine excavator for emergency traffic following the rarer major earthquakes.

Geotechnical design associated with major structures still had to meet the BM performance requirements, but the maximum tolerated level of co-seismic ground displacement affecting structures (nominally 50mm in the BM) from either a single MCE event or two sequential ULS events, was relaxed to a quantitative “target”, rather than a strict performance requirement. Larger displacements would be tolerated provided the Structural Engineer could demonstrate the performance requirements would still be met in terms of resilience objectives (tolerable damage, downtime, life safety and repairability, and no collapse).

*Table 2: Project specific seismic performance requirements: Geotechnical structures not-affecting bridges & major structures.*

	<b>Serviceability Level (SLS 1)</b>	<b>Minor (SLS2)</b>	<b>Design Level (ULS) &amp; Major (MCE)</b>
Post-quake function – immediate	Full operation	Usable by emergency traffic within 12 hours after temp. repairs	None
Post-quake function – after reinstatement	N/A	Minimal reinstatement necessary to cater to all design level actions.	Reinstatement need not be addressed in this contract
Acceptable damage	No damage	Minor damage, Practical to restore to full vehicle access within 2 weeks, at min. speed of 30 km/h	Damage may be extensive

Compliance with PRs and design checks were undertaken by independent technical peer reviewers, Gaia and Roadlab for geotechnical and structural design respectively, with a second review completed by NZTA’s engineers to the contract.

### 3 GEOTECHNICAL DESIGN

#### 3.1 Geological hazards

The geology and associated hazards were major controlling factors for the geotechnical design that was developed, from the project-specific performance requirements to the design solution itself.

##### 3.1.1 Geology

The surficial geology comprises alluvial silts (with organics) and sands, with dune sands on the southern approach to the river. Ground investigations identified the Whirokino Trestle to be underlain by an asymmetric paleochannel, where the river had been cut down during the last glaciation and subsequently infilled with interbedded sands and silt-clay deposits with rising sea-levels from about 14,000 years ago to the present. The greatest depth of the paleochannel (24 m) occurs at the proposed location for the southern abutment for the replacement Moutoa Floodway bridge (refer Figure 2). The new Manawatu River Bridge is situated outside this feature with a shallower depth (10 m) of recent alluvium overlying dense Pleistocene marine sands suitable for founding piles.

##### 3.1.2 Seismicity and active faulting

The site is in an area of high seismicity, with several active faults within close-proximity (refer Figure 2). The most significant in terms of slip-rate (5.5 mm/year), and therefore lowest recurrence interval (RI = 850 years) is the Wellington Fault, located 28 km from the site, and is capable of a M 7.5 event. Other major faults include the Northern Ohariu (RI = 2,550 years) and Rangitikei Offshore (RI = 3,830 years)

located between 14 and 18 km from the site and capable of M 7.2 – 7.4 events. Some likely fault-controlled anticlines are also noted close to the site (Langridge et al. 2016).

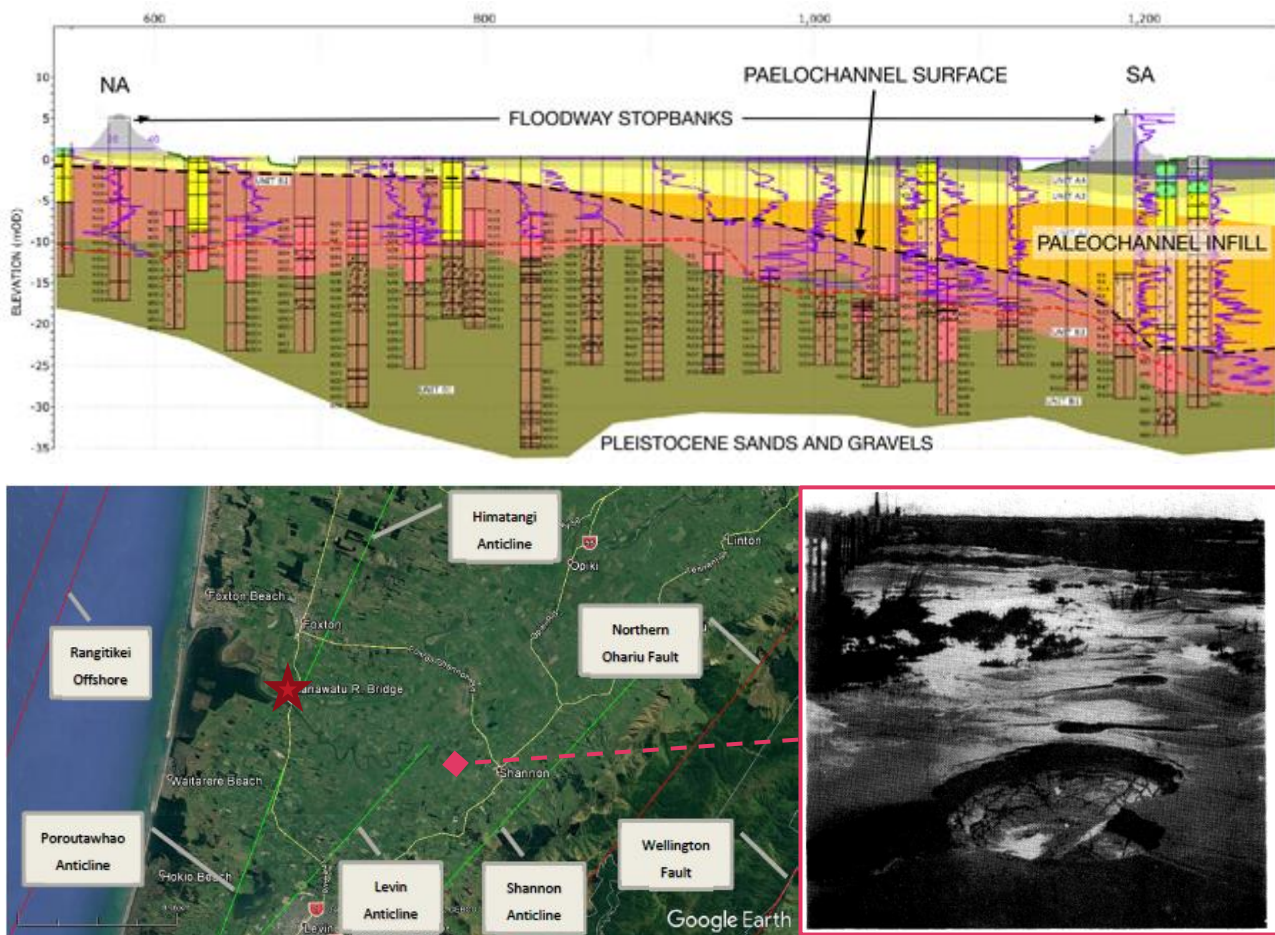


Figure 2: Top: Geological section (with borings and CPT overlaid) oriented North abutment (NA) -South abutment (SA) across the Moutoa Floodway showing Paleochannel infilling. Bottom Left: Active faults and fault-controlled features (anticlines) near site (red star). Bottom Right: Sand boils at Paika, near the Manawatu River, caused by the June 24 1942 Wairarapa earthquake. Source: Downes et al. (2001)

Historically the area has been subjected to significant shaking from large regional earthquakes, including two M 7 – 7.6 events centred near Wanganui in the 1840s, the M 8.2 1855 Wairarapa event, two ~ M 6.8 events near Palmerston North around 1880, and the M 7.6 1934 Pahiataua event (near Masterton). Post-construction of the Trestle bridge, two major events occurred in 1942; the M 6.8 & M 7.2 Wairarapa (Masterton) quakes which generated Modified Mercalli Intensity VII (very strong) shaking in Manawatu, and well documented reports of liquefaction and lateral spread related cracking along the river banks not far from the site (refer Downes et al. 2001, Figure 2). There were no reports of damage affecting the existing bridges or embankments at the site itself however. A median PGA estimate of 0.16g at the site for this event was obtained from the ground motion prediction equation (GMPE) of Bradley (2013), a similar amplitude to the design SLS2 earthquake (1/100 AEP).

### 3.1.3 Seismic hazard and site response considerations

The Project design ground motions were as defined by the BM, which is based on the Stirling et al. (2002) National Seismic Hazard Model. The BM usually requires this to be updated for high value projects by undertaking a site-specific Probabilistic Seismic Hazard Assessment (PSHA), which we would normally also advocate in order to better inform the design and incorporate 15 years of research improvements, however in

this case NZTA had indicated in the PRs that there was no requirement, judging by other major NZTA projects in the lower North Island which derived essentially the same design ground motions as the BM. Aside from this disincentive, a site-specific study may have been beneficial in the following ways:

- to constrain the design MCE event based on site ground motions predicted from nearby major active fault sources, rather than simply factoring by 1.5 the ULS ground motions, from 0.58g to 0.87g, which is very onerous and considered an unlikely scenario given the distance to the major active faults and deterministic 84%ile predictions from a modern GMPE indicated lower PGA;
- to provide hazard deaggregation data to inform the selection of ground motion records for advanced dynamic analyses, however the methodology for either structure or geotechnical design did not require this at detailed design.

In a D&C environment the PR's decision not to undertake a site-specific study was not challenged, as any perceived benefits would be focussed on efforts to achieve cost savings, rather than improve hazard estimates from a technical standpoint.

The benefits of non-linear site response analysis were also considered at tender phase to improve surface ground motion estimates over the use of code-based site class factors. The high design PGA values specified at ULS and MCE are theoretically not permissible with the weak soils present at the site; it would be expected that ground surface PGA would be dampened considerably by soil shearing and deformation. However, large PGA values were recorded on soft soil sites in Christchurch in February 2011 (e.g. Resthaven Geonet station), a similarly alluvial site featuring weak near surface deposits overlying a deep sedimentary basin. This may have been due to locally variable stiffness of soils or possibly the influence of surface waves (Bradley B.A. pers. comms. 2016). There was some doubt in the ability of 1D site response methods to accurately estimate ground surface PGA in such a similar situation, and consequently not pursued further. Any benefit that would occur was not quantified but merely considered qualitatively to likely add to the design's resilience. The influence of the site response of the embankments was considered in a simplified manner during stability analyses discussed later in this paper.

#### 3.1.4 Liquefaction hazard

The liquefaction hazard assessment indicated extensive triggering would be expected during the ULS and MCE design earthquakes in the paleochannel fill materials and recent alluvium deposits in the upper 10m at the Manawatu River Bridge abutments. Limited triggering in discrete layers was also predicted during the design SLS2 earthquake, although the highly stratified nature of the deposits (discussed further subsequently) and the lack of observations during the 1942 Wairarapa events discussed previously indicates significant liquefaction is unlikely. This was not considered to be enough evidence that triggering could be ruled out during a future SLS2 earthquake, due to variations in energy content, frequency and duration that might reasonably be expected to occur; and recognition that the uncertainty in the input earthquake energy in a liquefaction assessment is much greater than the uncertainty in the triggering model itself (NASEM, 2016).

One improvement over the specimen design evaluation was to recognise that the paleochannel deposits were highly stratified, featuring thin interbedded layers of sand and non-liquefiable silt-clay. The CPT test which the triggering assessment is based upon is unable to accurately resolve the density of thin layers of sand (i.e. < 1 m thick, refer Boulanger et al. (2016)), where measurements of thin sand horizons are affected by underlying soft clay layers, and thus CPT will tend to under-predict the liquefaction resistance. The thickest layers within the deposit were used to inform the expected density and triggering resistance of the thin layers formed in the same geological environment. The interbedded sands were predicted to trigger during ULS shaking, but importantly not trigger as extensively during SLS2 shaking, settlement estimates were reduced to tolerable levels, and furthermore the liquefied shear strength was notably greater - which improved embankment stability estimates. Significant conservatism was retained in that the influence of the non-

liquefiable thin clay layers within the interbedded deposit were effectively ignored, and the entire unit considered to exhibit the strength of the liquefied sand because of the complexity of the layering. This assumption also provided additional resilience to the design solution adopted.

### 3.2 Ground improvement design

The bridge abutments were the most critical slopes considered on the project, due to the high amplitude design earthquake motions [ULS 1/2500 AEP PGA of 0.58g, MCE PGA of 0.87g] underlying weak alluvial soils which triggered for liquefaction and were further surcharged with an earth embankment, and tight performance tolerances due to directly supporting the bridge structure. Here ground improvement comprising a cellular grid of concrete columns formed around the bridge abutment piles was designed to provide additional strength and stiffness to the alluvial soils (refer Figure 3 for layout). The grids were formed by overlapping low strength (min 2 MPa) Continuous Flight Auger (CFA) piles, the extent of which was optimised to:

- Prevent liquefaction triggering of soils within the cells at SLS and SLS2 level events.
- Provide confinement and containment of soils within cells at ULS and MCE shaking, and thus ground displacements remain within tolerable levels for the bridge abutment piles during rare events.

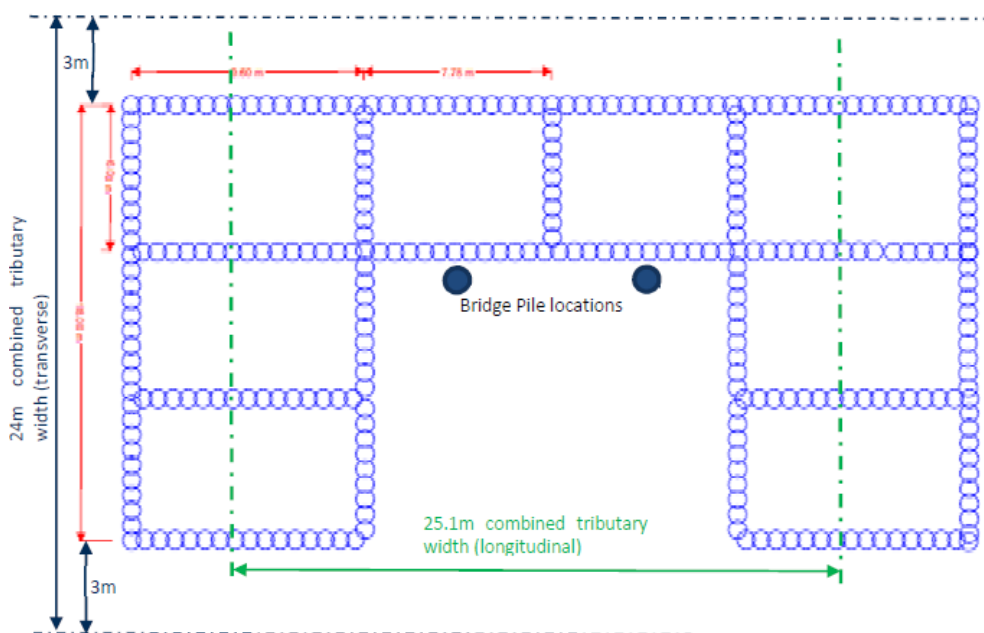


Figure 3: Plan of CFA cellular grid arrangement installed in front and to the sides of the bridge abutment piles, where the approach embankment ground slopes away from the centre. Diagrammatic/ not to scale.

The CFA piles had the advantage that they can be constructed in silty soils that can be difficult to treat with densification techniques and were the preferred technique of the constructor.

The ground improvement design comprised:

1. External stability analyses to confirm the required minimum geometry (width, length and depth) of a treated zone or “block” of improved ground at each of the abutments, and minimum equivalent composite shear strength requirements of the block;
2. Internal stability calculations to confirm CFA column strength was adequate for the following failure mechanisms:
  - i. Liquefaction of confined soils;

- ii. Brittle column failure during cyclic loading, in bending or shear.

A summary of the methodology for each aspect follows.

### 3.2.1 Embankment stability analyses

Relatively simple pseudostatic methods were adopted i.e. limit equilibrium slope analysis using software *Rocscience Slide* (version 7, 2017) to establish factors of safety and the yield acceleration ( $k_y$ ) at which slope failure initiated. Published regression equations based on rigid or dynamic Newmark sliding block models were used to estimate co-seismic slope displacements from the extent to which the design PGA exceeded  $k_y$ . Despite this relatively simple analysis approach, the design process is not straightforward with many scenario analyses required to inform and demonstrate the robustness of the design. The following considerations were made during the optimisation of the extent of ground improvement required:

- The extent of reasonable slope failure mechanisms was restricted to 4x the height of the slope, due to a limitation of the pseudostatic method (after Caltrans, 2012).
- Equivalent peak horizontal ground seismic coefficient,  $k_{max}$  for use in the pseudostatic analyses was determined for each of the respective design events using the predictive equation for the ratio of  $k_{max}/PGA$  presented by Rathje & Antonakos (2011) as a function of the natural period of the slope mass,  $T_s$ , the mean period  $T_m$  and amplitude of the incoming motion (PGA). This resulted in slight amplification at low PGA values (SLS and SLS2), but significant reductions for the ULS and MCE hazard levels.
- Regression equations for co-seismic slope displacement which incorporate the dynamic site response such as Rathje & Antonakos (2011), indicate significant damping would occur to reduce predicted displacements in liquefied cases. This was conservatively assumed to not apply to regression equations based on rigid sliding block models (e.g. Ambraseys & Srbulov, 1995). Consequently, the latter governed the slope displacement estimates for liquefied design cases, while for non-liquefied design cases, the dynamic regression models governed.
- For two successive ULS earthquakes, full mobilisation of the CFA grid peak strength was expected to occur during the first event. This meant that the subsequent event had to account for a reduction in CFA strength to residual properties (< 50% of peak strength values based on published testing, after Kivelo (1997)).
- The ‘pile-pinning effect’ - i.e. the mobilisation of pile shear resistance to provide stabilising force to the slope was considered in the *Slide* analyses (refer Martin et al. 2002; Boulanger et al. 2006; Ashford et al. 2011; Caltrans 2012; Murashev et al. 2014). The approach considers both the co-seismic ground displacement curve as a function of mobilised pile shear support (combining results from *Slide* analyses and Newmark regression equations), and a load-deflection curve for the piles derived from a pile push-over analysis (example figure shown in Figure 4). The latter was undertaken using software *Oasys ALP*, a Beam-on-Non-Linear-Winkler-Foundation (BNLWF) type model of a laterally loaded pile, where the pile-soil interaction is represented by non-linear  $p$ - $y$  springs. Modelling to account for liquefaction and lateral ground displacement followed the well-documented pseudostatic method (e.g. Cubrinovski et al. 2009).
- Layers of high strength geotextile reinforcement assisted in stabilising the abutments.

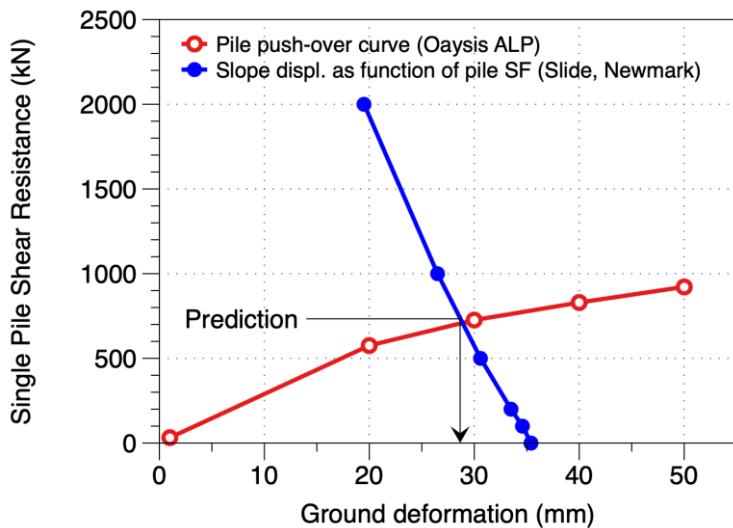


Figure 4: Force-displacement curves for pile pinning analysis (Example shows first of two design level (ULS) earthquakes affecting the Northern Abutment of the Moutoa Floodway Bridge)

### 3.2.2 Internal stability calculations

#### Controlling liquefaction of confined soil

The concept is that the cells are sufficiently rigid that the cyclic shear strains mobilised in the confined soil are low enough that triggering does not occur. The assessment procedure follows a cyclic-stress approach building on the familiar simplified liquefaction triggering procedure (e.g. Boulanger & Idriss, 2014). The method followed, after Nguyen et al. (2013), considers two effects of the installation of the CFA grids:

1. The effect on the dynamic site response and hence the ground surface PGA;
2. The spatial distribution with depth of the mobilised cyclic shear stress and strains during the design earthquake.

A shear strength reduction factor,  $R_{rd}$ , is calculated, defined as the ratio of soil column flexibility factor,  $r_d$  (as per the simplified liquefaction triggering procedure) for both a treated and untreated site. The design is optimised through iteration to determine the required number of CFA cells forming the grids that confine the treatment area, such that enough ground has been treated that liquefaction is not expected to trigger during the design SLS2 event in the untreated soil confined within the cells, and either prevented or confined within the cells at ULS and MCE hazard level (in practice targeted reduction of effective PGA during a ULS event to SLS2 level). This defines the required cement/soil area ratio,  $A_r$  (ratio of area of cement treatment to surrounding soil in the treatment area in plan). Replacement ratios required to completely and categorically prevent liquefaction triggering within the cells at ULS and MCE hazard levels were not considered economic or indeed required from a performance perspective.

#### Checking for brittle column failure

Further design checks were also carried out to confirm that the tensile capacity of the CFA columns exceeded the maximum seismic demand on the columns during the design seismic event, with a suitable factor of safety following guidance in Porbaha (2000).

The combined static shear stress beneath the embankment, and peak seismic shear stress impacting the grid (estimated using the pseudostatic approach after Seed & Idriss, 1971) was compared to the strength in bending of a single CFA pile (considered conservative owing to the column overlap to form intersecting

walls), also based on a relationship presented by Porbaha (2000), which resulted in a factor of safety of more than 2.0.

## 4 SUMMARY & CONCLUSION

The Whirokino Trestle & Manawatu River Bridge project provides an example of seismic engineering design practice for the development of critical infrastructure in a challenging high seismicity environment with weak alluvial soils. The intersection of economic drivers to achieve NZTA's required level of resilience at favourable cost has been managed through the D&C contractual framework. This provides incentives for the D&C delivery team to optimise for construction costs, while demonstrating minimum performance requirements are achieved by design. NZTA's project brief (outlining the Principal's Requirements) assisted the economic side of the equation by selectively relaxing the requirements in the BM and tolerating further damage to earthworks following rare major earthquakes, while maintaining high performance requirements for the more critical structures, and during more frequent design earthquakes.

In our experience the D&C procurement encourages further optimisation but discourages over-analysis or state of the art approaches in keeping design costs modest. This in turn reduces risk of design changes affecting programme and costs and helps to provide ease and transparency in the peer review and compliance checking process. Where advanced analyses could have been considered have been highlighted.

Deciding if ground improvement design procedures should be challenged requires an understanding of where the conservatism lies and applying judgement over the level of effort required to assess and reduce, and research or knowledge of published simplified methods to assist in clarifying, reducing and optimising design procedures with minimal effort. Well documented design methodologies and calculations assist the necessary peer review compliance process. A trade-off is present when undertaking more advanced or special studies to optimise further and, in some cases, may present diminishing returns.

Outlook: As routine investigation and simple state-of-practice analysis methods become more automated, analysis costs will be reduced further, and more advanced dynamic numerical methods are likely to become even less competitive unless they demonstrate significant cost savings in construction may be achieved with low risk to the parties involved. It is suggested that this will be best advanced through procurement models other than D&C but will need to be considered on a case-by-case basis.

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