Proceedings of the 11th ANZ Young Geotechnical Professionals Conference – 11YGPC
Queenstown, New Zealand
25 – 28 October 2016

Timber Piles for Ground Improvement in Fiji's Liquefiable Soil

A. Pereira[1] and I. McPherson[2]

[1]Geotechnical Engineer, MSc in Geotechnics, www.linkedin.com/in/anacatarinapereira, email: acatarina@gmail.com
[2]Aurecon New Zealand Ltd, PO Box 1061, Christchurch, New Zealand, PH (64) 21 783 194, email:Ian.McPherson@aurecongroup.com

ABSTRACT

This paper presents the design of the ground improvement for the abutments of four bridges in Fiji, where the earthquake induced liquefaction and cyclic softening could cause slope instability. A geotechnical investigation with boreholes and Cone Penetration Testing (CPT) was undertaken to define the ground model at the eight abutments. Liquefiable sand was identified at four abutments and very soft clay up to 50m+ depth was logged under all of the considered bridge abutments. The available machinery and material supplies dictated using driven timber piles as a solution for ground improvement, as the crane on site for piling the foundations could also drive the timber piles, and treated timber piles could be easily imported from New Zealand. The slope stability analysis comprised four load cases for the eight abutments: static, earthquake Serviceability Limit State, earthquake Ultimate Limit State and Maximum Credible Earthquake (MCE) with the soil properties (drained and undrained) modelled according to the load case. The ground improvement design was based on the timber piles densifying potentially liquefiable sand and strengthening the soft clays. Swedish screw testing was used as a cost effective method to assess the post treatment densification within the sand layers as neither Standard Penetration Testing (SPT) nor CPT were readily available in Fiji. The required densification was achieved within the costs and timeframes required for the project.

Keywords: liquefaction, ground improvement, timber piles

1 INTRODUCTION

This paper presents the design of the ground improvement for the abutments of four bridges in Fiji Islands. Vunivaivai and Vunidilo bridges were near to Suva, Lomowai Bridge was near to Nadi, both in Viti Levu Island and Cogeloa Bridge was near to Labasa in Vanua Levu Island (refer to Figure 1).

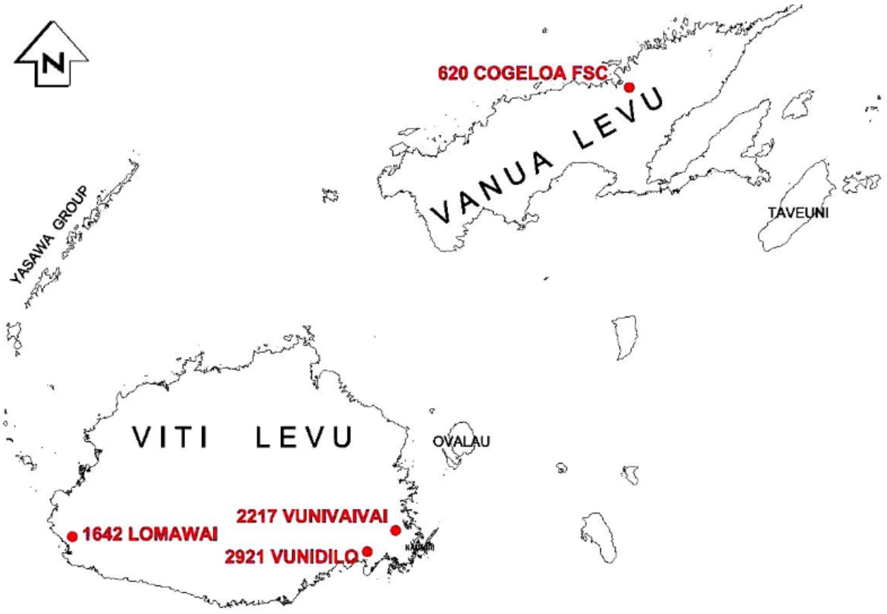


Figure 1. Bridges location plan, Fiji

There were old bridges at the same location as the new bridges in very poor condition (see Figure 2), requiring replacement. These bridges are important to the local community and economy as they are the only way to cross large rivers in the vicinity to connect villages, access schools and markets.

(a) *(b)*

Figure 2. View of the old bridges: (a) Vunivaivai bridge and (b) Cogeloa Bridge.

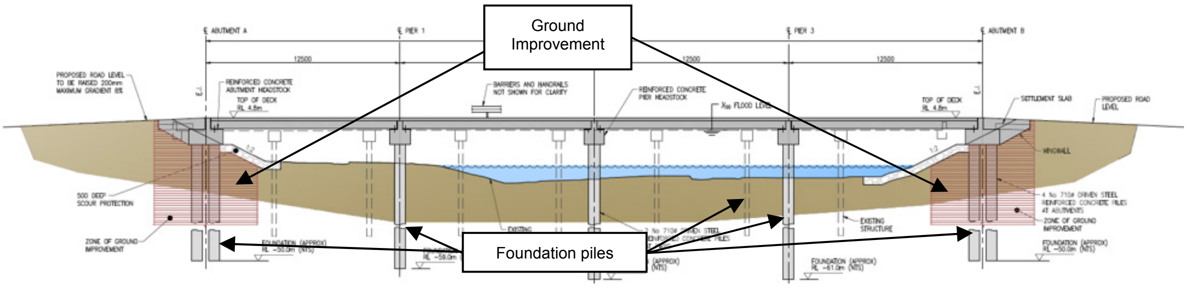


Figure 3. Long section of the new Vunivaivai Bridge

Lomowai and Cogeloa bridges also serve the sugar industry and have tracks for sugar cane trains. The sugar railway is crucial for the local economy as trains ship most of the sugar and this is the most important export item for the Fiji's economy.

The Fijian government let a design build tender for the four bridges which was won by Fletcher Construction. The design was based on the New Zealand and Australian Standards.

The construction program considered the need of a temporary pedestrian bridge to allow access to schools and markets, and the timing when the railway was needed for the sugar crop.

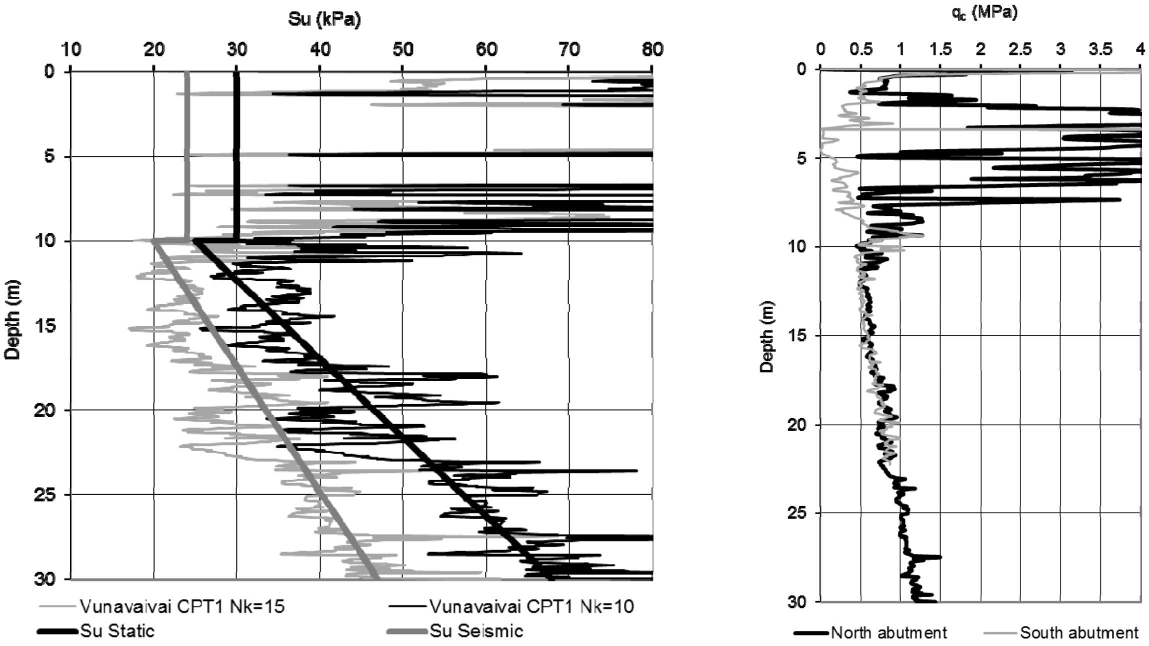
The design of the bridges started with an initial ground investigation with CPT and boreholes, and further shallow investigation with hand augers and shear vane testing. The geotechnical design of each bridge included the stability analysis of the abutments, ground improvement within the abutment and analysis of the foundation piles (see Figure 3). This paper will focus in particular on the ground improvement required for the abutments stability considering the particular conditions and available local Fijian resources.

2 GROUND CONDITIONS AND SEISMIC HAZARD ASSESSMENT

The geotechnical design of the four bridges in Fiji followed the traditional geotechnical design approach with an initial deep investigation comprising, three to four boreholes with SPT and two Cone Penetrometer Testing in each bridge location. Later in the project additional shallow investigation was undertaken with hand augers boreholes and shear vane testing at each bridge abutment.

The investigations indicated that the shallow ground condition at all bridges were similar with alluvial layers of interbedded soft silty clay and loose sand which required particular care for the performance of the abutment either in terms of bearing capacity or stability for static and seismic conditions. These soft and loose soils extended to:

- 55m in Vunivaivai, where no dense soils or rock was found to the depth investigated (55m);
- 18m bgl, underlain by weathered sandstone in Vunidilo
- 10m bgl underlain by weathered basalt in Lomowai
- 15m bgl underlain by weathered siltstone in Cogeloa



(a) (b)

Figure 4. Vunivaivai Bridge (a) Adopted static and seismic shear strength profile; (b) CPT profile

2.1 Soil properties

Cohesive soil strengths have been assessed primarily from the CPT logs and the shear vane results from the 4m deep hand auger bore holes (Figure 4). Typically soil undrained shear strength is assessed based on the following: $S_u = (q_c - \sigma_v)/N_{kt}$, where σ_v is the total effective stress and N_{kt} is the cone factor (Robertson, 2012). Based on a comparison of undrained shear strengths from the hand auger investigations with the undrained shear strength inferred from the q_c profile, we have taken a N_{kt} value of 10 for the design of bridge foundations and ground improvement at the bridge abutments for the static case.

Cohesive soils are not expected to liquefy due to their plasticity but the laboratory test information indicates that the soil water content is close to the liquid limit, therefore it could incur cyclic mobility in the cohesive soils with a resulting loss of strength.

The residual shear strength was determined based on the hand auger logs, particularly the residual vane shear strengths, and have been taken as lower bound strengths. The adopted static and seismic strength profiles with depth are presented in Figure 4.

Sand was modelled with a friction angle of 35° based on the CPT logs for the static case. For the seismic case, it was assumed that the sand liquefies with an average residual undrained strength of 5kPa based on the SPT blow counting in the logs and the method of Idriss and Boulanger (2008).

Given the presence of loose sand layers at shallow depth and shallow water table the liquefaction risk and subsequent lateral spreading and displacements were an issue addressed in the geotechnical design. These displacements could be significant and therefore affect the global bridge stability.

The slope stability analysis of the abutments was undertaken for all eight abutments to better understand the extent of the potential liquefaction induced ground damage, along with the loss of shear strength of the soft soil under seismic conditions. This paper will focus upon liquefaction and lateral spreading mitigation. Since the liquefaction mitigation design approach was similar for all four bridges, this paper only details the analysis completed for the North abutment of the Vunivaivai Bridge.

2.2 Seismicity of Fiji

Fiji lies in a complex tectonic setting along the boundary between the Australian Plate and the Pacific Plate. Southwards from Fiji the Pacific Plate is subducting beneath the Australian Plate along the

Tonga Trench forming the Tonga Ridge island arc system and the Lau Basin back-arc basin. To the southwest of Fiji, the Australian Plate is subducting beneath the Pacific Plate forming the Vanuatu Ridge island arc system and the North Fiji back-arc basin.

For this project the seismic hazard was considered for the ground improvement design, and seismic loading has been calculated in accordance with NZS1170.5, New Zealand Transport Agency Bridge Manual (NZTABM) and Fiji Roads Authority (FRA) standard using the following parameters: Class D site; Seismic hazard factor (Z) 0.23; Ru (SLS 1/50 year return period) 0.35; Ru (ULS 1/1000 year return period) 1.30; Soil factor (Class D or E) 1.12; PGA (SLS) 0.09g; PGA (ULS) 0.33g.

3 DESIGN ASSUMPTIONS

The geotechnical design of the four bridges comprised the design of the foundation system, ground improvement and engineered fill on top of the ground improvement.

The pile analysis / design considered a number of load cases including, traffic load, flooding with scour, seismic load, with both the lateral and vertical capacity of the piles checked for geotechnical and structural issues. Given the specific topography, ground conditions, flood conditions, road and rail traffic of each bridge, the piles design was carried out for each bridge and slightly different solutions were chosen.

The ground improvement design included liquefaction, lateral spreading and slope stability assessment, which indicate the need of ground improvement for the bridges abutments stability during a major seismic event. The ground improvement design itself followed an empirical method, described in the subsequent items, and proof tests were undertaken to verify the design assumption.

3.1 Slope stability analysis

The slope stability analysis undertaken for the Vunivaivai Bridge comprised a static load case and three seismic load cases: serviceability limit state (SLS), ultimate limit state (ULS) and Maximum Credible Earthquake (MCE). The flooding load case was also modelled for assessment of scour but without seismic load, as flooding and earthquake cases are unlikely to happen simultaneously.

The North abutment of Vunivaivai Bridge had the thickest sand layer of the eight abutments, therefore it is likely to be the most vulnerable to liquefaction of the eight abutments, and will be the example used in this paper. The ground profile was defined based on the geotechnical investigation and indicates approximately a 5m thick liquefiable sand layer (Figure 5).

The failure surface geometry can have a significant effect on the calculated factor of safety against failure particularly for seismic analyses where failure surfaces are much deeper than the static case.

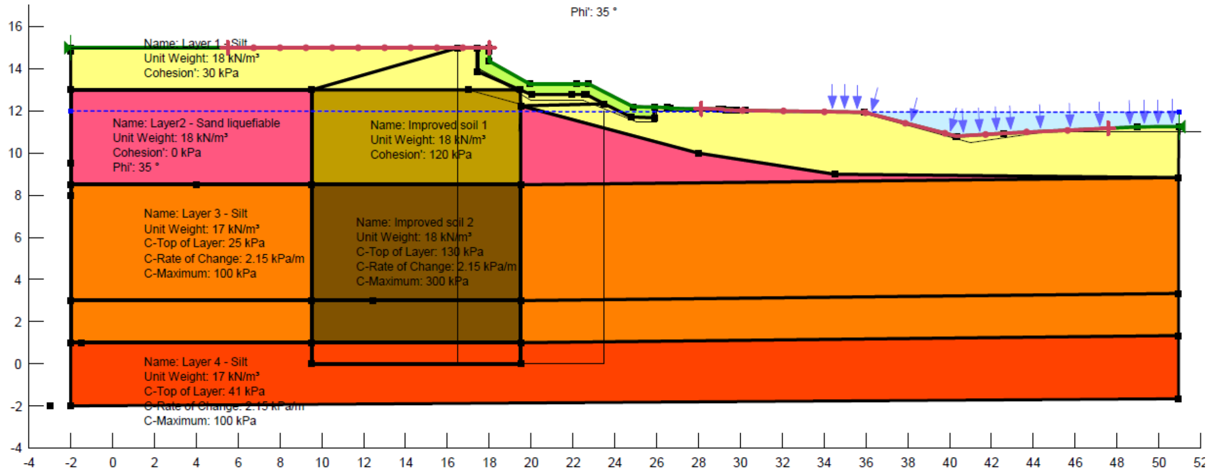


Figure 5. Geotechnical ground model for slope stability analysis (reference: SlopeW Software)

The failure surface was assumed to be constrained to 12m depth, which is considered to be the deepest credible surface. This assumption is based on Idriss and Boulanger (2008) who recommend limiting a lateral spread analysis to 2H depth where H is the height of the free face.

The analysis indicated that the slope would not be stable under a SLS earthquake therefore the following step was to find the appropriate ground improvement for the local ground conditions and local ready available materials and machinery.

3.2 Ground densification for liquefaction mitigation

Liquefaction occurs when loose sand is subject to vibration/ shaking, as it happens during an earthquake. Therefore, densifying loose sand would mitigate the liquefaction effects. The selected method to densify the loose and liquefiable sand was to drive piles into the ground, as replacing volume of soil and vibration during the driving piles process would densify the sand layers, and subsequently would increase the factor of safety against liquefaction. The effect of densification and stiffening on the soil liquefaction resistance was assessed based on the methods outlined by Baez and Martin (1993) and Rayamajhi et al (2012), where it is provided densification targets (N1(60)) according to the ground conditions and Stress Ratio to supress liquefaction. For this particular case the method indicates that a N1(60)=22 would supress liquefaction for a replacement ratio of approximately 8%. The design relied primarily on ground densification as the mechanism for ground improvement as the latest information indicates that relatively stiff inclusions such as timber piles do not increase significantly the liquefaction resistance.

As result of this analysis it was proposed to reinforce each abutment with 300mm SED timber piles at 1.2m centres. The average diameter of the piles was approximately 350mm at the depth of the sand layer therefore the replacement ratio is approximately 8%.

3.3 Ground improvement verification

During design, assumptions were made on the densification target using empirical methods and applying them to the local ground conditions. As the design would need confirmation of the assumptions made, post piles installation tests were undertaken. The verification tests were to demonstrate proof that the driven piles had sufficiently densified the soil to supress the liquefaction potential. Initially SPT was the preferred proof test because could be directly compared to the required target SPT-N value derived from literature (N1(60)=22) , however a SPT rig was not available in the islands when needed, hence the effectiveness of driven piles in densifying the ground was assessed using the Swedish Screw Test and use a correlation to N60 (SPT N-value corrected for field procedures and apparatus). The study undertaken by Habibi et al. (2006) presented a correlation from SWT to N60 that worked well for this project.

Swedish tests were undertaken prior to and post pile installation as this would allow a comparison pre installation between SPT/CPT and Swedish tests working as calibration. The tests indicated that a significant level of densification of the sand was achieved by the installation of the timber piles and largely exceeded the required N(60)=22. Figure 6 presents the results of tests in two locations pre and post installation of the timber piles. And indicates that a significant densification was achieved within the sand layer in the top 4m exceeding the target.

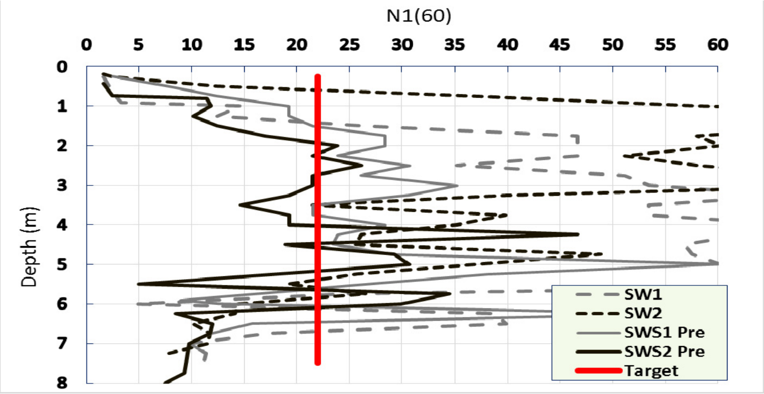


Figure 6. Swedish weight sounds for the North abutment, based on Habibi (2006)

4 CONCLUSIONS

The design of the ground improvement for the four bridges considered in Fiji was a challenge given the ground conditions and limited availability of heavy plant for pile driving and ground improvement.

The design had to comply the New Zealand and Australian Standards and at the same time be cost-effective accounting with the resources available in Fiji such as materials and machinery. The preferred type of inclusions in the ground was driven elements. Timber was known as available in the islands or easily shipped from New Zealand, therefore would be a cost-effective and sustainable solution. The technical solution of densifying the loose sand as a liquefaction mitigation method was considered to be the most appropriate for the local ground conditions. The inclusions in the ground would also strengthen the soft clays providing extra shear capacity for the slope stability.

The proof testing required to assess the level of densification induced by the timber inclusions was another challenge for the project as the usual testing equipment (CPT and SPT) was not available, therefore Swedish Screw testing was considered for that purpose and appropriate correlations had to be used and calibrated to the local ground conditions.

The success of the design and following construction stages of the new bridges was based on collaboration between design team, client and contractor to achieve a common goal of building new bridges in Fiji under the Australian and New Zealand design standards.

5 ACKNOWLEDGEMENTS

The authors would like to acknowledge and thank FRA, MWH and Fletcher Constructions Pacific for allowing them to present the above case study into the liquefaction mitigation measures applied at the four long bridges in Fiji replacement.

REFERENCES

Ambraseys, N. and Menu J. (1988). “Earthquake-induced ground displacement”. *Earthquake Engineering and Structural Dynamics,* 16, pp. 985-1006.

Ambraseys, N. and Srbulov, M. (1995). “Earthquake induced displacement of slopes”. *Soil Dynamics and Earthquake Engineering*, 14, pp. 59-71.

AS2159:2009. *Australian Standard Piling – Design and installation*. Standards Australia, Sydney, NSW, Australia.

Baez, J.I. and Martin, G. (1993) “Advances in design of vibro Systems for the improvement of liquefaction resistence” *Proceedings of the Symposium on Ground Improvement*, Canadian Geotechnical Society, Vancouver.

Habibi, M., Cheshomi, A. and Fakher, A. (2006), “A case study of liquefaction assessment using swedish weight sounding”*, 4th International Conference on Earthquake Engineering*, Taipei, Taiwan, October 12-13, 2006.

Idriss, I.M. and Boulanger R.W. (2008). *Soil Liquefaction during Earthquakes*. EERI Monograph Series MNO-12

Jibson, R. W. (2007). “Regression models for estimating coseismic landslide displacement.” *Engineering Geology*, 91, pp. 209–218

NZ Transport Agency (2013). *Bridge manual SP/M/022*

NZS 1170.5:2004. *Structural Design Actions Part 5: Earthquake actions – New Zealand*, Wellington, New Zealand: Standards New Zealand

NZS1170.0:2004. *Australia/New Zealand Standard, Structural Design Actions, Part 0: General Principles*. Standards New Zealand, Wellington, New Zealand.

Rayamadhi, D. et al. (2012). “Effect of discrete columns on shear stress distribution in liquefiable soil”, *Proceedings of GeoCongress 2012*, Oakland, California, March 25-29, 2012

Robertson, P.K, (2012). *Guide to Cone Penetration testing*, Gregg Drilling & Testing, Inc., California, USA