6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch, New Zealand

Geotechnical Performance and Analysis of ANZAC Bridge Following the 2010/2011 Canterbury Earthquakes

M.F.S. Lazzaro[1], I.J. Froggatt[2], P.T. Aynsley[3]

ABSTRACT

A significant number of the bridges along the Heathcote and Avon Rivers in Christchurch suffered liquefaction induced lateral spread damage following the 2010/2011 Canterbury earthquake sequence including the three span four lane Anzac Drive Bridge which crosses the Avon River approximately 4km upstream of the Heathcote-Avon Estuary. The banks of the Avon River at Anzac Bridge laterally spread during the February 2011 earthquake causing abutment rotation. Following an assessment of the structure it was determined that the piers would be retained and the decking and abutments replaced. A detailed geotechnical analysis was completed for the abutment design which included assessing liquefaction potential of the subsoil, determining the zone of lateral spread and providing lateral and vertical pile design parameters. One of the key aspects of the analysis was determining the depth of liquefaction with consideration of the soil's density and composition and the performance of the bridge.

Introduction

The investigations, assessment and analysis detailed in this paper were completed during the authors' secondment to the Stronger Christchurch Infrastructure Rebuild Team (SCIRT).

SCIRT was established to rebuild government owned infrastructure following the devastating February 2011 Christchurch earthquake. At its peak, the SCIRT professional services team was made up of around 200 designers and scientists from a number of local and global consultancy firms including Aurecon and GHD. SCIRT engineers were tasked with assessing and designing repair and rebuild works for the Christchurch City Council (CCC) and New Zealand Transport Agency (NZTA) owned bridges. The vast majority of the bridges were located along the Avon and Heathcote River. The focus of this paper is the NZTA owned Anzac Drive Bridge.

The work completed included developing a site geological model, assessing the depth and severity of liquefaction and lateral spread and determining the required depth and strength of the piles to resist future lateral loads caused by laterally spreading ground.

Existing Structure

Anzac Drive Bridge was completed in 2000 and carries State Highway 74 (Anzac Drive) over the Avon River. The location is shown in Figure 1. The bridge width is approximately 25m and

[1]Senior Geotechnical Engineer, Aurecon, MIPENZ, Christchurch, NZ, marcus.lazzaro@aurecongroup.com
[2]Senior Geotechnical Engineer, Aurecon, MIPENZ, Christchurch, NZ, ian.froggatt@aurecongroup.com
[3]Geotechnical Engineer, Aurecon, MIPENZ, Christchurch, NZ, paul.aynsley@aurecongroup.com

includes four traffic lanes, a median barrier and a footpath on the western side of the deck. The deck comprises double hollow core units with a central span of 18.6m and end spans of 14.9m for a total length of about 50m.

The deck is supported by 15 vertically driven steel H piles at each abutment and 1.5m diameter steel cased concrete piles driven to 22m at the river piers. The concrete pier piles terminate at river bed level with rhomboid shape columns above supporting a 1200mm deep crossbeam.

The bridge continues to take traffic, with the northbound lane reduced to one lane to prevent two lanes of heavy vehicles queuing on the bridge to enter the roundabout.

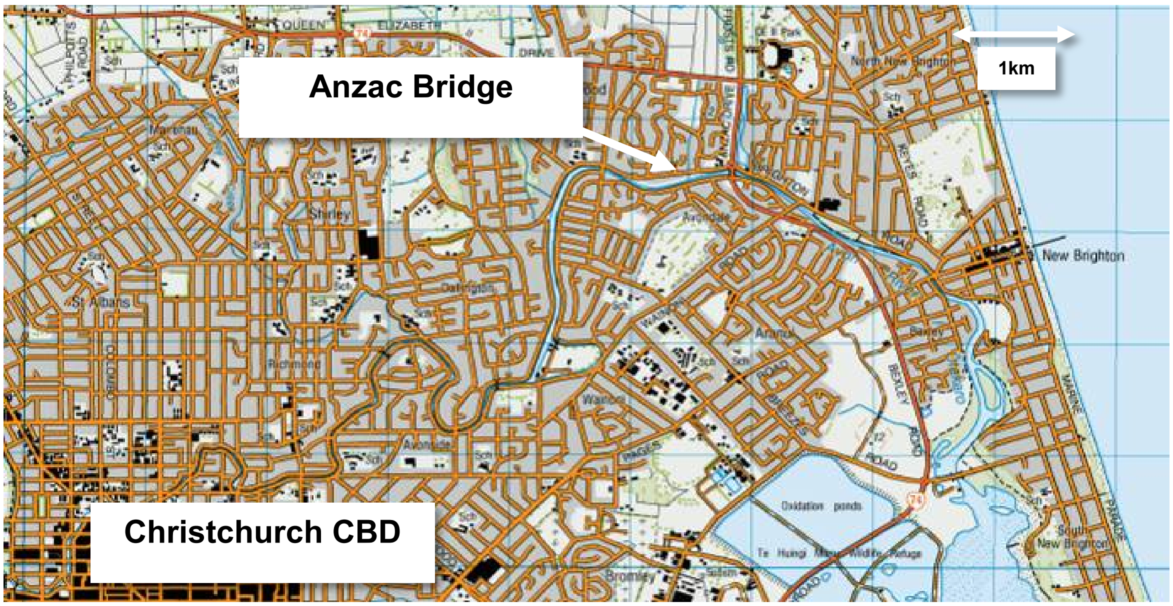

Figure 1 Location of Anzac Bridge (source: LINZ)

Figure 2 Anzac Bridge from bank (left) and aerial following Feb 2011 Earthquake (right)

Observed Damage

Land Damage

An understanding of land damage following the recent Canterbury earthquakes forms a vital part of understanding the performance of the ground, both during previous and future seismic events. Gaining an appreciation of the land damage at the Anzac bridge site has been achieved by completing a site inspection and reviewing the data held within the publically available Canterbury Geotechnical Database (CGD).

Numerous inspections of the bridge have been completed since the February 2011 earthquake, including a site visit by the authors in March 2014 in which the areas surrounding the bridge were inspected for evidence of land damage.

It is acknowledged that some of the evidence of lateral spread and liquefaction has been lost over the three years since the February 2011 event particularly in grassed and vegetated areas; however evidence of lateral spread was observed near the north abutment. The footpath either side of the north abutment includes sections damaged by long, open cracks parallel to the Avon River. Figure 3 shows some of the cracking inspected.

Figure 3 Footpath beside north abutment

The CGD is available to professional engineering companies involved with the Canterbury recovery. The database includes map layers which show the extent of observed liquefaction ejecta, lateral spreading damage and ground cracking. The mapping was completed shortly after each event and indicates widespread liquefaction and lateral spread damage (including ground cracking) in the surrounding area following the February 2011 event. Widespread surface manifestations of liquefaction can be seen in the aerial provided in Figure 2.

Land damage helps to understand the damage to the bridge and vice versa – the structural damage (i.e. abutment rotation) helps in the understanding of land performance. The following section provides a summary of the structural damage observed.

Structural Damage

Anzac Drive Bridge suffered only minor damage in the September 2010 earthquake but in the February 2011 there was extensive liquefaction and lateral spread damage. The following summarises the structural damage recorded during several site visits by SCIRT structural engineers in 2013 and 2014:

- The abutments and the steel piles (310UC137) supporting them have rotated towards the river by 4° to 6° as evident from measurements of the abutment, the abutment piles and the light poles connected above the abutments. The abutment 'hood' has significantly tilted up forming a large gap with the pedestrian underpass wall. Figure 4 shows the rotation of the south abutment from the west and east side of the bridge;
- The pedestrian underpasses beneath the bridge has displaced horizontally towards the river by between 300mm and 750mm.

Figure 4 Rotation of south abutment from west (left) and east (right)

Summary of Observed Damage

The damage sustained by the bridge appears to be predominantly a result of land damage. The evidence observed on site and recorded in the CGD indicates widespread liquefaction and lateral spreading along the banks of the Avon River. The movement of the banks towards the center of the river (lateral spread) has caused rotation of the abutments and movement of the piers inwards. Some of the damage may also have been caused by the intense shaking.

Ground Model

The ground conditions at the bridge needed to be well understood to prepare a rebuild or repair solution which will meet code requirements during the next large earthquake. The geotechnical investigation was undertaken over a number of phases and is summarised in the following section.

Geotechnical Investigations

Geotechnical data and information was collected and compiled from a number of sources, including a geotechnical investigation completed by Opus Consultants in 2012, SCIRT in 2014 and several nearby investigation points from within the CGD. In total 13 CPTs and four boreholes were used to develop the ground model for the bridge. The CPT testing was completed near the abutments and at each pier and extended to depths of between 9.2m and 35.7m below ground level (bgl). The four boreholes were advanced using sonic method to depths of between 15m and 43mbgl.

Subsurface Conditions

The surface geology map of Christchurch (Brown and Weeber 1992) indicates the site is located in an area of alluvial sand and silt overbank deposits of the Holocene Age Springston formation. Areas of dune and beach sand exist to the south and north east of the bridge.

The site specific ground model has been developed using geotechnical information from the investigations completed over the previous three years. The site is underlain by geotechnical units of variable thickness and properties and a geotechnical profile is provided in Table 1 and is shown in the Slope/W profile provided as Figure 5. The groundwater table lies at between 2m and 5mbgl at the top of the riverbanks.

Table 1 Generalised soil profile for Anzac Drive Bridge.

Unit	**Top of Unit Reduced Level (RL) (m)**	**Thickness of Unit (m)**	**Typical CPT Cone Resistance, q_c (MPa)**	**SPT N-Value Range**
1.Silt/Sand/Peat	-	0 to 4	0.3 to 2	1 to 10
2.Sand	6 to 8	7 to 12	5 to 15	10 to 40
3.Sand	-4 to -1	19 to 24	15 to 30	20 to 80
4.Sand/Gravel	-25 to -23	-	not penetrated	44 to 74

Liquefaction Assessment

CPT and SPT data was used to assess liquefaction susceptibility of the underlying soils. Seismic loading parameters were derived for use in the assessment.

Seismic Loading

The recent seismic activity around Christchurch was recorded by GNS National Seismic Network (GeoNet) seismographs. The bridge is located approximately 100m from the Hulverstone Drive Pumping Station (HPSC) strong motion seismograph. Data was recorded for each of the large seismic events and the resultant maximum horizontal accelerations are provided in the table below.

Using methods described in the NZTA Bridge Manual (NZTA 2013) Section 6.2, a Peak Ground Acceleration (PGA) and effective magnitude of 0.37g and M=6.1 was derived for a ULS earthquake for the Anzac Bridge Site.

Table 2 Anzac Drive Bridge site PGAs for ULS and key seismic events.

Earthquake Event	**Magnitude**	**PGA (g)**
4 September 2010	7.1	0.16
22 February 2011	6.2	0.25
13 June 2011	6.0	0.43
ULS Event	6.1	0.37

Liquefaction Assessment

The liquefaction assessment was undertaken using the Cliq software package (v1.7) and cross-checked using spreadsheet calculations developed by SCIRT. The method of Robertson and Wride (Robertson 1998) was used, with estimates of potential ground deformation based on the procedure presented by Ishihara and Yoshimine (Ishihara 1992).
The following observations are made from the results (refer to Table 1 for soil profile):

- Under ULS loading (magnitude 6.1 and PGA 0.37g) the Cyclic Stress Ratio (CSR) corrected with the Magnitude Scaling Factor (MSF) is typically between 0.15 and 0.25;
- For ULS conditions, which approximately equate to the 22 February 2011 event (see Table 2), liquefaction induced ground settlements are predicted to be between 50mm and 100mm;
- The assessment indicates that the loose to medium dense sands of Unit 2 are vulnerable to liquefaction;
- Thin zones of Unit 3 are predicted to liquefy.

Although contrary to the liquefaction assessment, the sands of Unit 3 are considered unlikely to liquefy under ULS conditions for the following reasons:

- Cyclic Resistance Ratio (CRR) charts presented by Idriss and Boulanger (Idriss 2008) indicate sand with 5% to 10% fines and a normalised corrected CPT tip resistance (q_{c1N}) in excess of about 150 are unlikely to liquefy for CSRs between 0.15 and 0.25. The corrected normalised tip resistance of Unit 3 typically exceeds 150 and particle sieving completed by Opus in 2012 indicates the soils of Unit 3 to comprise between 5% and 10% fines;
- The existing river piers and abutment piles are founded at approximately 22m below ground and do not appear to have settled following any of the recent seismic events. This observation is consistent with no significant liquefaction occurring below the toe level of the existing piles (~22 mbgl).
- The simplified procedures for liquefaction triggering are not verified for depths exceeding about 15m to 20m.

Lateral Spread Assessment

Significant damage, including rotation, occurred to both abutments following the February 2011 earthquake. The damage is consistent with lateral spread damage; which is supported by translated soil, ground cracking and rotation of the abutments. The Newmark Sliding Block method was used to estimate the expected horizontal displacement during future seismic shaking. The Newmark displacements are considered indices of slope movement rather than precise predictions of actual displacements. To complete a Newmark displacement estimate the following is required:

- Peak Ground Acceleration (a_{max}) –see Table 2;
- Yield, or critical, acceleration (a_c) – determined using the liquefied shear strengths (τ/σ) of the Unit 2 soils.

Limit equilibrium (Slope/W) was used to estimate the yield acceleration using a τ/σ of 0.15 (based on available charts and cone tip resistance) for the Unit 2 soils. The acceleration was increased until a FoS of 1 was achieved. A FoS of 1 indicates the minimum acceleration required to initiate downslope movement. A yield acceleration of 0.012g and 0.009g was required to obtain FoS=1 for the north and south abutment, respectively. Output from Slope/W is provided below as Figure 5.

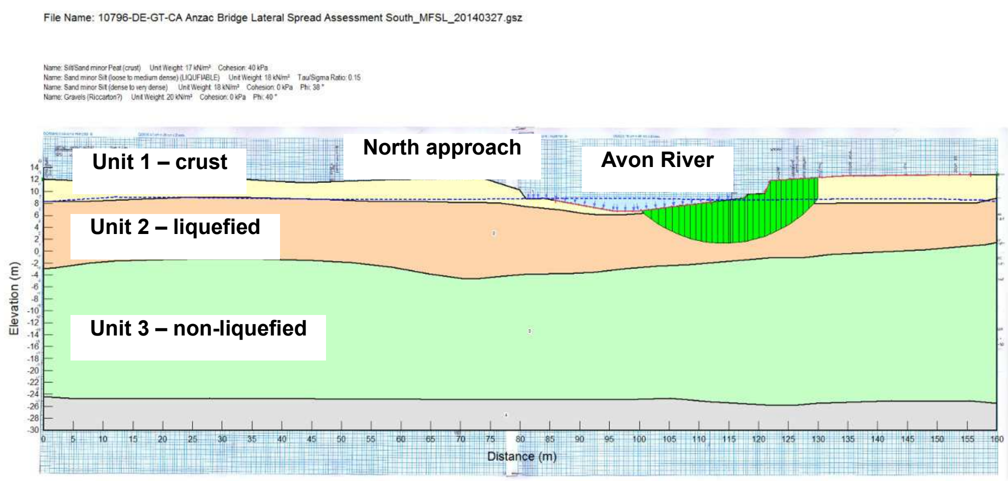

Figure 5 Slope/W output for south abutment lateral spread assessment

Using the Jibson developed Newmark equations and charts (Jibson 2007) lateral displacements were estimated for the February 2011 magnitude and acceleration and for ULS conditions. The results are summarised in the table below and show good results for predicted and observed movement during the February 2011 event. The authors acknowledge that the General Limit Equilibrium (GLE) and the Newmark sliding block methods are based on the assumption the soil behaves as a rigid, perfectly plastic material and have limited applicability for weakening soil failure mechanisms such as lateral spread. The methods adopted are simple and provide an indication/index of expected movement and deformation and appear to correlate well for recorded lateral displacements at the bridge.

Table 3 Estimated lateral displacement (using Jibson 2007) versus measured displacement.

Event	**North Abutment**	**South Abutment**
22 Feb 2011 – measure displacement (mm)	200 to 300	500 to 750
22 Feb 2011 – predicted displacement (mm)	300	400
ULS event – predicted displacement (mm)	1000	1500

Pile Assessment and Proposed Remediation

The steel piles at each abutment are sufficiently damaged (yielded) and require replacement. As part of the remediation works two new piles will be provided under each abutment beam and have been designed to withstand the lateral loads imposed by the spreading crust. The piles will penetrate the liquefiable Unit 2 sands and find partial fixity within the dense sands of Unit 3.

The lateral demand on the piles can be evaluated by either the displacement or limiting pressure method. The limit pressure method assumed sufficient displacement occurs to fully mobilise lateral earth pressures. For the displacement method a displacement is applied to the pile at the level of the crust with the soil modelled using soil spring constants. Although the lateral spread assessment indicates up to 1500mm of lateral displacement, 500mm of displacement is adopted for the Anzac Bridge abutments for the following reasons:

- The MBIE guidance document (MBIE 2012) recommends that for the given bridge location lateral movement of between 300mm and 500mm should be adopted for design. This is perhaps too low but does provide general guidance for the expected deformation.
- The measured displacement (between 200mm and 750mm) provides a good indication of the expected movement following a large earthquake event (such as February 2011) and should be considered a reasonable value for design.

Structural analysis was completed using Microstran with lateral spread displacements applied to the pile over the thickness of the non-liquefied crust and degraded springs installed for the liquefied soils (Unit 2). The piles are designed based on the structural demands (i.e. bending moment and shear force) caused by 500mm lateral movement of the crust and the movement associated with the moving liquefied mass of Unit 2. Essentially the crust moves as a rigid body on top of the liquefied Unit 2 soils and the base non-liquefied layer (Unit 3) is fixed. The movement of the pile in the liquefied layer ranges from zero at the interface with Unit 3 to a maximum at the top of the liquefied layer – 500mm.

Conclusions

The abutments of Anzac Drive Bridge have suffered rotational damage and are due for replacement. The damage to the abutments and supporting piles is predominantly a result of liquefaction induced lateral spreading which was widespread across the surrounding areas.

A detailed geotechnical investigation and subsequent liquefaction assessment indicate the soils to be loose to medium dense sands and potentially liquefiable to a depth of between 10 m and 16 m below ground level. This soil layer is in turn underlain by a thick deposit of dense to very dense sands with occasional lenses of silt. The new piles will be installed into the underlying, dense Unit 3 soils to achieve partial fixity during a lateral spread event. The piles have been designed to withstand the loads imposed by movement (lateral spread) of the non-liquefied crust.

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