Replacement Of Four Long Bridges In Fiji Islands

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Abstract: Four bridges in the Fiji Islands were recently replaced using a Design and Construction procurement method for the Fiji Road Authority. The bridges provided vehicle, pedestrian and Fiji Sugar Corporation cane trains over waterways subjected to significant flood events. Design of the replacement bridges was specified to Fiji and New Zealand standards for a 100 year design life. This high specification required additional concrete cover and high strength concrete mixes with supplementary cementitious materials and corrosion inhibitors to ensure durability. Ground conditions at the sites were generally poor and one site had no rock proven in investigations to 65m below ground level. Design requirements to NZ standards required stringent seismic performance criteria to be met. To meet these, the design used ground improvements at the abutments to control abutment stability and movements and the bridge structures were detailed for a ductile seismic response. Standardisation of the replacement bridges was developed into the designs to achieve efficiencies through repetition. Construction of the bridges required innovation to build these in remote locations and within the limitations of supply and construction equipment on island.

Keywords: Bridge replacement, Detailed design

1. Introduction

Four bridges in the villages of Vunidilo, Vunivaivai, Lomawai, located on Viti Levu and Cogeloa located on Vanua Levu in Fiji, see Figure 1 below, have been recently replaced for the Fiji Roads Authority (FRA). The bridges were replaced using a Design & Contract procurement method.

The bridges provided access for vehicles, pedestrians and Fiji Sugar Corporation (FSC) cane trains over waterways. The bridges were located on local road networks with traffic volumes of approximately 100 vehicles per day, however, where important for routes for FSC trains were these were carried by the bridges. The existing bridges were multiple span structures with total lengths of between 30 to 60m long and typically constructed using short I-girder spans with a timber deck. The waterways where in most cases prone to significant flooding and over topping of the bridge decks.

Geotechnical ground conditions at the sites where typically poor with weak materials overlying rock at depth. At the Vunivaivai site, no rock was found in the geotechnical investigations to depths of 65m below ground level with undrained shear strengths of less than 10kPa to 50m depth.

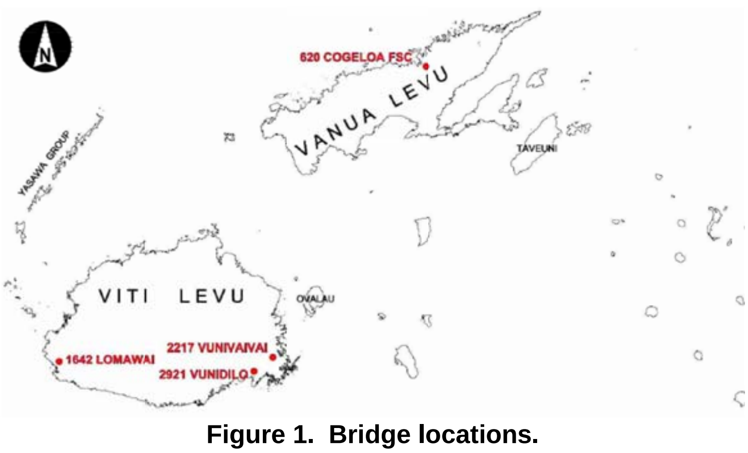

Figure 1. Bridge locations.

Figure 2. Existing Vunidilo bridge.

2. Concept replacement bridge structures

In tender conceptual design, a standardised design solution was developed to achieve efficiencies in both design and construction of the bridges through repetition. Contract requirements specified the existing bridges to be replaced online, with the replacement structures maintaining the existing horizontal and vertical road/rail alignment. During construction the bridges were typically closed to allow online replacement with an alternative detour route provided. At Vunidilo, where no alternative route was available, vehicle access was provided on the temporary construction staging with temporary traffic management.

Figure 3 shows the general elevation for a typical replacement bridge, the Cogeloa replacement bridge is shown. The replacement bridge abutments and intermediate pier supports where spaced around the existing bridge foundations whilst maintaining the existing bridge length.

Concept design of the replacement bridge structures is discussed below.

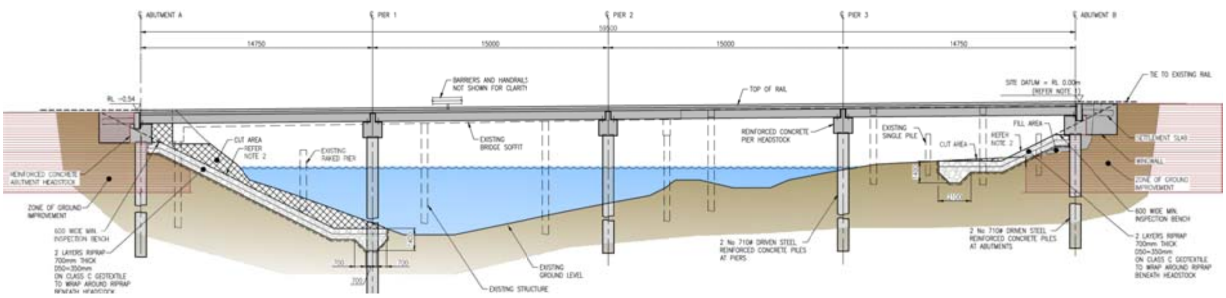

Figure 3. Cogeloa replacement bridge general arrangement.

2.1 Superstructure

Bridge configurations for the replacement structures required spans between 12.5 and 15.0m long. Prestressed hollowcore girders or similar would typically be suitable for these spans ranges, however, these girders were too heavy for the limited crane capacity in Fiji. Smaller, British section T beams where selected due to their reduced weight and a mould being available for the project.

A standardised bridge superstructure design using T beams was developed that was suitable for all the bridge structures including those supporting FSC rail loading. The deck, see Figure 4, was constructed using 13 no. precast prestressed concrete inverted 535mm deep T beams. This provided a total deck width of 7600mm with a 3500mm vehicle/rail lane, 600mm shoulders and a 1700mm pedestrian lane.

A cast in-situ reinforced diaphragm and 170mm deep topping slab form the deck surface and structurally connect the beams together. The deck slab cantilevers 450mm on each side of the structure to provide the overall deck width. The T beams were typically designed as simply supported for ease of construction, however, the deck slab was continuous over the piers with the use of a link slab.

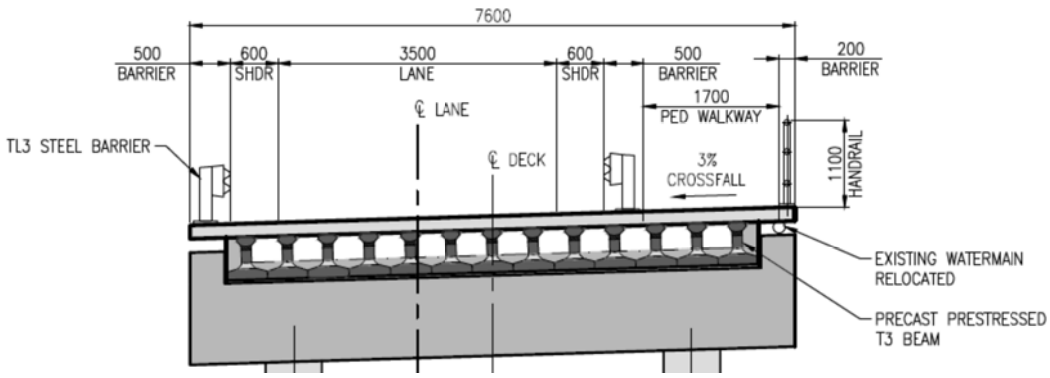

Figure 4. Typical deck cross section.

2.2 Substructure

A standardised substructure was developed for the bridges with reinforced concrete headstocks supported on 2 no. piled foundations at the abutment and intermediate pier supports. This configuration provided favorable portal frame action for transverse seismic and flood loading whilst being simple to construct.

The abutments were spill though slopes with 1V:2H batter slopes and rip rap rock scour protection.

2.2 Foundations

The bridges were supported on piled foundations that where typically constructed using bottom driven closed steel tubes with the exception of the Lomawai Bridge. At the Lomawai Bridge where a shallow Basalt rock layer was encountered, a bored pile with a rock anchor was used.

Piles were typically 710mm diameter tubes to keep piling equipment within the capacities available in Fiji. The steel casing was used as permanent formwork for ease of pile construction over water. The piles were designed as non-composite sections with a reinforced concrete in-fill in the upper portions of the pile where design actions were high. In the lower portions of the pile, where moment and shear demands were insignificant an un-reinforced concrete in-fill was provided.

Ground improvements were provided at abutments to control seismic induced movements. For ease of construction, driven 300mm SED timber poles were proposed.

2.3 Articulation

Semi-integral design was typically adopted to eliminate the need for bridge expansion joints and the inspection and maintenance requirements associated with these. At the abutments and piers, the bridges were fixed for transverse and longitudinal translational movements with use of concrete shear keys. Rotation capacity was provided using plain un-reinforced elastomeric bearing strips which do not require replacement.

3. Project requirements and design criteria

The requirements for the project where stipulated in the Employer Requirements and the *FRA Design Standard – Bridge, Jetty and Culvert Structures*. The FRA design standard extensively references the NZTA Bridge Manual Edition 3 and other relevant NZ Standards with amendments for durability requirements and FSC train loading. The structures were designed as Importance Level 2 structures.

3.1 Durability requirements

A 100 year design life was specified for this project which is understood to be the first time such a high design life has been requested in Fiji. Local concrete suppliers were not accustomed to producing the high specification concrete with high concrete strengths and supplementary cementitious materials required to ensure durability.

For concrete structures, the FRA design standard adopts the approach within NZS3101, however, increases the assessed exposure classification by a further category, e.g. B2 to C to account for Fiji conditions of prolonged high temperatures and humidity. Where the class was assessed as C, a new category C+ was prescribed. Table 1 below gives the concrete exposure classifications for exposed surfaces at the various sites. Due to the existing vertical alignment being maintained in the replacement bridge designs, the headstock was partially submerged in brackish water in a MHWS tide at the Vunidilo site. The site was tidal and approximately 4km upstream from the Suva harbour.

Table 1. Site concrete exposure classification

Bridge	Vunidilo	Vunivaivai	Lomawai	Cogeloa
Site exposure classification	B2/C+	B2	C	B2

For C+ exposure classification, a concrete mix with 30% fly ash type C as a supplementary cementitious material to meet the requirements of Cl 3.7 NZS3101 was selected. In addition to the supplementary cementitious material, BASF Masterlife 2006 corrosion inhibitor was added to the mix to meet the FRA requirements for a C+ category. Where the exposure classification was C, 30% fly ash type C was added only. Fly ash was selected due to the reduced cover requirement of 50mm compared to 60mm required for micro silica as noted in the draft amendments dated October 2014 to Table 3.7 of NZS3101. This minimised the need for modification of the T beam mould to meet cover requirements.

3.2 Seismic design and seismic induced movements

Seismic design requirements where to NZTABM Edition 3 requirements for a ULS 1/1000 design level event. The NZTABM gives stringent seismic performance criteria for the level of damage acceptable for various design level intensities, including ability of the structure to be used immediately for emergency traffic and being able to be fully reinstated to all design level actions after a design level ULS event.

Seismic loading was derived using Hazard factor, Z from FRA design standard and the site sub soil classifications given in Table 2 below.

Seismic induced movements due to this medium level seismic hazard with poor ground conditions were significant and required ground improvements to ensure abutment slope stability and control movements. With ground improvements provided, seismic induced movements exceeded the limits within NZTABM for zero displacement permanent displacement in regions Z<0.3 at the ULS design level event. A departure was accepted to allow permanent displacement of 100mm at ULS and 200mm at MCE. This level of movement was considered acceptable and met the seismic performance criteria of the NZTABM.

Table 2. Seismic design criteria

Bridge	Vunidilo	Vunivaivai	Lomawai	Cogeloa
Hazard factor, Z	0.23	0.23	0.23	0.28
Site subsoil class	C	D	B	C
Structural performance factor, S_p	0.80	0.70	0.90	0.80

3.3 FSC train loading

FSC train loading was specified in the FRA design standard as a HN loading case with the axle loading and spacing reproduced below. The uniformly distributed HN load was not applied. Impact loading was applied with a dynamic factor of 1.3.

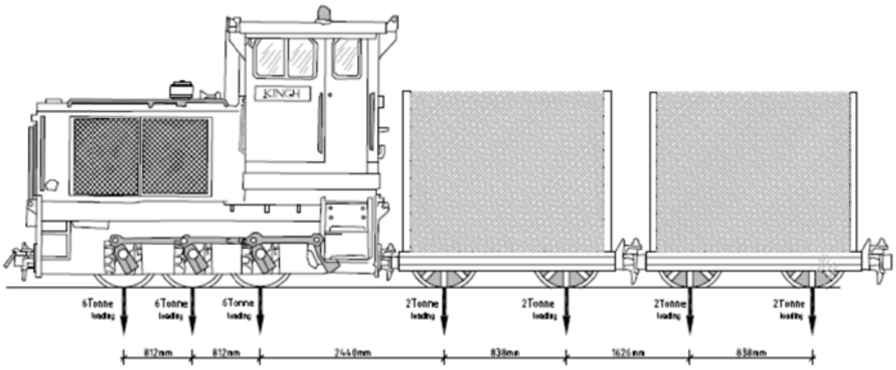

Figure 5. FSC trail loading (from FRA design standard).

4. Detailed design

The design was standardised across the bridges with the development of standard drawings to document a majority of the bridge detailing including; precast beams, deck concrete and reinforcement and secondary elements e.g. barriers, handrails and rail fixings. Specific drawings where used for each bridge to document the general arrangement, pile details and set-out.

Design methodology for key aspects of the design are discussed below.

4.1 Live load analysis

Grillage analysis was used to determine the load distribution and design actions in the bridge deck. The analysis followed the guidance within BCA/CIRIA Recommendations on the Use of Grillage Analysis for Slab and Pseudo-slab Bridges Decks by R. West.

Longitudinal grillage members were used to represent the stiffness of the precast beams and composite in-situ deck. These members were given the torsional stiffness of the enclosed box section formed by adjacent precast beams and in-situ top slab and diaphragm. Transverse grillage members were used to represent the stiffness of the in-situ top slab and participating diaphragm. These members were given an effective torsional stiffness accounting for the idealised box beam with open lattice sides. The vertical stiffness of the elastomeric bearing supports was included in the analysis.

Effective section properties to account for cracking where adopted in the analysis with; 0.1 x gross inertia for prestressed and reinforced concrete members in torsion and 0.5 x gross inertia for reinforced concrete sections in flexure.

4.2 Seismic design

4.2.1 Seismic design philosophy

The bridge structures were designed in accordance with NZTABM forced-base seismic design requirements as dynamically responding structures. Resistance for seismic inertia loading in the longitudinal direction was provided by flexure in the abutment and pier pile/columns and passive soil resistance at the leading abutment. Transverse seismic inertia loading was resisted by diaphragm action in the deck and portal frame action in the abutment and pier pile/columns. Seismic inertia loads in the deck where transferred into the substructure by longitudinal and transverse shear keys provided at the abutment and pier locations.

A ductile response was adopted in the design and capacity design was used to protect non-ductile elements from overstrength actions. Potential plastic hinge zones in the piles were detailed for a ductility $\mu = 3.00$.

4.2.2 Seismic design cases

The ground conditions at the bridge sites were typically soft soils to tens of meters of depth. While the soils were generally not considered to be susceptible to liquefaction, cyclic mobility of the soils with a reduction in soil stiffness and strength was expected in the design level event. Guidance for design of bridge foundations for liquefaction and lateral spreading effects was taken from NZTA Research Report 553.
The following seismic design cases were considered in design:

- Cyclic analysis without cyclic softening,
- Cyclic softening analysis, considering simultaneous kinematic loads and structural inertial loads accounting for stiffness and strength degradation of the soil,
- Lateral spreading analysis without structural inertial loads.

Under cyclic softening the soil strength was been taken as 80% of the static undrained strength.

4.2.3 Seismic analysis

Modal analysis of the structure was used to determine the dynamic response of the bridge to seismic loading and estimate the structural inertia loads. Soil springs were reduced to account for strength degradation when determining inertia loading coinciding with cyclic softening. A structural damping coefficient of 5% was adopted in the analysis.

Pseudo-static analysis of the bridge structure was used to model the soil-pile interaction. This analysis was used to determine the inelastic response, if any, under movements of the abutment slopes. The soil stiffness was represented by a bi-linear soil spring and a moment-curvature relationship was used to model the plastic hinge response.

4.2.4 Seismic performance criteria

The bridge structures were designed to meet the seismic performance criteria given in the NZTABM. The performance of the bridge structures is summarised as;

- Minor level earthquake (SLS) – Bridges designed to respond elastically, no damage expected, available for immediate use post-earthquake,
- Design level earthquake (ULS) – Bridges designed to respond either elastically or inelastically with a limited ductile response of up to $\mu=2.00$, minor damage could be sustained in plastic hinge regions e.g. spalling of cover concrete in the pile sections immediately beneath the headstocks. Damage is however repairable to original strength. Available for emergency traffic immediately and full traffic loading after repair, if required,

- Major earthquake (MCE) – Bridges design to respond inelastically with a limited ductile response up to μ=3.00, however, bridge collapse is avoided.

4.3 Geotechnical design

Site investigations comprising drilling with SPT testing were completed prior to contract award and supplementary Cone Penetration Testing (CPT) and hand auger boreholes with shear vane testing were completed as part of the contract works. The investigation results indicated that surficial geology comprised alluvial deposits of soft to very soft silty clay and loose to medium dense sands. There was significant variation in stratigraphy and the depth to rock varied from 65m+ at Vunavaivai to less than 10m at Lomawai.

Because of the variable geology each abutment was assessed for liquefaction and lateral spreading of the sandier soils and cyclic softening and slope failure in the cohesive soils. The liquefaction assessment was carried out at each abutment where there was a significant thickness of sand logged in the borehole or CPT hole. The design accelerations were site specific and varied from 0.09g to 0.13g for SLS shaking and 0.33 to 0.48g at ULS shaking. Liquefaction was predicted to occur at various depths and layer thicknesses and so driven timber poles were selected as a cost effective and practicable solution to densify potentially liquefiable soils. The pole spacing was designed using the method of Baez and Martin (1993) which indicated a replacement ratio in the order of 8% was required to suppress liquefaction. The timber poles vary in diameter from about 300mm to 325mm at the small end to 400mm or more at the top end. The replacement ratio was based on the pole diameter at the depth of liquefiable layers.

The effectiveness of ground improvement was assessed using the Swedish Weight Testing (SWT) before and after pole installation. The SWT was selected as it was man handable which was very important on isolated site with limited access. The results indicated acceptable levels of densification had been achieved.
The initial stability analyses indicted that ground movement could occur in the softer cohesive soils during large earthquakes and that ground improvement was required to achieve the design factors of safety. The effect of the poles installed to suppress liquefaction to also improve stability was therefore assessed by increasing the soil strength based on the replacement ratio of the pole. As with the liquefaction assessment the combined strength of the pole and soil was based on the pole diameter at the depth of interest. The poles were also designed to extend sufficiently deep below the deepest credible failure surface to generate the full shear capacity of the pole with poles extending up to 17m below finished ground levels in one case.

All bridges expect Vunavaivai were founded into rock which had ample end bearing capacity for the magnitudes of the loads and pile diameters. However, at Vunavaivai there was at least 65m of soft soils and the piles were design to carry the loads in skin friction. Two pile load tests were carried out at Vunavaivai to provide more information on pile capacity and to allow the use a higher capacity reduction factor with the objective of reducing pile length. The testing indicated more favourable ground conditions than anticipated and that after a week the pile capacity could be four times higher than indicated by the Hiley Formula at the end of pile driving. After reviewing pile test results and assessing the risk, a pile set up factor of two was adopted (i.e. pile driving could stop once the Hiley Formula indicated 50% of the long term design capacity had been achieved).

4.4 Lomawai pile design

A unique pile design was developed for the bridge structure at Lomawai. Geological conditions at the site consisted of clayey, silty and sandy soils overlying a shallow Basalt rock layer between 8 to 13m below ground level. Given the shallow depth to rock and susceptibility of the overlying materials to scour, a driven pile solution was considered to have insufficient lateral resistance. An option for a bored pile with embedment into the rock layer to provide moment fixity was considered, however, not adopted due to concerns of difficultly in construction of a socket in Basalt rock.

A 1050mm diameter bored reinforced concrete pile solution with a small embedment into rock and a centrally located rock anchor was proposed and developed at detailed design. The concept for the design was to provide a "pinned based" connection the to the rock layer and rely on portal frame action in both the longitudinal and transverse directions to develop lateral resistance capacity. Using rock anchors to develop a full moment connection was not considered to be feasible for the pile diameter proposed. To create portal action in the longitudinal direction, the superstructure was made integral with the substructure at all abutments and intermediate pier supports. The rock anchor was post-tensioned to prevent uplift under ULS design loading and ensure contact for the transfer of shear by shear-friction.

The rock anchor was detailed for a 12m long socket into Basalt rock with a 150mm diameter core and grouted for durability over full length of pile and rock socket. The anchor was post-tensioned at deck level following grouting operations. The anchor consisted of 5 no. 15.2mm diameter 7 wire ordinary strand with a UTS of 261kN per strand. The anchor was bonded over the lower 6m of the rock socket only with the remaining length unbonded using greased sheaths. This was detailed to allow rotation of the pinned base without causing yielding of the prestressing strand assuming rigid body rotation of the pile section. Elongation of the prestressing strand due to rotation was able to be accommodated over the full debonded length reducing the increment in strain due to rotation to a minimum and within fatigue stress limits.

5 Construction

5.1 Precast construction

It was chosen to manufacture the precast beams for all of the bridges in Suva and incur the additional cost of transporting them to the sites on trucks that were specially procured by our logistics supplier for the project. This decision was quality driven as the yard's close proximity to substantial infrastructure meant that we could have oversight of the suppliers supplying the materials for the precast beams as well as back up generators for reliable power supply and mains potable water for curing the concrete and cleaning the bed ensuring fast turnaround of the beams.

The benefits of this location were evident when, due to poor quality aggregates in the Central Division of Viti Levu, the 50Mpa concrete required for the beams proved challenging to make. To overcome this, concrete suppliers would use a very high cement content of 550Kg/m^3. This high cement content coupled with warm ambient temperatures resulted in a mix that would lose workability quickly, thus affecting the placement and compaction of the concrete in tight forms as shown in Figure 6. The precast yards close proximity to an established batching plan overcame this issue.

The precast mould was recycled from another project in New Zealand and shipped to Fiji. Traditionally in New Zealand the stirrups for the beams would be cut and bent using computer software to ensure that their dimensions are consistent to meet the cover requirements in the project specification and wastage is minimised. In Fiji, these components were bent by hand and each stirrup had to be checked by fitting it into a template with 3mm tolerance. This required dedicated quality checks and it took a lot of time and wasted steel to set up the bending process. However, once the process was established there was confidence that the project specification was met.

Figure 6. Precasting operations in the Suva yard.

5.2 Piling

Although the piling methods used on this project were common industry practice, working away from an established plant yard in New Zealand made the procurement of specialist piling equipment critical to staying on programme. The Lomawai Bridge in particular, with its excavated piles which required stressed pin anchors in them, needed substantial equipment such as a rock chisel to break up the basalt layer and excavation tools to remove the spoil to ensure that the pile was keyed into competent rock. Not only did this require procurement well in advance but also thorough training of Fijian crane operators in the use of this equipment.

The manufacturing and installation of the anchors took considerable planning also. Anchors were manufactured in Auckland in order to ensure the quality of the components. Despite the fact that using an experienced New Zealand-based team ensured quality, it meant that plenty of contingency in anchor length was required as the exact depth that the piles would found at was not confirmed before fabrication. Thorough planning in regards to the stressing equipment was also required. For example the jacks cannot be calibrated in Fiji and this had to be taken into account when procuring them, nor could we return to the yard and fetch a simple screw to hold the stressing wedges in place. Detailed checklists had to be developed and adhered to, to ensure that all the required specialist equipment arrived on time. For the grouting of the anchors, a grout pump and operator was brought in from New Zealand. This ensured a quality operation and allowed local staff to be trained for future projects. This ensured that we mitigated the risks involved with this critical activity while achieving our goal to build capacity in our local teams, enabling this work to be carried out with confidence by local teams in the future.

During piling at the Vunivaivai Bridge we were required to drive piles using a bottom driven method to a depth of approximately 62m and prove their capacity using the Hiley formula. During the piling operation the casing that was being driven collapsed above the drop hammer trapping it inside at a depth of 55m. Prior to looking at alternative design solutions we chose to initially prove the pile capacity by conducting a static load test which if passed would allow us to build the bridge using the current pile location. This test method was chosen as it would allow us to use a reduced capacity reduction factor as set out in the Australian Piling Standard AS2159:2009 when compared with using the Hiley formula to prove capacity of the pile. The equipment for this testing is readily available to projects in New Zealand, however to the project team in Fiji this would mean a four week delay while it was imported to the country. To overcome this, a test frame was made using the 710UBs from the staging system and a 600t hydraulic jack from the precast prestressing operations. Figure 7 shows the pile load testing arrangement. Fortunately the load test proved the capacity of the pile at the shallower depth and we could proceed with the construction using this pile. This is an example of the innovation that is required to overcome construction challenges when working in a remote environment.

Figure 7. Pile load testing at Vunivaivai.

5.3 Construction on Vanua Levu

The fourth and final bridge was constructed at Coqeloa village on Vanua Levu, Fiji's second largest island. Although supported by the main city of Labasa only 30 minutes away, the construction infrastructure was limited which meant that we had to transport the majority of our plant and materials via barge from Suva. This required two trips on a 1000T barge and required thorough forward planning because not having something as simple as the correct bolt would cause significant delays.

In addition to the logistics challenges described above, the other main challenge we faced at this site was that the only concrete plant in Labasa (Figure 8) was dated and used a very manual process where cement was added by hand. This resulted in limited production of concrete: in the vicinity of $10m^3$/ hour. As the quality of the concrete was key to the project's outcomes, we engaged the supplier five months prior to commencement on site to conduct initial trials to overcome these concerns. Although the concrete reached the desired 40Mpa in the initial test, there remained concerns around quality control at the plant and their ability to deliver the consistent concrete that we required to ensure correct placement and compaction. Initial investigation required that the scales have calibration certificates issued and that a flow meter be installed to ensure that the correct amount of water was added to the mix. We employed the technique of cooling the aggregate stock pile overnight and conducting early morning pours to help combat the concrete becoming unworkable on the 30 minute commute to site. This in turn created consistency issues as the supplier had traditionally used an assumed water content in their mix design and did not carry out any adjustment to allow for the water in the aggregates. To overcome this issue an adjustment spreadsheet was developed so that our site staff could calculate an adjusted mix design based on the moisture content of the aggregates. Thus it was ensured that the accuracy of measurement in table 2.2 of NZS3104:2003 was achieved and that consistent mix was delivered to site.

Figure 8. Concrete batching plant in Labasa.

6. Acknowledgements

The authors acknowledge the support of Fletcher Construction and Fiji Roads Authority in allowing to permission to present this project. The contribution of the wider team including John McNeil, Ana Pereira, Mehdi Sarrafzadeh, Norbeminda Salinas and Geoff Thompson is also acknowledged.

7. Conclusions

Despite newly introduced design requirements that specified durability and seismic performance requirements higher than previously used in Fiji, four replacement bridges where successfully delivered in the Fiji Islands to the satisfaction of FRA.

8. References

Fiji Roads Authority, “Design Standard, Bridge, Jetty and Culvert Structures Version 1.2”, 2013, Fiji.

New Zealand Transport Agency, “Bridge Manual Edition 3”, 2016, New Zealand.