FERRYMEAD BRIDGE – FROM WIDENING AND SEISMIC UPGRADING TO REPLACEMENT, A CASUALTY OF THE CHRISTCHURCH EARTHQUAKE

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ABSTRACT

Identified in the Christchurch Lifelines Study as a bridge vulnerable to damage in a major earthquake, the Ferrymead Bridge on the key arterial route connecting the suburbs of Redcliffs and Sumner to the rest of Christchurch has subsequently been under investigation by the Christchurch City Council to increase its traffic capacity and upgrade its earthquake resistance. A contract was let in 2010 to undertake these works.

Surviving the September 2010 Darfield earthquake undamaged, the bridge fell victim to the February 2011 Christchurch earthquake with extensive liquefaction and soil lateral spread occurring at the site, displacing the abutments and piers inwards towards the centre of the river. After extensive investigation into options for recovering the bridge, the decision was finally taken to replace the bridge with a new structure.

This paper outlines the initial design to widen and seismically upgrade the original bridge, the damage sustained by the bridge from the Christchurch earthquake and measures instituted to stabilise the bridge as a result of that damage, and focuses particularly on the design now developed for the replacement structure. The significant issues involved in achieving earthquake resistance at a highly liquefiable site and in constructing in an environment of ongoing earthquake activity are discussed.

1 INTRODUCTION

1.1 Background

The Ferrymead Bridge site (refer Figure 1) has a long and interesting history dating back to the ferry service over the Heathcote River from which the site takes its name. A Cobb Cottage associated with the ferry remains on site, though it was badly damaged in the February earthquake.

The first bridge at the site completed in 1864 (Figure 2) was a timber structure with a rotating steel centre section to allow for navigation. This rotating section was retained after demolition shortly after 1907 and is now a single span bridge further upstream at Bowenvale. The second timber structure completed in 1907 (Figure 3) included a bascule bridge, parts of which are incorporated into a tram bridge at the nearby Ferrymead Historic Park. The third structure that was completed in 1967 comprised a 3 span prestressed concrete structure shown in Figure 4 and carried approximately 30,000 vehicles per day until it was recently demolished. The structure was also part of the over-dimensioned overweight route to the Port of Lyttelton and carried vital services including power, water, waste water and telecommunications.

The intersection on the east side of the bridge site has five legs including two on the lower slopes of Mt Pleasant which complicates the geometry and traffic management with increasing pressure over time for improvement. There were also concerns about the bridge's seismic performance and overall traffic capacity as a result of significant development in the area over the last 25 years.

The 2004 Christchurch Lifelines study highlighted the need to investigate the performance of the pier columns, in particular and the possibility of liquefaction. Also, with slips and rockfalls predicted along the only alternative route via Bridle Path Road, suburbs to the east were at risk of complete isolation.

Since 2004 many solutions have been considered including additional bridges upstream and downstream and an alternative route from Sumner across to the Brighton Spit, but the consistent response from the community was to request widening and strengthening of the existing structure.

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Figure 1: Ferrymead Bridge site location.

Figure 2: The 1864 Swing Bridge at Ferrymead (JN Taylor photograph, WA Taylor collection, Canterbury Museum).

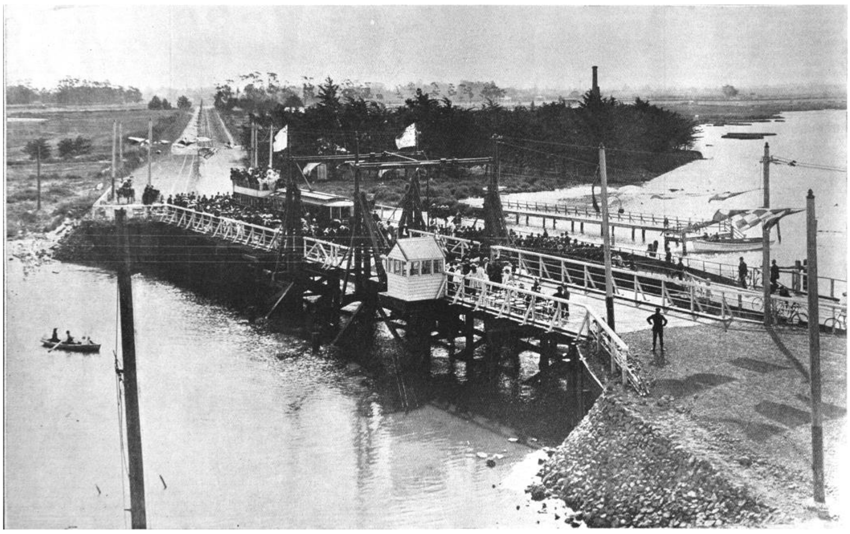

Figure 3: The first electric tram to cross the new bridge over the Heathcote River, 20 March 1907 (The Weekly Press photograph, Bishop collection, Canterbury Museum).

Figure 4: The recently demolished 1967 Ferrymead Bridge.

1.2 Strengthen and Widen Option

Developing a workable concept for widening and strengthening proved elusive. Options considered included adding land spans, river bed props, compressible backfill behind abutments and ground improvement. Many ground improvement options have been considered over the years including mix in place piles, stone columns and jet grouting, but with 20 m plus of liquefiable material, multiple services, complex traffic management demands and very high cost, ground improvement solutions were not favoured.

After much iteration a concept was eventually developed to envelope the existing structure in a new structure, completely underpin the existing superstructure and cut down the existing abutment walls such that they could displace under the new structure during lateral spreading. The concept also included short land spans over a void behind the abutments which reduced lateral spreading pressure on the new abutments and provided a useful space for services to deform under lateral spread displacements imposed on the approaches. This concept was taken forward for detailed design by Opus.

The design included reinforced concrete underpinning beams at the piers and abutments that spanned an ambitious 30 m. Longitudinal and transverse restraint was provided by 8 ~ 700 mm diameter piles at each end of each of the pier beams, 4 raked transversely and 4 longitudinally. Abutments were supported by vertical 1.1 m diameter bored piles at each end with minimal (pinned) sockets in the volcanic bedrock. The structure was designed to resist soil lateral spreading loads without the need for ground improvement.

The widened carriageway alignment necessitated asymmetric widening in order to avoid the historic Cobb Cottage and particular attention was paid to replicating the arched form of the existing structure. This made for an extremely complex design but the final form was well received by the public and careful staging of construction allowed for minimal disruption to traffic and services.

Despite the complexities of the design, tenders received were within budget and construction started a week before the 4 September 2010 Darfield Earthquake.

1.3 Canterbury Earthquakes Effects

The Magnitude 7.1 Darfield Earthquake on 4th September 2010 with an epicentre at a distance of 40 km, caused no damage to the bridge apart from minor cracking, and no liquefaction effects were observed, which came as somewhat of a surprise. The suggested explanation was that due to the relative location of the source of the earthquake, the nearby Port Hills may have influenced the ground effects at the site and the Ferrymead Bridge remained vulnerable, which proved prophetic.

The epicentre of the 22 February 2011 earthquake was within 2 km of the bridge site and generated very large horizontal and vertical accelerations. This caused major damage to the existing structure, its approaches, to the completed bridge works and to the contractor’s temporary works.

Damage to the existing structure and approaches was very much as expected with liquefaction generating distortion, cracking and settlement of the approaches and damage to the water mains and pumped sewer main. Lateral spreading generated abutment rotation and pier pile cap displacement towards the centre of the river, as can be seen in Figures 5 and 6, and liquefaction caused settlement of those piles not founded on rock. The bridge was immediately closed pending assessment.

Figure 5: The Mt Pleasant abutment, displaced forward and rotated backwards about its raked piles founded on the bedrock; new bridge widening piles displaced outwards by lateral spreading.

Figure 6: Pier columns rotated due to inward displacement of their supporting pile caps.

The alternative route via Bridle Path Road also became impassable due to rockfall risk, with Sumner effectively cut off from the City. Alternative routes over the Port Hills were also closed and remain so.

Significant damage to the widening and strengthening works also occurred due to lateral spreading displacing the already completed abutment piles and their partly completed headstocks.

Damage to temporary works, including coffer dams (for new pier construction) and staging bridges, was extensive. Both staging bridges were damaged beyond repair and significant temporary strengthening works were required just to enable removal of the 150 t cranes.

The coffer dams, in varying degrees of completion, suffered perhaps the most telling damage with significant distortion in plan, together with vertical rotation and joint failures.

The fully completed south east coffer dam was inundated within seconds due to piping failure from liquefaction and joint failure due to the imposed distortion. Fortunately no one was in the coffer dam at the time, however, the site staff were very concerned at what the implications may have been for staff who were about to descend into the coffer dam when the February event occurred.

1.4 Christchurch Earthquake Response

The immediate response to the 22 February 2011 Christchurch earthquake was to close the bridge, divert traffic around Bridle Path Road and await assessment. As expected the Bridle Path Road alternative proved a nightmare and priority was given to urgent assessment of the bridge. The assessment showed the bridge superstructure to be in remarkably good condition but the substructure was significantly distressed. However load paths were still viable and following proof loading, access for light vehicles was reinstated two days after the earthquake, while heavy vehicles continued to use the highly undesirable Bridle Path Road.

More rigorous inspection and assessment then followed together with ongoing monitoring of deck levels and abutment rotation, particularly following the numerous large aftershocks. Assessment showed critical issues with the continuous superstructure caused by differential settlement with the city pier having sagged relative to the rest of the bridge deck. Despite considerable overstress, the deck visually appeared undamaged. Before heavy vehicles could be reintroduced, and to protect the bridge superstructure for possible recovery, the relative sag needed to be jacked out. A jacking system was implemented and the bridge was de-stressed and finally returned to Class I status by Easter 2011.

In addition, two measures were taken to secure the pier pile cap against further displacement. Firstly, the material directly contributing to lateral spreading behind the pier pile cap was removed, and secondly, tie bars were introduced to restrain the pile cap as shown on Figure 7.

Figure 7: Tie rods installed to restrain the pier pile caps against further inward displacement.

The newly completed Mt Pleasant abutment was secured by the installation of a temporary shear key.

As a further contingency, the Contractor's temporary staging bridges were reconstructed with the ability to convert these to take traffic should the need arise.

All of the securing works were tested many times in the aftershocks that followed, particularly during the 16th April and 13th June 2011 events, and apart from minor damage the bridge and the reinstated temporary works performed well.

1.5 Recovery Options

The remaining issue was whether to continue with the widening and strengthening contract or abandon the existing structure and design a completely new bridge.

In order to continue with the existing contract a different approach would be required for design and construction. Due to the now sloping deck, displaced piles, distorted geometry and conflict with tie bars, an already complex design was becoming even more complex requiring a complete redraw of the project and redesign of the foundations. The foundations became a contractual issue with health and safety concerns around coffer dams requiring either a completely different approach with sheet piling driven to rock and significantly enhanced strengthening of the coffer dams, or a change to the pier foundations with very large 3.5 m diameter vertical caissons cantilevered out of the volcanic rock. Both options were very expensive and carried significant risk. A new bridge started to look more and more attractive offering opportunity to improve resilience and serviceability.

An interactive approach with the Contractor regarding the alternative of a new bridge concept resulted in a proposal for a two span bridge with either a steel or Super T Beam superstructure. This proposal was then compared to continuing with the widening and strengthening option. Following a matrix evaluation and external review, the recommendation was made and accepted to design and construct a new bridge, though it was accepted that this would still be a challenging undertaking, just as expensive as continuing with the existing and not without risk, particularly in relation to foundations and the construction risk from ongoing seismic activity.

2 FACTORS INFLUENCING DESIGN OF THE REPLACEMENT BRIDGE

In addition to the usual factors associated with bridge design of roading geometry, traffic and pedestrian capacity, waterway capacity and provision for services, factors influencing the design of this bridge replacement, particularly from an earthquake resistant design perspective, included the following:

- *Site geological and geotechnical conditions:* The site comprises estuarine deposits of variable depth along the bridge, underlain by bedrock of a highly variable quality.
- *Site seismicity:* High seismicity with ongoing seismic activity during construction of the bridge.
- *Combined seismic loading and soil lateral spread effects:* The confirmed potential for liquefaction induced lateral spreading loads on the sub-structure combined with the seismic inertial loads, not only during the design life, but potentially during construction.
- *The potential effects of ongoing seismic activity on construction:* Potential lateral loads during construction of the bridge, particularly on the unpropped piles, as experienced on the bridge widening piles in the February 2011 event.
- *Tsunami*: Located on a coastal estuary, the potential exists for this bridge to be subjected to tsunami.
- *Maintenance of traffic flows:* Maintenance of traffic access across this crossing point of the Heathcote River is essential, particularly given the vulnerability or closure of other roads into Sumner.

3 SEISMICITY

3.1 Design Seismicity

Based on the NZTA Bridge Manual requirements prevailing at the time for seismic design, the bridge was categorised as an importance level 2 structure for which the design earthquake event is one with a 1,000 year return period, and the site was classified as class C, a shallow soil site.

As a consequence of the seismic activity being experienced in the vicinity of Christchurch, the seismic zone factor for the area has been reassessed by the Department of Building and Housing and increased to Z = 0.30 from 0.22 previously, resulting in a significant increase in the seismic response to be designed for. In addition the Council expressed a desire, due to heightened concern over route security to the eastern suburbs, for the replacement bridge to be categorised as an importance level 3 bridge to be designed for a 2,500 year return earthquake event if possible, further increasing the design seismic loading. The peak ground acceleration for a design earthquake event with a recurrence interval of 2,500 years is 0.52g.

3.2 Seismicity during Construction

There is a high likelihood of a significant aftershock earthquake occurring inducing liquefaction and lateral spread of the approach embankments during construction of the bridge. Local earthquakes in the Christchurch area of a magnitude greater than or equal to 5.5 are considered likely to induce liquefaction at the Ferrymead Bridge site. The predicted probabilities of an earthquake of greater than or equal to magnitude 5.5 based on GNS (2012) was:

- Within 1 month: 5%
- Within 1 year: 44%

It is also possible that earthquakes from other sources (eg Alpine Fault, foothills faults) could occur during construction, but the likelihood of such events during an 18 month construction period for the piles, is relatively very low. The construction of the substructure is expected to happen over a period of 18 months from about March 2013. Thus there is a high probability of an earthquake inducing liquefaction at the site occurring during the period of construction of the bridge piles, before the piles are supported at their tops by the bridge superstructure.

4 GEOTECHNICAL CONDITIONS

4.1 Geology and Geomorphology

Ferrymead Bridge crosses the mouth of the Heathcote River where it opens out into the Avon Heathcote River Estuary. It is situated on the fringe between the outwash alluvial plains and the Port Hills. Estuarine deposits of the Holocene aged Christchurch Formation overlie basalt and trachytic lava flows interbedded with breccia and tuff of the Late Miocene aged Mt Pleasant Formation. The approach fills for the existing bridge have been formed with sand and gravel.

4.2 Site Investigations

Previous site investigations include borehole investigations prior to 1965 for the design of the existing bridge, with boreholes and Static Cone Penetration testing carried out in 2003 to assess the risk of liquefaction at the bridge site. The results of these investigations used for the concept design were supplemented with specific investigations in 2012 undertaken to provide information for the detailed design and construction of the proposed replacement bridge. This comprised the drilling of 17 boreholes and 7 piezo-cone Penetration Tests. The Majority of the boreholes were located so as to double as pile proving holes for the replacement bridge. Laboratory tests included particle size distribution testing of the soils and unconfined compressive strength tests of various rock core samples. Additional measurements were made using strain gauges to assess the elastic properties of the rock.

4.3 Ground and Groundwater Conditions

The site investigations confirmed the ground profile consisted of 2 m to 3 m of dense gravel and loose sand fill at the approaches to the bridge, overlying approximately 7 m to 21 m of estuarine sediments comprising loose to medium dense, fine sand. Beneath this was bedrock consisting of inter-bedded basalt and pyroclastic material. The pyroclastic material comprising breccia and volcanic agglomerate was recovered as a relatively weak rock or residual soil, making strength testing of this material difficult. Testing of the more intact pyroclastic material gave unconfined compressive strengths in the range of 2 MPa to 15 MPa. Testing of Basalt samples gave unconfined compressive strengths of over 120 MPa, but commonly failed at lower loads along pre-existing defects giving strength values in the range of 25 MPa to 75 MPa. The thickness of the soil increases, and the soil-rock interface dips in the northwest direction as shown in Figure 8. Groundwater levels at the approaches behind the abutments vary from 2.7 m to 4.3 m, and fluctuate with river and tide levels.

4.4 Liquefaction

An assessment of the potential for liquefaction using the Robertson and Wride (1997) method indicated that the upper layer of gravel fill at the abutments was not susceptible to liquefaction, but the lower sand fill was loose and prone to liquefaction. The estuarine sand deposits down to bedrock, except for some local layers of dense sand, were also prone to liquefaction, with assessments indicating that liquefaction will occur in earthquake events of 150 years return period or greater. The soil layers that are likely to liquefy in a 2,500 year return period design event are shown on Figure 8.

Liquefaction of the loose, saturated layers underlying the abutments could result in lateral spreading of the soil towards the Heathcote River. The assessed potential for liquefaction and lateral spreading is confirmed by the observation of damage during the 22 February 2011 earthquake and subsequent aftershocks. As the liquefiable materials laterally spread, they would impose relatively large loads on the abutment and piles. In addition, the reduction in soil stiffness of the liquefied deposits will result in a loss of support to the sections of piles in the liquefied deposits.

5 DEVELOPMENT OF THE DESIGN CONCEPT

5.1 Issues from the Bridge Widening Contract and 2011 Earthquakes

Through the course of evaluation of options for recovery from the damage to the previous bridge, a number of options for a replacement bridge were investigated leading to the broad form of the replacement bridge being decided on.

Sheetpile cofferdams installed for the construction of the pier pile caps intended for the previous bridge were not able to be extended down to bedrock due to conflict with the raked piles. Ongoing earthquake events with the potential to cause liquefaction at the site, destabilising the cofferdams, led the contractor to consider that working within the sheet pile cofferdams and using them to support the equipment for installing the raked piles was too dangerous to his construction workers and too risky to the equipment. Consequently, taking account of and agreeing with the contractor’s concerns, the Council agreed that the founding solution be changed to one comprised of vertical cylinder foundations. This form of solution enables the cylinder casing to be extended above the high tide level to act as a cofferdam for the construction of smaller diameter columns above, but also requires that the cylinders be socketed into the bedrock in order to provide longitudinal restraint to the bridge against seismic response and soil lateral spread loads. Restraint against transverse seismic response can be achieved through portal frame action of the pier and abutment bents.

The length of the crossing lends itself to a two or three span bridge solution, with a two span solution expected to result in less out of balance loading acting on the pier piles from soil lateral spreading. Precast prestressed concrete and steel superstructure options were considered, with a two span bridge comprising precast prestressed super-T beams being selected as the preferred solution giving consideration to cost, durability and maintenance requirements in the coastal environment of this bridge (refer Figure 9).

5.2 Foundation Concepts

The fundamental cause of damage to bridges in Christchurch was liquefaction and lateral spreading (Brabhaharan, 2012), and was also the case with the Ferrymead Bridge. Therefore a foundation design concept that addresses liquefaction and in particular lateral spreading loads on the abutment and pier piles was of paramount importance for the new bridge.

There are potentially two fundamental foundation concepts to making bridges resistant to liquefaction and lateral spreading, which are:

a) Ground improvement to make the soils, particularly at the abutments, resistant to liquefaction or to reduce the potential for and magnitude of lateral displacement. Such an approach was suggested by Chapman and Brabhaharan (1994) and used to retrofit the Cobham Bridge in Wanganui to improve its seismic performance (Brabhaharan *et al*, 2009), and subsequently was also used for the bridges along the Christchurch Southern Motorway. The ground improvement at the Christchurch Southern Motorway bridge abutments that had been constructed at the time of the 2010-2011 earthquakes performed well with no noticeable damage (Greg Saul pers comm).

b) Stronger foundations and associated bridge structure to resist the substantial loads imposed by liquefaction and associated lateral spreading.

Early in the design process the use of ground improvement was discounted due to the presence of numerous services which were considered to be very difficult and costly to relocate or protect; the depth of liquefiable deposits to over 20 m depth at the city end abutment; the perceived uncertainty about the performance of the ground improvement, particularly because an adjacent block of flats on improved ground had not survived the earthquake; and the potential cost of ground improvement.

The following measures were developed to address the liquefaction and lateral spreading loads on the bridge and its ability to withstand the soil loads:

a) Isolate the main bridge from larger lateral loads from the liquefaction resistant crust of gravel fill by forming a void behind each abutment with a short land span to bridge the void.

b) Minimise the number of piles and their sizes at the abutments to minimise the loads imposed on the main bridge substructure. Three 1,100 mm diameter vertical piles were adopted at each abutment.

c) Adopt abutments tightly linked to the superstructure so that the lateral spread loads acting on one abutment can be propped by the superstructure across the bridge to be resisted by the pier and the abutment on the opposite side.

d) Locate the pier away from the river banks to minimise any lateral spread loads on the piers. A two span bridge with a central pier was adopted.

e) Use a larger number and size pier piles to enable them to take some of the lateral spread loads. Four 2,400 mm diameter cylinder piles were adopted at the central pier.

f) Permanently case the piles so that they are more robust to the effects of liquefaction and lateral spreading, particularly at the interface between liquefiable and non-liquefiable ground, where piles have been observed to be damaged in past earthquakes.

g) Socket the piles into bedrock so that the piles do not displace due to lateral spread loads.

The foundation concept was confirmed during detailed design of the bridge to be resistant to liquefaction and lateral spreading.

Consideration was given to the manner in which the piles could be constructed and whether or not to utilise the flexural strength of the pile steel casing in the design of the piles. For the design of the substructure, the dominant loading case was soil lateral spread loading combined with the earthquake response in the longitudinal direction. Soil lateral spread loads are a direct function of the diameter they act against and thus it was desirable to seek to minimise the number and size of piles located at the abutments.

Using the pile casing to provide flexural strength was subject to several constraints: minimum casing thickness requirements in order to avoid plate buckling, maximum plate thickness that the contractor was able to roll into casing, and a need for the internal surface of the casing to be free of protrusions (e.g. shear connectors) to facilitate the operation of the contractors equipment for installing the casing.

The method of construction of the bored piles was considered, that is whether to take the permanent casing all the way to the bottom of the pile through bedrock. Using the casing for flexural strength requires the casing to extend almost to the bottom of the socket and infilling of the gap between the casing and the wall of the rock socket with grout, which is not easily done. Not using the casing for flexural strength, which might possibly allow open-holing (i.e. uncased excavation) in the bedrock, was considered, but would have required larger diameter piles. Also open-hole excavation introduced additional risks associated with collapse of the weaker volcanic breccia and agglomerate layers formed by pyroclastic volcanic soils. A pile casing taken to within 1 m of the pile base was adopted together with grouting of the pile casing-rock interface, as discussed in Section 6.4.

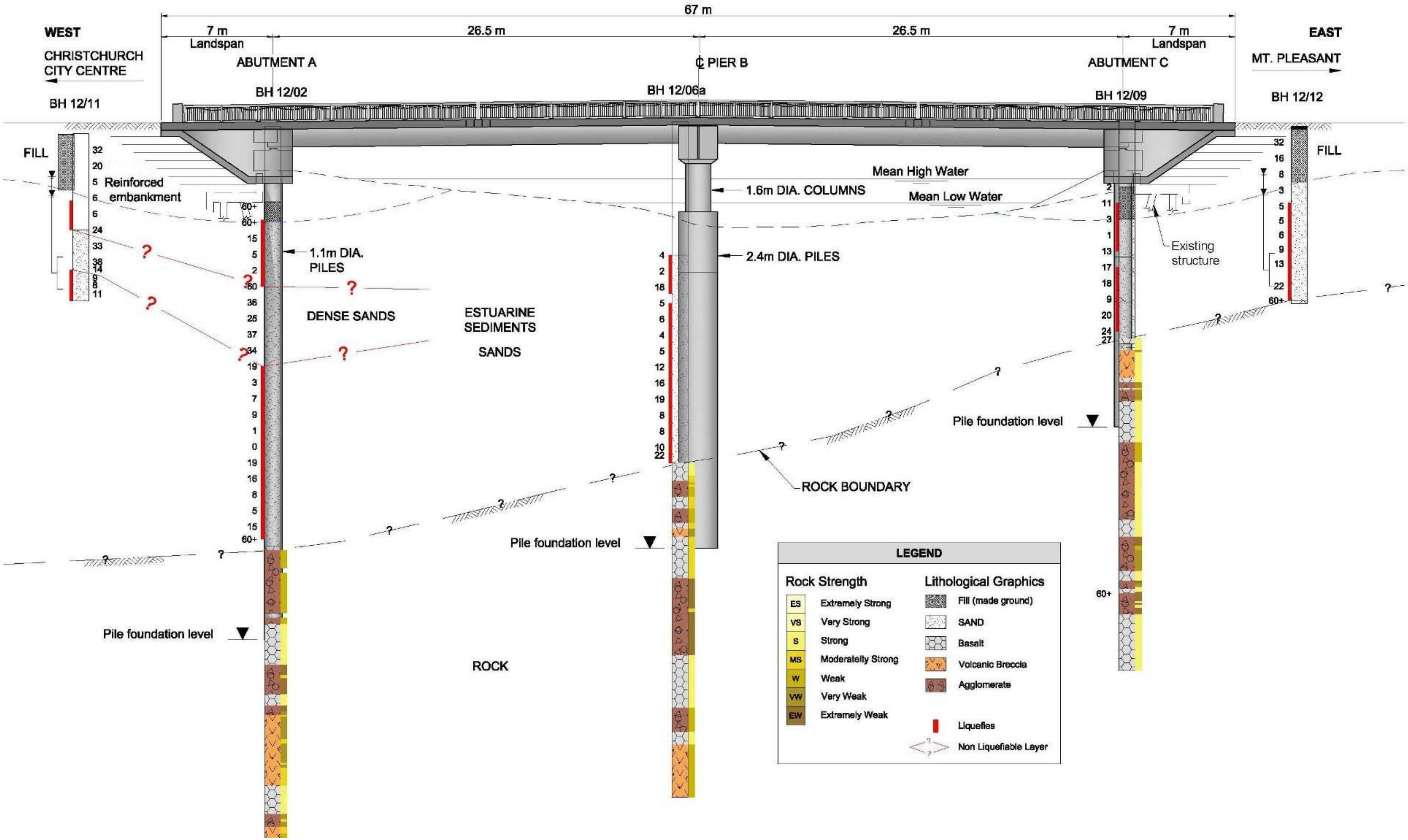

Figure 8: Longitudinal section showing the ground conditions and liquefaction vulnerability.

Multiple analyses were conducted to optimise the piling size and arrangement. 3 ~1.1 m diameter piles with 20 mm thick steel casing have been adopted at each of the abutments, and 4 ~ 2.4 m diameter piles with 32 mm thick steel casing for the pier. The piles are designed on the basis of the casing contributing flexural strength, without composite action being taken into account.

5.3 Superstucture Concepts

Further evaluation was then undertaken into more detailed aspects of the structural form. Firstly, consideration was given to whether the superstructure spans should be constructed as simply supported or integral with the supports. Factors compared and weighed up included: robustness of the structure, ease of accommodation of differential temperature effects, the benefits and effects of providing longitudinal frame action, the effects of flexibility in the pier headstock on the distribution of moments in the longitudinal girders, the ease of beam erection and building in, the accommodation of services, the design effort involved and the implications for project cost. Simply supported construction was selected as the preferred approach on the basis of design and construction simplicity and lower potential to give rise to cost increases.

Figure 9: Photo-montage of the proposed replacement Ferrymead Bridge.

6 SPECIAL ISSUES

6.1 Magnitude of Liquefaction induced Lateral Spread Loads on Piles

The lateral loads on the piles due to liquefaction induced lateral spreading were estimated as 4.5 times the passive pressure on the piles to account for three dimensional effects of the flow loads on the pile. Post-liquefaction strength of the soils was used to assess the flow loads.

6.2 Partial Removal of Existing Abutment and Piles

The existing abutment piles are a front row of closely spaced raked piles and a rear row of more widely spaced vertical piles. The relatively close spacing of these piles can cause the soil to arch between them, preventing the soil from 'flowing' around the piles. Under liquefaction induced lateral spreading, this arching can potentially impose large loads on the new piles as the soil and piles can act as a 'wall'. Removal of a section of the existing piles and associated abutment structure, as shown in Figure 10, has been adopted to ensure the existing piles do not attract and impose large additional loads on the proposed new piles. The piles installed for bridge widening in 2010/2011, displaced by the liquefaction and lateral spreading in the 2011 earthquakes have been abandoned but can remain in place below ground level, being assessed to be outside the area of influence on the new piles.

6.3 Foundation Design

The variability of the interbedded rock layers with depth and across the site required careful consideration when assessing pile founding levels. In addition, as multiple piles provide support to the abutment and pier headstocks, these elements are sensitive to differential settlements. Based on structural analyses, a differential settlement of a maximum of 20 mm was adopted between adjacent piles to ensure that the pier and abutment headstocks do not become distressed by the settlement of the piles. Longitudinal differential settlement between the abutment and pier piles was less critical due to the spans being simply supported, and was satisfied when the assessed differential settlement between adjacent piles was within the tolerable limits.

As discussed in Section 4.2, the drilling of proving holes for the foundations was brought forward to the design stage to allow consideration of pile foundation levels at each pile as part of the detailed design. Investigations in the vicinity of two of the central pier piles showed the founding layer varies significantly in thickness locally across the width of the piles.

The piles were founded a minimum depth into bedrock to provide lateral load capacity and adequate fixity. Due to the uncertainty associated with the in-situ rock properties of the pyroclastic material (volcanic breccia and agglomerate), each pile depth and potential settlement and differential settlement between adjacent piles was considered iteratively to arrive at a depth for each pile. After ensuring the fixity requirements were satisfied, the settlement requirements were achieved by ensuring the pile founding level had a minimum thickness of basalt beneath the pile tip to reduce the settlements.

6.4 Contact Grouting of Pile-Rock Interface

Contact grouting to fill the space and any voids that develop during construction of the pile has been adopted to ensure that the piles have good lateral support and can resist the lateral loads imposed without undue rotation of the piles.

A number of approaches to grouting were considered:

- Internal grouting from within the pile casing using a de-watered pile casing or divers in a wet hole, by installing ports through the pile casing and grouting under pressure from within the casing, similar to that adopted by Ramsay and Marshall (1995) for the Ewen Bridge in Lower Hutt. This approach had risks associated with operating within the casing, particularly in a seismically active environment, and also very high costs.

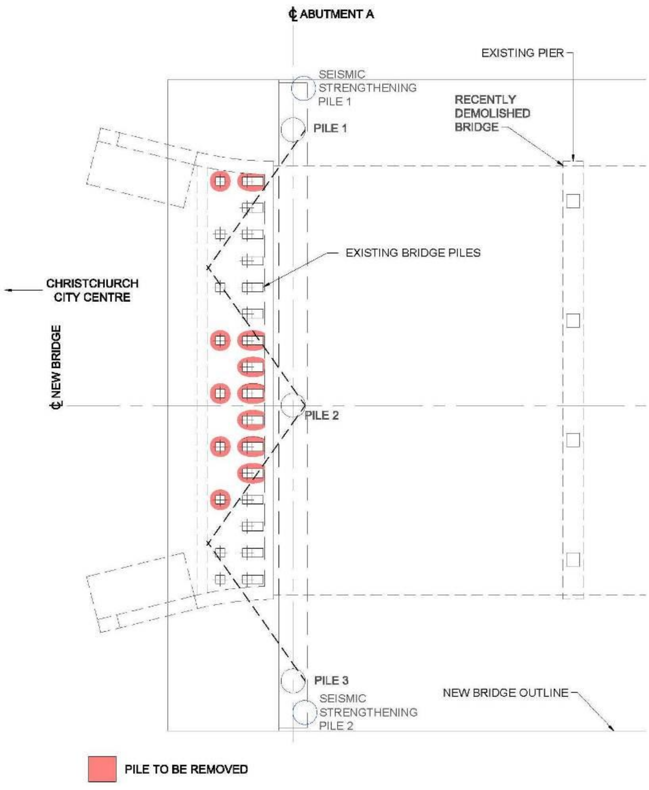

Figure 10: Core group of piles at the City abutment considered for removal, (Mt Pleasant abutment similar).

- External grouting was developed as an alternative technique using tube-a-manchette methods, with high pressure grouting similar to that developed for post-grouting of soils nails (Brabhaharan, 2007).

The external grouting approach was adopted, and a grouting methodology comprising the following broad steps has been developed and put in place in the bridge contract:

1. After excavation with associated installation of permanent casing to within 1 m of the pile base, install internal grout tubes at 1/3rd points around the circumference of the pile.
2. Drill external observation holes close to the pile casing, and observe any rise of grout while concreting the pile.
3. Fracture the concrete at the base of the pile using high pressure water injected through the internal grout tubes, and grout through the internal grout tubes using coloured grout.
4. Drill external grout holes at 1/3rd points around the pile casing (staggered with the location of the internal grout tubes), observing the presence of voids and grout infiltration, and install grouting tube with grout nodes. Use different coloured grouts.
5. Install tube-a-manchette through the grout nodes and grout alternate nodes using pressure grouting, and fracture the grout using high pressure water.
6. Grout through the other alternate nodes using high pressure grout, fracture the grout after 12 hours and repeat as necessary.
7. Drill another three external grout holes staggered with the initial grout holes, and examine grout filling of voids, and repeat grouting episodes as necessary.
8. Review grout records and confirm acceptability of the grouting of the casing to rock interface.

Figure 11 shows the indicative grouting of the casing – rock interface. The grouting methodology has adequate monitoring and verification procedures to check the adequacy of the grouting and the grouting process may be able to be terminated early if the verification provides adequate confidence of grouting of the interface.

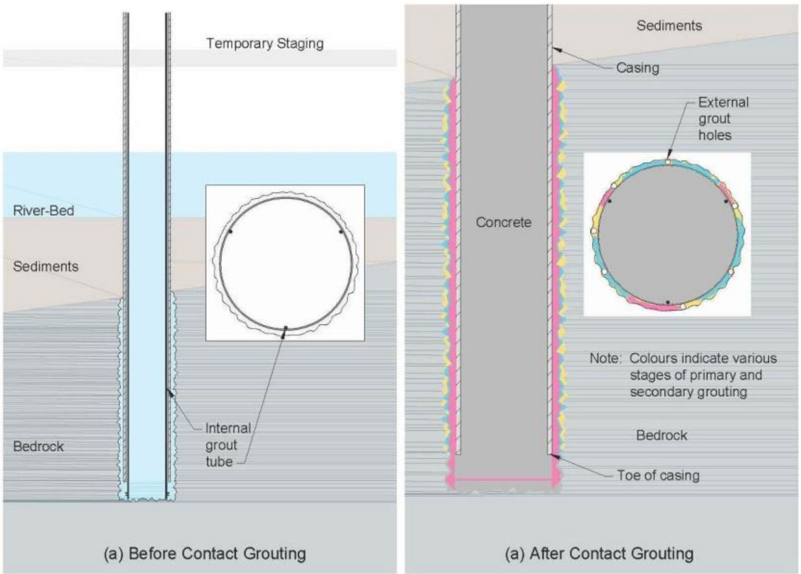

Figure 11: Proposed pile grouting.

6.5 Mitigation of Risk to Piles during Construction

There is a risk to the proposed new piles during construction due to liquefaction and lateral spreading. The piles under construction will be vulnerable to the large lateral loads and consequent displacement beyond acceptable tolerances, until the time they are propped by the reinforced concrete beams associated with the permanent superstructure. This is significant because the bridge widening piles were displaced and damaged by the 2011 earthquakes and there is an ongoing risk of significant aftershocks as discussed in Section 3.2. This risk was carefully considered, and risk mitigation measures have been developed to substantially reduce this risk, until the new superstructure beams are in place. These measures are:

1) Piles socketed into the underlying rock and grouted into place by external grouting of the pile-rock interface, so that they would develop fixity at the bottom (the pile cannot displace or rotate at the base where it is in rock) and are less prone to displacement due to liquefaction, as happened with the piles in the recent earthquakes.

2) Building greater tolerances into the design of the pier and abutment headstocks, so that small displacements can be tolerated.

3) Excavate soil and maintain a gap behind the proposed new abutment piles, down to about mean low water level in order to reduce the lateral spread loads on the piles, should an earthquake and consequent liquefaction and lateral spreading occur during construction.

4) To further reduce the potential lateral displacement of the new piles, the pier piles will be tied together with temporary beams, and the new abutment piles will be propped against the more robust pier piles, during their construction, and until they are propped by the permanent superstructure beams.

5) Optimise the sequence of construction so that:

a) The construction period during which the piles most vulnerable to liquefaction induced lateral spreading will be constructed later in the construction period, so that:

   - They are exposed to a lower probability of significant earthquakes (the probability of significant earthquakes decays over time since the initial Canterbury earthquakes).
   - They will remain un-propped by the permanent superstructure for a shorter period of time.

b) The more robust 2,400 mm diameter pier piles, which are the furthest away from the river banks and are the least susceptible to lateral spreading due to liquefaction will be constructed first, followed by the abutment piles at the Mt Pleasant end where bedrock is significantly shallower and hence the piles much stiffer. The abutment piles at the city end, which are the most vulnerable to displacement by lateral spreading, will be the last to be constructed.

c) The abutment piles will be temporarily propped using steel struts against the more robust pier piles during their construction, They will remain propped until the superstructure beams are in place and permanently prop between the abutments and the pier.

d) The construction of the superstructure closely follows the completion of the substructure.

The risk mitigation measures as outlined above will minimise the risk to the piles during construction of the new bridge.

7 CONCLUSIONS

Bridges have existed at the Ferrymead River crossing since 1864 with the first two being quite novel in their form, firstly a horizontally pivoting “swing bridge”, followed a bascule bridge. These bridges in turn were followed by construction of a modern prestressed concrete bridge in 1964.

Damage to this latter bridge and to the initial works to widen and seismically strengthen it by the 2011 earthquakes, was similar to damage experienced generally to the bridges in Christchurch as a consequence of liquefaction and lateral spreading, and has resulted in this bridge being written off. The design of the replacement Ferrymead Bridge, now under construction, involved a number of challenges due to the presence of deep estuarine deposits that are prone to liquefaction and lateral spreading underlain by variable volcanic rocks. Thus there has been a need to ensure that the new replacement bridge is resilient to earthquakes and consequent liquefaction.

There are two approaches to mitigating the effects of liquefaction and lateral spreading. One is using ground improvement as has been demonstrated by bridge retrofit and new bridge projects over the past 10 years. This was not selected due to complexities at this site and the potential cost.

The other approach of design of the foundations, substructure and superstructure to resist the liquefaction loads was adopted for this bridge. In addition, the lateral loads were reduced by forming a void behind the main bridge abutments and using a short land span, which can be quickly restored after any damage in an earthquake. The liquefaction loads are resisted by the piers and by the passive pressure resistance of the foundations at the opposite abutment.

There were a number of other challenges that were addressed in the design of the bridge. The location and effect of existing piles on the new bridge foundations was carefully considered, and a section of the existing piles and abutment are to be removed to ensure that they do not attract and impose a higher soil load on the new piles. The piles were designed with a permanent casing and are to be socketed into the rock, and this required development of an external contact grouting methodology to ensure good lateral capacity. Given the active seismic environment with potential for significant aftershocks, the risk to the unpropped piles during construction has been carefully considered and is proposed to be mitigated by socketing the piles into rock, programming works to minimise exposure and temporarily propping the vulnerable abutment piles against more robust pier piles during construction.

Following demolition of the previous bridge, construction of the new bridge commenced in May 2013.

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