concretenz
BUILDING RESILIENCE
CONFERENCE 2022
13-15 October 2022
Energy Events Centre | Rotorua

# WAIKATO EXPRESSWAY – HAMILTON SECTION

TED POLLEY

City Edge Alliance, Hamilton

## SUMMARY

The Hamilton section of the Waikato Expressway is a key piece of infrastructure supporting this growth and will be the primary strategic transport corridor for the Waikato region. It is the largest roading project ever undertaken in the Waikato, a region considered to be highly susceptible to liquefaction during a seismic event.
The Hamilton section is being delivered by the City Edge Alliance comprised of Waka Kotahi (client); Fletcher Construction and Higgins (constructors); and Beca and Coffey (designers). The 22km expressway involves 4 million m3 of earthworks, 17 bridges, and five major intersections.
This paper discusses innovative detailing allows rapid and reliable reinstatement for traffic, thereby providing structural resilience for a key lifeline route.

## INTRODUCTION

The Waikato Expressway forms the key transport corridor between Auckland and Hamilton. It has been constructed in stages over the last several decades. The expressway will improve economic growth and productivity through efficient movement of people and freight. A key outcome is to significantly reduce the number of fatal and serious injury crashes in this area. The Hamilton section is the final stage to complete the Waikato Expressway.
The Hamilton section forms 22km of expressway, commencing at the Lake road junction (end of the Ngaruawahia section of the Waikato Expressway). It then runs south to the east of Hamilton, connecting to the existing Tamahere Interchange which was constructed as part of the Cambridge Bypass section of expressway. The Hamilton section has 17 bridge structures and 5 major intersections (see Figure 1 below). There is approximately 4 million m3 of earthworks for approximately 22km of expressway. The construction programme is four years, with completion expected late 2021.

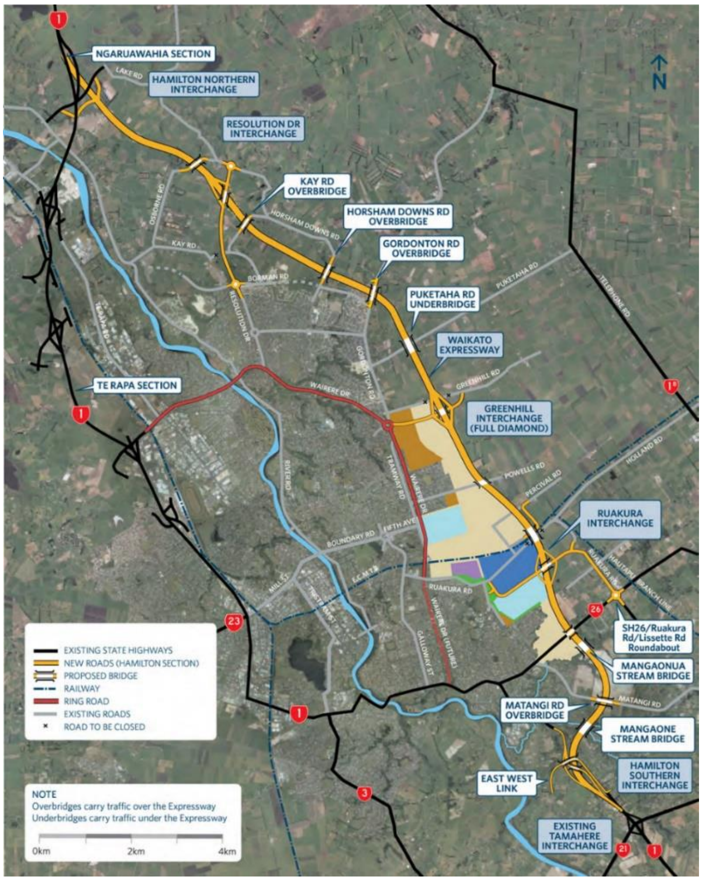


*Figure 1: Plan of Hamilton Section of Waikato Expressway showing major structures*

This section of the expressway is delivered as an Alliance. The Alliance partners are: Waka Kotahi as the Client, Fletcher Construction and Higgins (now part of FCL) as the Constructors, and finally Beca and Coffey as the Designers. A sub-Alliance with Hic Bros was formed to generate efficiencies with the earthworks.

## SEISMICITY

A site-specific spectrum was prepared by GNS Science for the Waikato Expressway in 2010. Seismic accelerations were slightly lower than NZS 1170.5 spectra values for the Waikato region. In addition to the seismic Design Event (DE), the updated Waka Kotahi Bridge Manual (3rd edition, Amendment 1) required the design to consider the effects from the Major Earthquake (ME = 1.5xDE) and to detail the structure to avoid collapse under this load case.
The nearest active faults are more than 20km away, so near fault factors were not required.
The site subsoil classification for most of the Hamilton Expressway was Class D for deep and soft soil sites. However, in some locations of particularly poor soils such as the gully bases, subsoil Class E category was considered more appropriate.
Bridges carrying the expressway were designed for Importance Level 3 loading (2500-year return period). Bridges crossing above the expressway were designed for Importance Level 2 (1000-year return) as specified in accordance with the project Minimum Requirements.

## LIQUEFACTION ASSESSMENT

The Hamilton section of the Waikato Expressway is located in the middle of the Hamilton Basin shown in Figure 2 below. The light pale areas shown in the geological map are extensive low-lying areas with thick sequences of relatively young soils. Many of them are volcanic pumiceous sediments deposited by the Waikato river. These low-lying deposits of mostly young pumiceous sands typically have a high-water table, particularly in the northern end of the Hamilton Basin.

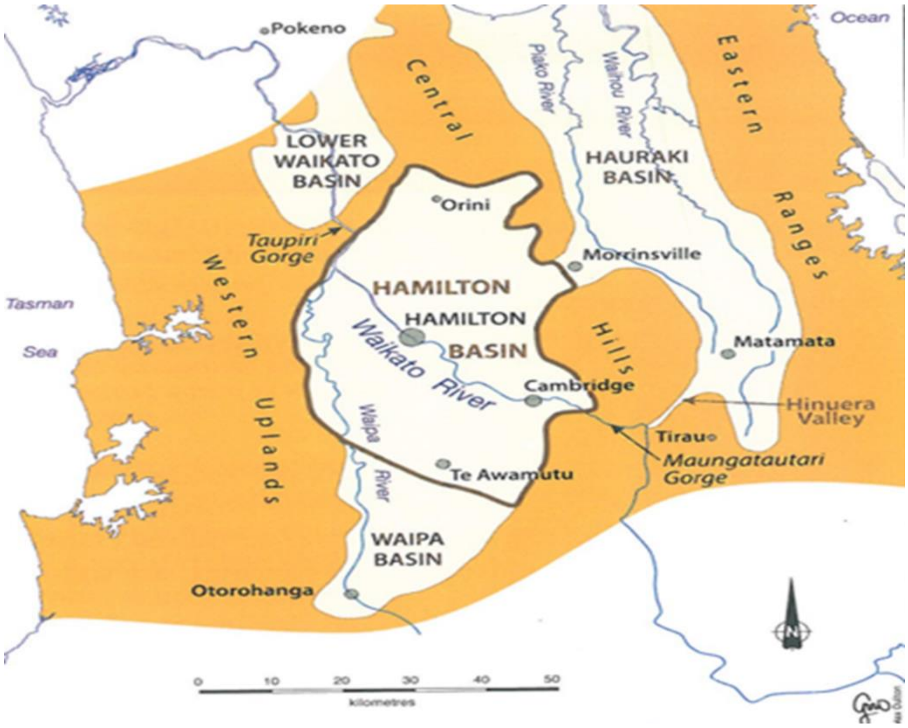


*Figure 2: Geology of Region*

Based on previous experience, it was anticipated that there would be a significant liquefaction risk in the young pumiceous soils of the lowlands of the Hamilton Basin. Historically, liquefaction potential has been assessed using what is referred to as the Simplified Method. The Simplified Method involves undertaking some penetrometer-based testing which involves pushing or hammering a device into the ground measuring the resistance. This resistance is then compared to a plot of conditions where liquefaction has and has not been observed in the past.

Over time penetrometer-based methods have become more sophisticated and the Simplified Method has become more complex. The penetrometer methods all have the potential to crush the pumice in the soils and conservatively underestimate the liquefaction resistance. The Waikato pumice matrix is different from the Canterbury pumice (which is where the Simplified Method was developed) and so it was decided to use shear wave velocity-based assessment for determining the onset of liquefaction. This method uses low amplitude seismic waves to measure the stiffness of the soil. Shear wave velocity-based method was considered less conservative than the Simplified Method, resulting in significantly less ground improvement as discussed further in this paper.

In order to demonstrate to the client that shear wave velocity was a more realistic assessment, a paleo liquefaction study was carried out (meaning study of ancient liquefaction) to investigate if historically liquefaction has happened at the site. The results were used as a check on whether the penetrometer-based method or the shear wave velocity-based method predicted liquefaction most reliably. Practically, the paleo liquefaction study involved digging trenches across selected parts of the site to look for features indicating historic liquefaction.

As seen from the Christchurch earthquake, liquefaction often causes ejecta to travel up from the liquefied layer, through cracks in the ground to escape on the surface. The locations where these features were found along the Hamilton Expressway Section tied nicely with the locations identified as being liquefiable from the shear wave velocity-based assessment. On the basis of this study, it was concluded that the project team could adopt the liquefaction potential indicated by the shear wave velocity-based assessment.

## GROUND IMPROVEMENT AT STRUCTURES

The Specimen Design incorporated extensive ground improvement at the bridge structures. The process for the liquefaction triggering assessment described above, together with the adopted bridge structural forms allowed a significant reduction in quantity of ground improvements at the structures (as compared to the Specimen Design). Refinements to the vertical alignment and groundwater control measures further contributed to the reduction of ground improvement for the project.

In general, ground improvement was deemed not necessary at any of the structures located in cuts or the stream gully bridges. The exception was the Mangaharakeke Stream southern bridge where a row of driven piles below the northern abutment was required to stabilise the abutment slope.

Limited ground improvement was still required at all of the structures located in fill zones, although the extent and depth were significantly reduced from the specimen design. The ground improvement comprised of lattices using low strength overlapping concrete columns constructed with a continuous flight auger (CFA). Depth of the ground improvement was typically 5 to 6m.

The ground improvement blocks at the fill bridges extend two to three metres beyond the MSE wall block to support the higher stresses near the face of the MSE walls and promoted uniform load transfer / settlements over the full footprint of the block.

## STRUCTURES DESCRIPTION

The aim of the design was to achieve simple, robust, reliable, and seismically resilient structures.

Concrete structures have typically been used for all local road overbridges and short span expressway bridges. These comprise single / twin span standard precast pre-stressed Super-T girders. A long span steel structure was used for the Mangaonua Stream Bridge. The long span was used because of the expensive substructure caused from a very deep soft peat layer.

Construction details were standardised to achieve construction efficiency and economy and to provide a consistent and unified project design. Consistency in bridge barrier shape and urban design features has been used throughout the project.

All bridge superstructures are semi-integral with the abutments except the Mangaonua Stream Bridge (steel structure) and the Southern Interchange Cambridge Road on ramp, both which use expansion joints due to length or skew. This therefore eliminates the need for movement joints and the associated future maintenance on all but two bridges.

Foundations for the bridges in cut, the stream gully bridges, and the Southern Interchange bridges all used traditional bottom driven steel tube piles. The exception was Mangaonua (long steel bridge) which is located in deep liquefiable soils and required larger diameter bored piles for lateral resistance.

All single span bridges with abutments located on fill comprise spread footings supported by MSE (Mechanically Stabilised Earth) walls built with in-extendable steel reinforcement. These walls are detailed to allow movement under the seismic Design Event.

All bridges on the project were detailed to accommodate large post-earthquake permanent movements. It is expected that many will not require any intervention / require repairs following the Design Event earthquake. Post-earthquake reinstatement works for the fill bridges was eliminated or reduced, compared to more conventional piled solutions. This was due to the innovative bridge form which allows the abutments and structure above to accommodate more movement. The fill bridge form is considerably more resilient than a conventional piled solution and so will be able to accommodate displacements even larger than the expected movements. This resilience is a major advantage over the more conventional piled abutment solution and is discussed in detail below.

## Bridges in Fill – Resilient Floating Abutments

The review of ground conditions at the bridge sites in fill indicated deep soil zones that exhibited risk of liquefaction. Analysis indicated that the triggering of liquefaction typically occurred between the 1/250 and 1/500 AEP events for these bridges.

A soil-structure interaction analysis was carried out at each bridge location assuming no deep ground improvement. The results indicated that a reliable and robust solution appropriately detailed could developed at each site that achieved the performance requirements of both, the Project Minimum Requirements, and the Waka Kotahi Bridge Manual, with targeted and limited ground improvement.

These bridges use simply supported Super-T beams and are considered to be "locked-in" longitudinally with semi-integral abutments supported on spread footing abutment beams. All bridges located in fill have the bridge superstructure seated on free float pot bearings which are connected to abutment spread footings, in turn are seated on MSE walls. The girders are detailed as simply supported for better performance during the post seismic vertical and horizontal movements.

Longitudinal resistance is provided by passive resistance of the abutment via a back wall cast at the end of the girders. Free float pot bearings have been detailed to accommodate the anticipated post seismic movement of the MSE walls. Elastomeric bearings did not have sufficient lateral displacement capacity to be used.

Transverse concrete shear keys are provided at the abutments. The transverse seismic load is transferred from abutment beam to MSE wall via friction. (See Figures 3 and 4 below).

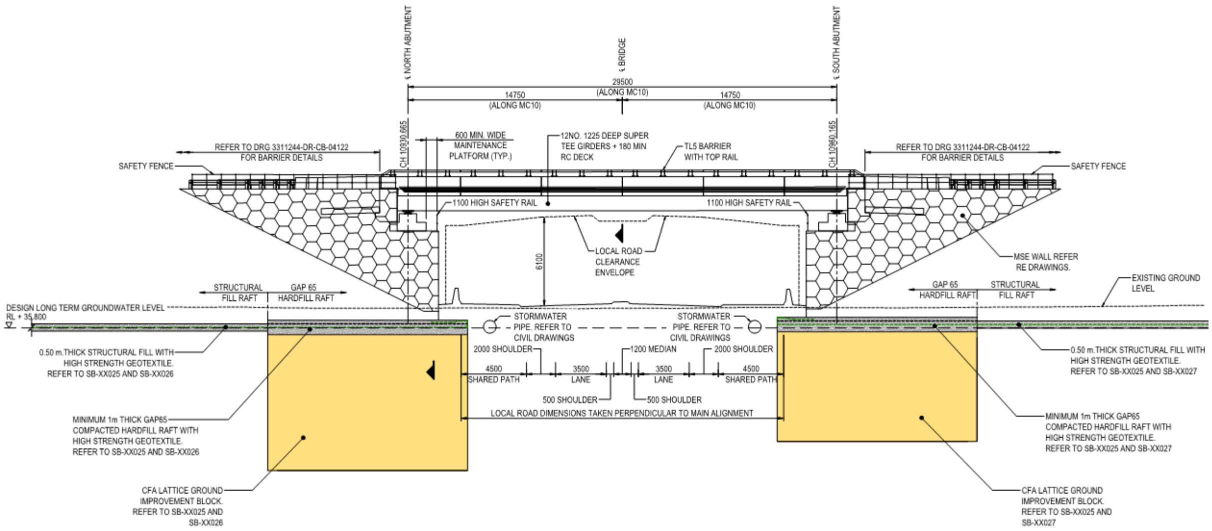


*Figure 3: Typical Bridge in Fill*

The MSE walls are considerably more resilient and more able to cope with the anticipated post seismic movements compared to a bridge supported on piles. Both the structure and the supporting MSE walls were detailed not only to accommodate the design seismic events, but to enable rapid and reliable reinstatement for traffic in the event of an earthquake. Hence, achieving the performance requirements of the bridge manual as summarised below:

- Emergency traffic able to use the bridge immediately after the seismic Design Event.
- After remediation / repairs, bridge is open to all traffic.
- Usable by emergency traffic after repair if subjected to a seismic Major Event.

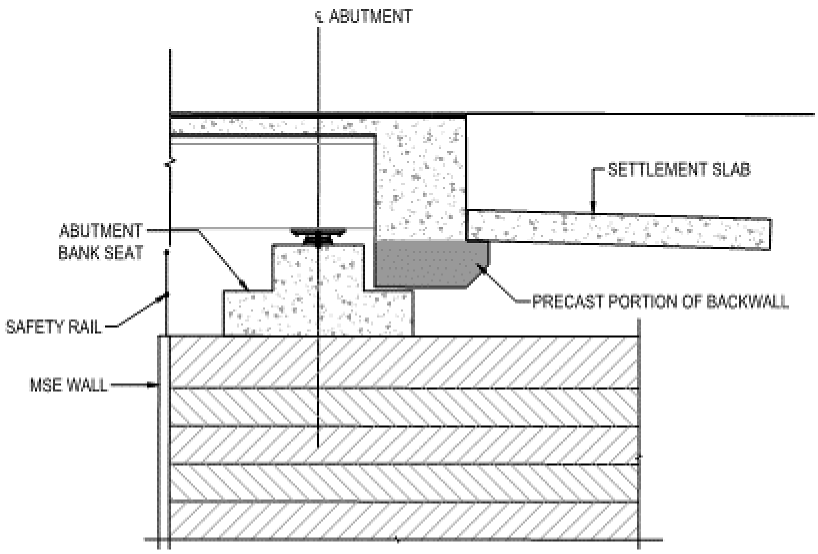


*Figure 4: Floating Abutment Detail*

The fill bridges meet the performance requirements of Table 5.1 of the Waka Kotahi Bridge Manual by specific details described below:

- The bridge backwalls were designed for full seismic passive pressures using upper bound soil properties.
- Using free float pot bearings to allow for large post seismic horizontal embankment movements whilst still being able to support full dead load with emergency traffic.
- The bridge detailing allowed for vertical jacking of the bridge back to original levels in the event of the MSE walls settling under liquefaction.

The global stability analyses indicated that the fill bridges will experience horizontal displacements of up to 300mm in a seismic Design Event and would require little or no intervention prior to reopening of the bridge to traffic. The bridge MSE walls are located far enough back that there is no reduction in the required horizontal clearance at each bridge site.

It is important to note that with this design approach, both the bridge and approaches will move together in a more sympathetic fashion. Analysis indicated that settlements for both structure and the adjacent embankment will be of a similar order of magnitude, substantially reducing the extent and impact of differential settlement. This will significantly improve the post-earthquake performance and reduce the quantum and extent of reinstatement works. It is expected that only minor works will be required to reopen these bridges to traffic after a seismic event.

Vertical settlement of between 100 to 300mm may occur at either one or both abutments, requiring inspection and possible jacking and re-levelling of bridge bearings. The extent of re-levelling required will depend on the level of differential movement between the bridges and the adjacent approach embankments. The bridge superstructure has been assessed for 50% differential settlement across one abutment. Emergency traffic can access the bridge immediately, although it will be at limited speed.

In contrast, the more rigid approach using piled bridge foundations at these locations would have likely resulted in substantial differential settlements between approach embankments and bridge structures, requiring additional reinstatement work and potentially more detailed structural inspections to confirm structural adequacy of the piles. Hence, piled structures were typically avoided for bridges in fill.

The anticipated response of the floating abutment bridge and approach embankments with respect to the Major Seismic Event was also assessed and confirmed that the structural solution meets the “no collapse” criteria of the Bridge Manual. A key feature of this type of bridge form is its ability to withstand displacements that are even larger than those anticipated with little increase in the extent of repair.

Damage to these structures at the Major Earthquake event would be expected. Damage would likely include significant cracking of MSE wall panels, potentially beyond the point of repair. The bridge may still be suitable for traffic, depending on the nature and magnitude of settlement, although this would be subject to a significant amount of reinstatement work on the approach embankments.

As noted earlier all bridges will be designed and detailed to accommodate jacking for bearing replacement (as a maintenance function). This functionality also provides for re-levelling after abutment settlement (for both design and major events).

If piled bridges were adopted for all bridges, we would expect greater differential settlements between the bridge and adjacent embankment, potentially leading to a different damage profile. In the Major Event, damage to bridge piles is likely to be substantial, most likely leading to demolition of the structure, whereas the more resilient reinforced earth abutment solution is likely to be more usable, perhaps with a new precast concrete facing system applied.

Another principal benefit of the proposed MSE block abutments over other foundation options is that they substantially reduce the extent and depth of ground improvement at each abutment because larger post seismic movements can be accommodated by the structure when detailed appropriately. This significantly reduced cost and critical path programme during construction.

## Bridges in Cut - Piled Bridges

The bridges located in cut areas are constructed using the "top down" method i.e., construct the bridge first at existing ground level, then excavate the soil material underneath after. Therefore, these bridges are piled. As noted earlier, post seismic movements are expected to be much lower for these structures since the cut slopes were designed to limit global stability movements.

The cut geometry comprises one vertical to two horizontal cut slopes at the bridge locations. Longitudinal subsoil drains are provided at the base of the cuts to maintain a permanent lowered groundwater level.

These bridges use Super-T structures and are considered to be "locked-in" longitudinally with semi-integral abutments supported on bottom driven steel tube piles. Longitudinal resistance is provided by passive resistance of the abutment via a back wall cast at the end of the girders. Tall elastomeric bearings have been detailed to accommodate the anticipated movements required to develop passive resistance so negligible longitudinal load is carried by the abutment piles. Pier piles have been designed for the maximum longitudinal displacement which occurs as the abutment backwall develops passive resistance i.e., a displacement-based design approach.

Transversely the bridges were designed as limited ductile structures. Transverse concrete shear keys are provided at the abutments and the piers. The relatively short length of bridge means most of the transverse bridge loads are transferred directly to the bridge abutment piles. The piled piers are flexible relative to the abutments and the number of piles at each pier were determined from Ultimate Limit State (ULS) dead and live loads.

The typical multi span local road bridge piers consists of a reinforced concrete pier headstock beam with bottom driven steel tube piles extending up into the headstock. The exposed section of steel casing was considered non-structural, but corrosion protection was still applied to the exposed section of the steel tube piles for aesthetic reasons.

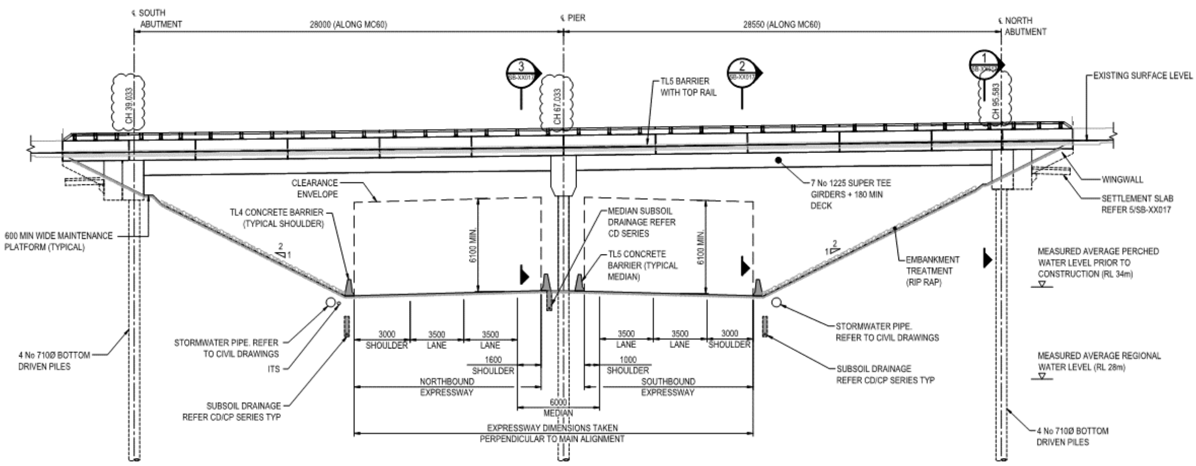


*Figure 5: Typical Bridge in Cut*

The abutment piles were required to accommodate relatively small (compared to the fill bridges) post seismic movements. These piles were assessed using Waka Kotahi R553 method which models a soil block pushing on the piles by imposing displacements on soil members as shown in the Figure 6 below.

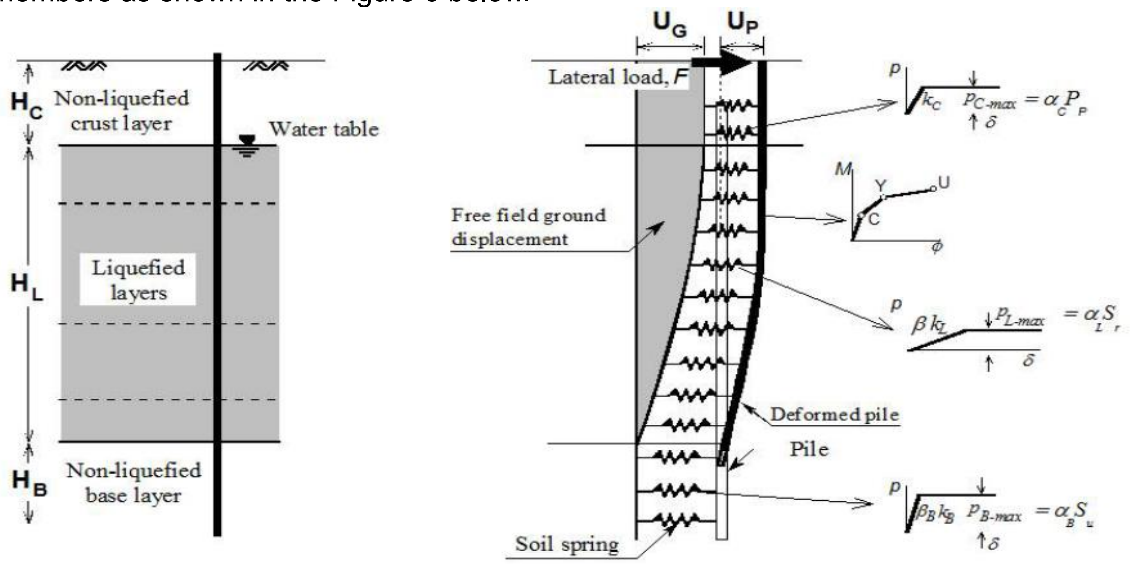


*Figure 6: Pile Design for post seismic embankment movement using R553 Method*

It should be noted that the seismic Design Event was not considered additive to the post seismic liquefied movements for ULS design since liquefaction induced embankment movements typically commences near the end of a seismic event.

All piles were detailed as fully ductile (mu=6) although allowable rotation curvatures in flexural plastic hinges were restricted to limited ductile limits for the two Design Event cases noted above.

In addition, the piles had to carry significant negative skin friction loads due to post seismic event settlements.

<u>Steel Bridge</u>

The Mangoanua Stream Bridge (see Figure 7 below) was designed as a three span 150m long steel ladder bridge due to the ground conditions in the gully. Maximum bridge span was 55m. Deep piles compounded by construction complexity of piling through, and aquifer meant that the substructure foundations were particularly expensive.

Four girders were used with crossbeams between the two pairs of girders at nominal 3.5m centres. Each girder was supported on a column linked together by a single long pile cap. Ten 1200 diameter piles per pier supported the bridge.

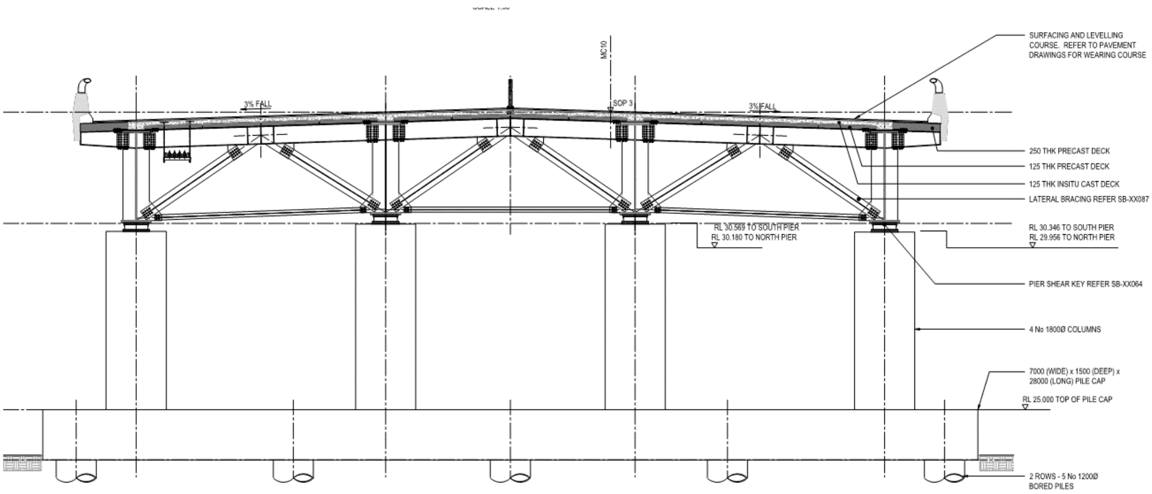


*Figure 7: Mangaonua Stream Bridge*

The soft ground conditions meant that girder erection was a critical load condition for the pile design. The large 600T crane placed lateral loads onto the piles which were considered to be locked in. The existing site was preloaded to remove a large proportion of the settlement and lateral loading which were expected during girder erection.

Liquefaction of the bridge site was expected over the top. However, due to the geography, the effects of liquefaction were expected to only reduce the soil stiffness to near zero, but not place significant lateral loading onto the piles. The large diameter piles combined with the pile cap provided the required stiffness by portal action since the top layers of ground provided very little lateral resistance.

## CONCLUSION

The “floating abutment” solution for all bridges constructed in fill was to support the bridge on free float pot bearings seated on abutment spread footings located on Mechanically Stabilised Earth (MSE) foundations. The reinforced earth foundations are considerably more flexible and more able to cope with the anticipated post seismic ground movements compared to a bridge supported on piles. Both the structure and the supporting reinforced earth foundations were detailed not only to accommodate the design seismic events, but also enable rapid and reliable reinstatement for traffic after an earthquake event, achieving the performance requirements of the Waka Kotahi Bridge Manual.

The solution which allows the bridge abutments to move with the ground meant that costly ground improvement was significantly reduced, providing Waka Kotahi with a value for money solution while also improving the whole of life performance. The floating abutment solution is considerably more resilient than a conventional piled solution and so will be able to accommodate much larger than anticipated movements if required. This was a major advantage over the more conventional piled solution for this project.