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Upper Hutt City Council

Silverstream Bridge Seismic Strengthening

Final Report

July 2007

1. Executive Summary

1.1 Introduction

In February 2005 GHD Limited was engaged by Upper Hutt City Council (UHCC) to provide professional services to "strengthen the bridge against a seismic event and protect the Fergusson Drive approach against a flood event (as per Option B in the Council report included in Appendix A).

The Silverstream Bridge (B29/1) is located 50 m from State Highway 2 – Fergusson Drive Intersection at the south western end of Upper Hutt. It is carrying nearly 14,500 vehicles per day and is strategically important to Upper Hutt's roading network serving as a primary arterial route for one of the main access routes into Upper Hutt.

The bridge also has importance to the greater Wellington Region as the bridge carries the Wellington Regional Council's bulk water main that supplies over 45% of the total wholesale water supply for that region.

Construction records indicate that the bridge was built in 1937/1938 and is therefore approximately 66 years old. The existing bridge consists of eight 18.9 m spans each with four haunched cast insitu concrete beams and integral concrete deck. The bridge has two 3.35 m traffic lanes, a 1.2 m footpath on each side of the bridge and an overall width of 9.65 m. Inspection records indicate that the bridge is in reasonable condition for its age. A general arrangement drawing of the is included in Appendix B

A report considering the preliminary assessment of options was provided to UHCC in August 2005. This outlined the results of the seismic risk assessment process and outlined options (and costs) to mitigate the risk to acceptable levels.

This completion report reiterates some of this information for completeness and also outlines the final solution adopted, issues that needed to be overcome, the design codes adopted, any change in waterway / structural capacity and maintenance / inspection requirements.

1.4.2 Preliminary Investigation

The preliminary design confirmed that the possible solution noted in a report prepared by others in 2000, i.e. installing piles adjacent to the existing piers and tying back to the bridge pier caps, would be beyond the budget currently allocated for the project and unlikely to be very effective.

The analysis confirmed that the bridge is at serious risk under seismic loading in the longitudinal direction while it would respond satisfactorily for a transverse seismic event (remain serviceable after a 450 year event). While this is less than the requirement stated in the request for tender, i.e. "Wellington fault event" it was accepted by Council as satisfactory given the likely residual life of the bridge and large cost of strengthening for achieve additional capacity.

To protect against the critical longitudinal event options considered included restraining the bridge longitudinally via "deadman" at each abutment and improving the ground behind each abutment. The restraining option was recommended for further development and detailed design.

1.4.3 Final Design

After assessment of various restraining options to carry the longitudinal seismic load a final "best fit for the site" option was recommended to and accepted by, UHCC.

This options utilised post tensioned high strength bars running diagonally from deck level (at the pier and abutment diaphragms) to ground level, where they transfer load to rock anchors.

The post tensioning in the high strength bars was designed based on analysis of the structure / ground interaction.

Detailing of the bars and rock anchors was carried out in such a manner as to minimise long term maintenance by proving double corrosion protection and also enclosing the group of bars to reduce the likelihood of debris catching them in a flood event.

2. Introduction and Background

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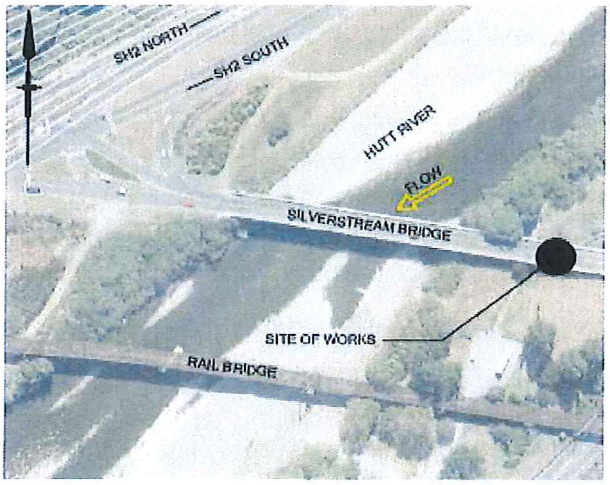
3. Description of Bridge

3.1 Structure

Construction records indicate that the bridge was built in 1937/1938 and is therefore approximately 66 years old. The existing bridge consists of eight 18.9 m spans each with four haunched cast insitu concrete beams and integral concrete deck. The bridge has two 3.35 m traffic lanes, a 1.2 m footpath on each side of the bridge and an overall width of 9.65 m. Inspection records indicate that the bridge is in reasonable condition for its age. A general arrangement drawing of the is included in Appendix B



3.2 Residual Life and Plans for Modification

The Council report indicated that the bridge would likely be able to reach its design life of 100 years given its current condition and existing maintenance practices. This gives the bridge as residual life of 33 years. The report notes that the bridge will likely reach its current traffic capacity within 20 years. A decision to replace or widen the bridge will therefore need to be planned in relation to that event. A maximum residual life has therefore been assumed to the purpose of this exercise.

4. Scope

The scope if the project is detailed in the professional services contract. Key aspects of UHCC requirements are included in the following bullet points along with key assumptions included with the GHD offer of service.

- To provide professional services to "strengthen the bridge against a seismic event and protect the Fergusson Drive approach against a flood event (as per Option B in the Council report included in Appendix A).
- Collect additional sub soil data at the site (if required).
- Greater Wellington will be replacing their water pipeline in association with the seismic strengthening of the bridge. The consultant will liase with GWRC through the professional services contract. This has not been carried out to date.
- The strengthened bridge and pipeline are to remain serviceable following a seismic event of at least that of a Wellington Fault Movement Event earthquake. Unless a fault displacement at the site causes direct damage.
- The bridge shall remain trafficable during the physical works contract except as outlined in the contract.

The offer of service submitted by GHD included the following conditions:

- The proposed solution for strengthening the bridge (as outlined in the Council report) was to install additional piles adjacent to the existing bridge piers. Council also noted their preliminary budget for the seismic strengthening work was $980,000 (derived in 1999). GHD noted that installation of additional pile was unlikely to be the most efficient means of strengthening the bridge and would likely to cost far in excess of the budget noted. It was our intention to assess and cost other more cost effective options and associated risk exposure regimes. It was noted that detail design of such options would likely be more design intensive and this work would need to be carried out as an additional service.
- Collection of actual bridge material properties and site specific seismic would be carried out as additional services on request by UHCC.
- It is assumed that the design and physical works contracts for the GW pipeline and any seismic strengthening would be separate contracts.

5. Strengthening Philosophy

The strengthening design philosophy adopted for this project is as outlined in the Transit Bridge Manual, i.e. if the behaviour is satisfactory at the design intensity (a), it is expected with appropriate detailing to behave satisfactorily in design scenarios (b) and (c) (as summarised below).

a) After the design return period event, the bridge shall be useable by emergency traffic, although damage may have occurred, and some temporary repairs may be required. Permanent repair to reinstate the design capacities for both vehicle and seismic loading should be feasible.

b) After an event with return period significantly less than the design value, damage should be minor, and there should be no disruption to traffic.

c) After an event with return period significantly greater than the design value, the bridge should not collapse, although damage may be extensive. It should be useable by emergency traffic after temporary repairs and should be capable or permanent repair or a lower level of loading may be acceptable.

As noted in the project brief, due to the close proximity of the bridge to the Wellington Fault and the possibility that the main fault (or at least secondary faults) pass under the bridge "it not expected that the bridge or pipeline will accommodate the horizontal and vertical displacement at the fault at the site".

It should be noted that given age of the bridge existing detailing will not be in accordance with current standards and hence it will not provide the same level of ductility as would be provided by details suggested by current standards. Local damage to the bridge, i.e. spalling etc due to repeated shaking cycles will therefore be greater at plastic zones etc, than for new structures. The basic premise of the performance scenarios noted above can still be complied with however.

In this instance the level of strengthening (if any) required to comply with (a) above is essentially related to either / or :

- The maximum longitudinal displacement of the bridge as a result of the seismic event, i.e. as a result of the abutments progressively moving as passive soil resistance is exceeded by seismic inertia. If the displacement is excessive there is potential for catastrophic failure mechanism or irreparable damage to the structure to result.
- Inadequate shear capacities of seismic resisting members during the formation of plastic hinges subject to repeated cycles of shaking resulting in catastrophic shear failure.

The design philosophy for this project will therefore be to:

- Ensure passive resistance provided by strengthening piles and abutments is not exceeded by the design earthquake or where this is exceeded only feasibly repairable displacements of the structure will result.

- Plastic hinges critical to providing seismic resistance of the bridge are able to form at appropriate locations with adequate shear capacity to ensure these are able to develop.
- In the longitudinal direction the bridge is essentially a series of portal frames and therefore very stiff and with limited ductility. The final strengthening option will need to accommodate these constraints.
- Analysis confirmed that the bridge is at serious risk under seismic loading in the longitudinal direction while it would respond satisfactorily for a transverse seismic event (remain serviceable after a 450 year event). While this is less than the requirement stated in the request for tender, i.e. "Wellington fault event" it was accepted by Council as satisfactory given the likely residual life of the bridge and large cost of strengthening for achieve additional capacity.

6. Methodology

6.1 Concept / Preliminary Design

6.1.1 Seismology

In 2002 and 2003 the Water Group of Wellington Regional Council (now Greater Wellington) commissioned the Institute of Geological and Nuclear Sciences (GNS) to report on the likely location of the Wellington Fault in the vicinity of the Silverstream Bridge. The conclusions of the report state:

"The location of the Wellington fault at the Silverstream Bridge is known to within approximately 60 m. On the basis of the compiled data, the fault appears to be most likely located immediately to the west of the western end of the bridge. Some faulting, secondary to the Wellington Fault principal displacement zone, may pass under the bridge".

GNS (Discussion McVerry / Sneddon June 05) recommended that the recent seismic response spectra derived for near fault event as outlined in NZS 4203 would be appropriate for this study. It was indicated that a site specific seismic survey would provide minimal additional accuracy.

The site subsoil category as defined in the Transit Bridge Manual is category (a) (rock or very stiff soil site – less than 25 m of dense sandy gravel with N> 30).

6.1.2 Analysis

The preliminary investigation took a fresh approach to the seismic risk issue. The key aspects of the approach included:

- A desk top review of previous geotechnical test information at the site and preparation of a brief to collect additional information (including information required GW in relation to the location of the fault).
- Letting a contract to collect this additional site specific subsoil information, i.e. thickness and SPT values for overlying gravel layers at each abutment. GW were advised of rates received from the Drilling contractor to carry out diagonal drilling to collect additional information in relation to the location of the fault but these were declined on the basis they were significantly higher than expected.
- Development of a 3 D computer model of the existing structure including its interaction with surrounding soil (based on as-built records of the bridge and testing / inspection carried out as a result of this project). No springs were assumed over the top one metre of each pile.
- Consider from a fresh perspective the performance of the existing bridge in the design event based on available spectra and material properties. (Note equotip testing of a representative sample of bridge reinforcement has been recommended and accepted by UHCC). This be carried out over the next month and incorporated into the detailed design phase.

- The hierarchy of failure of the existing bridge in seismic event was assessed (with corresponding deflections and associated return periods noted. An assessment is made to whether performance of the existing bridge is adequate or not.
- Specific details of the existing structure were reviewed and assessed whether strengthening should be recommended to ensure performance of the structure as noted in 3.1 above.
- Develop various strengthening options and assess viability and preliminary cost estimates to enable recommendation to be made to UHCC.

6.2 Detailed Assessment of Options and Final Detailed Design

The focus of the final design was on options to strengthen the bridge in the longitudinal direction given that analysis confirmed the transverse response of the bridge was acceptable given its likely residual life and cost of strengthening for this scenario.

Once the decision to strengthening the bridge by restraining it longitudinally had been made (rather than by, improving the surrounding ground, installing additional piles and tying back at deck level or locally strengthening piles / abutment) options to achieve this were considered. In this process the site constraints and advantages were carefully considered.

These include:

- Numerous services are located both on the approaches to the bridge and the bridge itself (including 1000 m diameter pipe carrying 45% of the regions water supply), meaning that work behind the abutments would be very difficult.
- High traffic volumes over the bridge meant any road closures had to be minimal.
- Highly fractured rock beneath the west abutment and close proximity of the Wellington fault (circa 50 m) further limited restraining options at this location.
- Being adjacent to the Hutt River that contains a significant trout population (resource consent is required for work in the waterway over extended periods of the year during the trout spawning periods).
- The bridge structure itself is effectively a series of interconnected portal frames and therefore very stiff in the longitudinal direction with limited ductility.
- The bridge contained a construction joint at the central pier and the resulting seismic response and design implications needed to be considered.
- The river is subject to severe flood events and the solution needed to consider the impact of this both during construction and long term.

At its most basic level seismically strengthening the bridge involves providing an alternative / additional route for the seismic inertia to transfer to ground level. Typically this is achieved by improving the ductility of the supporting structural components, i.e. steel or fibre glass wrap, or retraining the abutments via, "deadmen", friction slabs, ground improvement (grouting etc) all of which require significant work behind the abutments.

To minimise disruption to traffic, services and close proximity of the fault, a restraining mechanism beneath the bridge itself (rather than beneath the approaches) was investigated. To overcome the natural stiffness of the bridge in the longitudinal direction however, the transfer of the seismic inertia force needed to be very direct and solutions such as additional piles (acting in cantilever mode) would need to be very large to attract the load required.

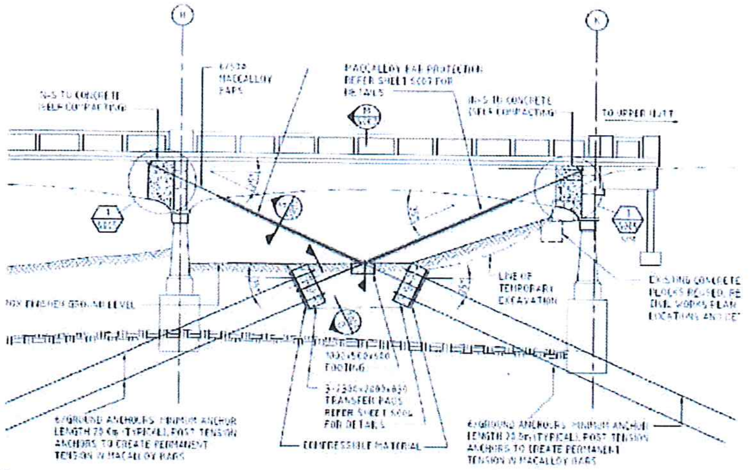
Different innovative solutions were therefore required to overcome such significant site restraints.

On the positive side it was fortunate that the rock level towards the eastern end of the bridge was relatively close to the surface (circa 5 – 7 m). This made the idea of utilizing rock anchors to transfer the load in tension to the underlying rock quite attractive. The terrain under this span also provided easy access to the underside of the bridge and was well away from the river waterway. The challenge was therefore how to use these particular advantages in any solution.

Further issues to be addressed to take advantage of these benefits include:

- If rock anchors were adopted how would the seismic load be transferred from deck level to ground level (truss or shear wall mechanisms were postulated)?
- Any strengthening members would need to be very stiff to attract the required seismic load?
- How to mitigate the effect of exposed members on water flow in a flood event? Consents from regional authorities would be required and could be difficult to obtain as the bridge was a known flood constraint on the River. This was a risk to the entire project that needed to be carefully mitigated. To manage this risk we actively promoted good communication and consultation with all parties throughout the project to ensure "buy in".

Structural analysis indicated that post-tensioning high strength bars (running diagonally between diaphragms to the rock anchor stressing block at ground level) would provide adequate stiffness to these members to attract the required seismic load. The amount of post tensioning was assessed by careful analysis of the structure / strengthening / ground interaction.



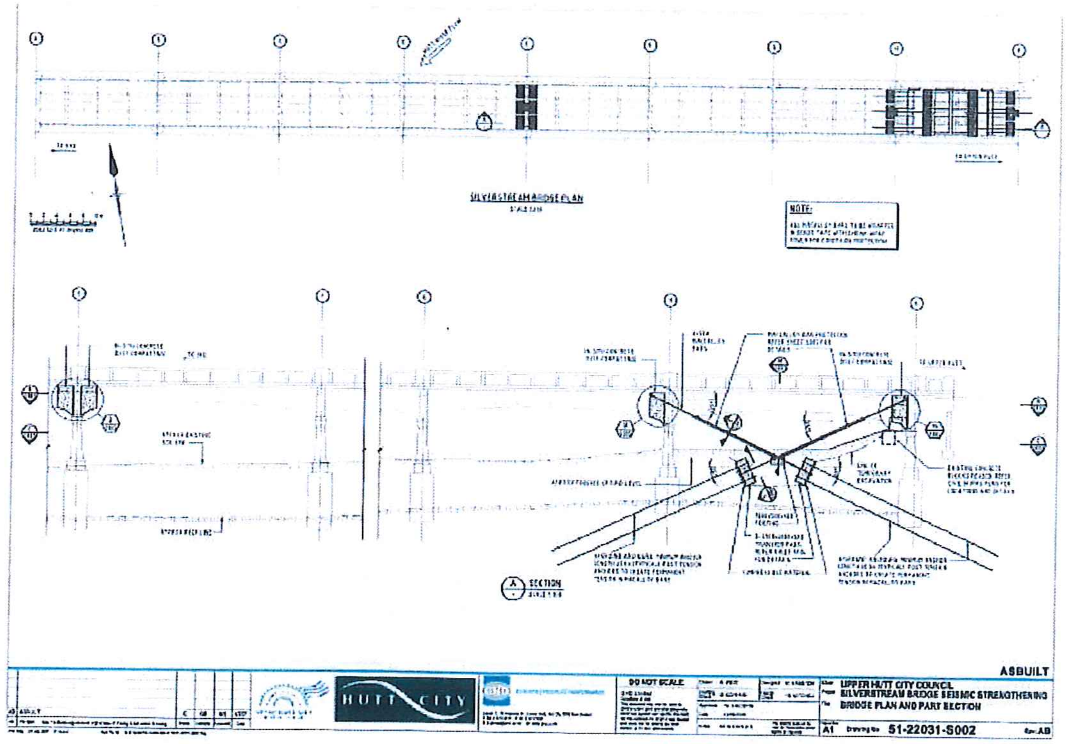
Significant strengthening of the diaphragms was required where these were attached to the diagonal members. The ease of access to the underside of the bridge at the chosen location made this task relatively straightforward.

To mitigate the impact of the prestressed bars on water flow during a flood event and to prove to the Regional Council the appropriateness of the solution, the hydraulic model of the Hutt River at the site was modified and rerun. The effect of different debris "blockage" scenarios (within the strengthened span) were considered by modelling and the final design incorporated a lowered ground level (within the end span) to ensure the effect of the exposed bars and debris would be negligible. In addition the bars were enclosed in continuous shield to minimise the possibility of debris "catching" between rows and compounding the blockage. Final agreement to the proposal was obtained from Regional Council after a meeting on site to discuss the issue and solution while viewing the actual area / site it would be located.

Prestressed bars were provided with double protection against corrosion.

A specimen design was provided for the rock anchors to give contractors the opportunity to provide alternatives should they foresee cost advantages in doing so.

To overcome the construction joint at midspan it proposed that the bridge be tied together across this joint so the bridge would seismically respond as one structure. The linkage bars would be detailed to allow normal temperature expansion and contraction to take place.



Self-compacting concrete was used to strengthen the diaphragms and abutment where the post tension bars were required and access for pouring and compacting was difficult. This was a great success and ensured the integrity of the final product in place.

6.2.1 Design Standards

The bridge strengthening was designed in accordance with the Transit New Zealand Bridge Manual (and all the standards referred to therein) and relevant UHCC standards.

6.3 Effect of Strengthening on the Load Carrying Capacity and Waterway Capacity of the bridge

Both the load carrying capacity and waterway capacity of the bridge remain unchanged by the incorporation of the seismic strengthening.

6.4 Resource Consent

A resource consent was required for the physic works as the adopted solution could potentially act as a restriction to the flow in a flood event as outlined above.

Modelling of the river in flood and various debris blockage scenarios were considered to support the resource consent application with the final solution detailed to ensure the effect on water level in a flood event was negligible.

7. Construction and Maintenance

7.1 Construction

Construction was carried out over the late summer and winter of 2006 and finished early in 2007. Progress was slower than originally envisaged due to numerous southerly storms that hit the region over that period and also some difficulties in drilling the initially rock anchors.

Practical completion was issued in 26 January 2007.

7.2 Maintenance

The strengthening elements have been designed to maintenance free over the next 25 years. All steel members are double corrosion protected and shielded to minimise debris blockage and vandalism.

All concrete members have been detailed to provide a 100 year life.