The Strengthening of the Norbert Street Cable-Stayed Footbridge, Hutt Valley, New Zealand

Duncan Peters BSc(Eng) M.IPENZ Registered Engineer
Senior Associate, Connell Wagner
and
Timothy Brook BE (Hons)
Engineer, Connell Wagner

SYNOPSIS

The Norbert Street footbridge is a 106m long cable-stayed bridge crossing the Hutt River near Wellington, New Zealand. Soon after its construction, concern was expressed about the bridge's lively response to pedestrian and wind loading. In February 1998 one of six cables tying the bridge pylon back to the west abutment pulled out of its anchor socket placing the bridge in danger of collapse. An independent structural analysis revealed the need to strengthen and stiffen the bridge. As a result remedial measures including replacement of the cables, stiffening of the deck and strengthening of the pylon foundations, were undertaken.

1 INTRODUCTION

The Norbert Street footbridge is a cable-stayed bridge crossing the Hutt River near Wellington, New Zealand. The bridge provides pedestrian and emergency vehicle access across the river and carries a 225 mm diameter water main on the upstream side of the deck.

The bridge was tendered on a design and construct basis by Upper Hutt City Council. The Council sought an aesthetically pleasing solution. Tenderers were advised that the Council had available wire rope and sockets which they could use if they wished. The successful tenderer proffered a cable-stayed bridge which was aesthetically pleasing and utilised the wire rope and sockets in the Council's possession. Construction of the bridge was completed at the end of 1993.

2 DESCRIPTION OF THE BRIDGE

The cable-stayed footbridge has an overall length of 106m. The deck is made up of six prefabricated units of between 17.4m and 21.3m in length. Each unit consists of a ribbed concrete deck slab with a light steel truss on either side. The bottom chords of the trusses are cast into the deck slab. The deck units are connected by pairs of steel angles bolted to the outside edges of the deck slab. This arrangement effectively provides rigid connections between units in the horizontal plane and pinned connections in the vertical plane.

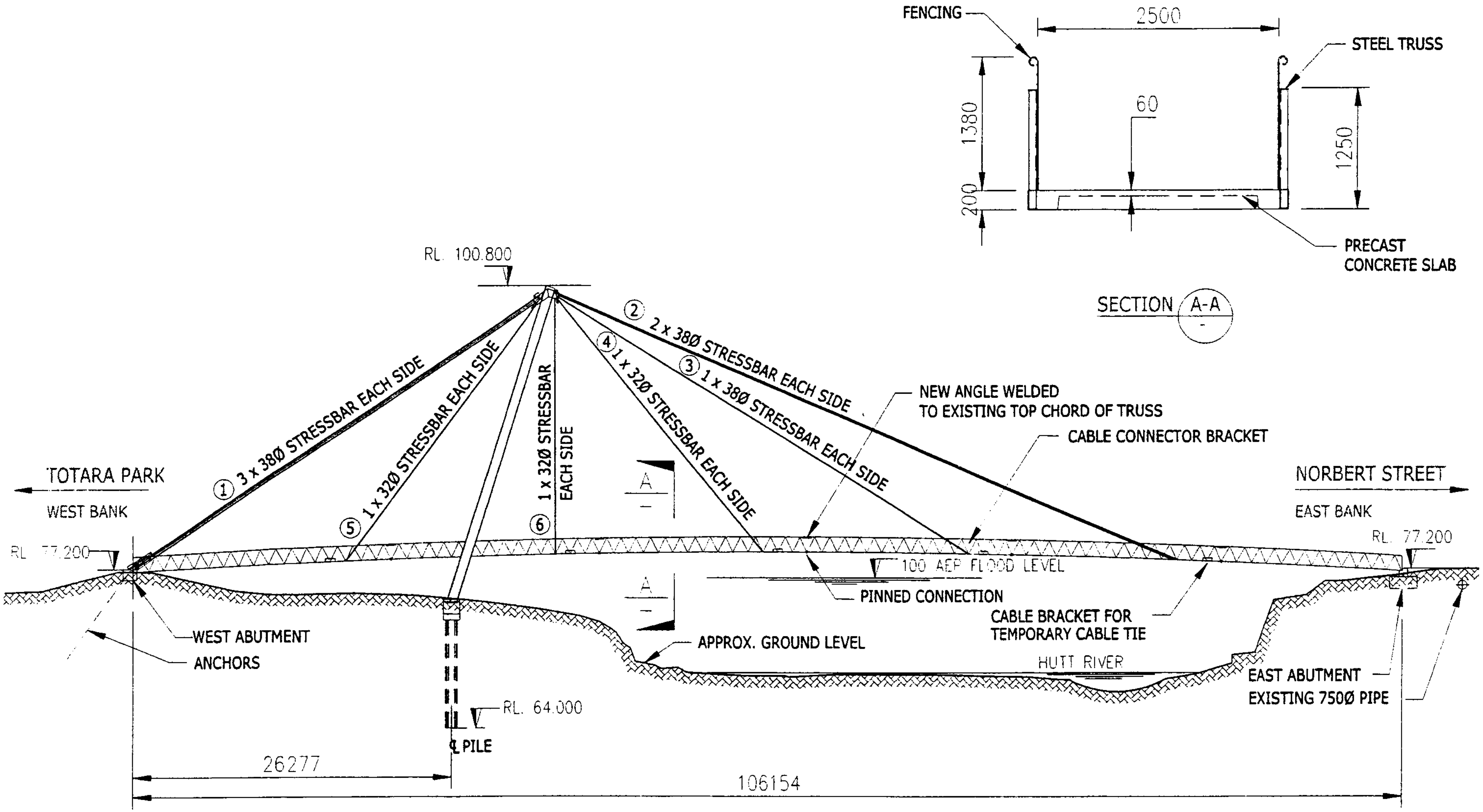


Figure 1 *Elevation Of Strengthened Norbert Street Bridge*

The deck is supported by an asymmetric radial arrangement of cables attached to the apex of an inclined A-shaped pylon (Figure 1). The pylon is tied back to the western abutment by two sets of three cables. Each original cable comprised a single 24mm or 28mm diameter galvanised steel wire rope. The ropes were anchored by placing their splayed out ends into drop forged sockets and filling the sockets with molten white metal. (White metal comprises 80% lead, 15% antimony and 5% tin). Apparently one of the anchorages was load tested during the bridge's construction.

The pair of precast prestressed concrete pylon legs are bolted together at the apex and supported on a reinforced concrete hammerhead at the top of a bored cast in-situ concrete pile (Figure 2).

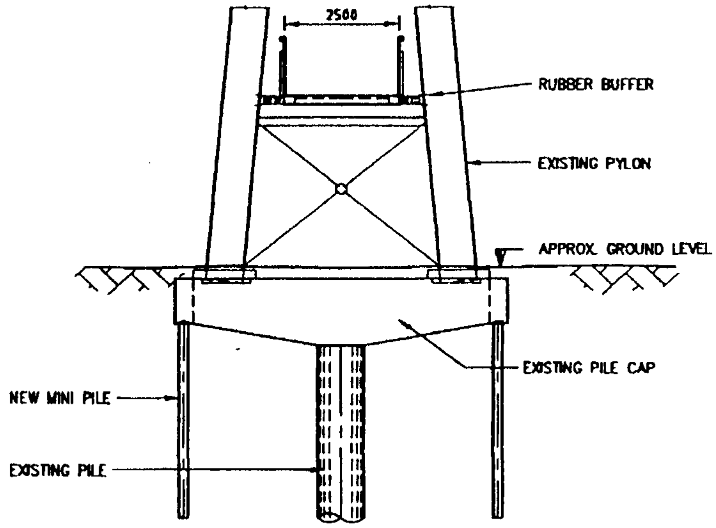


Figure 2: Elevation of Pylon

The western abutment is anchored to the ground with two Titan injection anchors. The injection anchors comprise 73mm diameter continuously threaded tubes which were installed in boreholes drilled into the underlying dense alluvial gravels and grouted in with cementitious grout. The length of the anchors was not recorded but apparently one of them was tested to twice its working load. The eastern abutment consists of a simple pad footing which provides vertical support and horizontal restraint to the bridge deck.

3 CABLE FAILURES

Soon after its construction concern was expressed about the bridge's lively response to pedestrian and wind loading. In February 1998, four years after the bridge was built, one of the six cables tying the bridge pylon back to the west abutment pulled out of its top socket. The cable was re-attached to the top socket and re-stressed. Shortly afterwards the designer of the bridge recommended to the Council that the six cables between the pylon and the western end of the bridge be replaced with Stressbar of greater strength than the original cables. Stressbar is a cold-worked high-tensile threaded steel bar with a nominal tensile strength of 1080 MPa.

Regular measurements of the amount of white metal extruding from the sockets anchoring the pylon cables to the western abutment during the three months prior to the award of the remedial works contract in June 1998 indicated that the cables were slowly pulling out of their sockets. The bridge was closed to the public and the area surrounding it was cordoned off to safeguard passers-by in the event of the bridge collapsing. Two weeks after the award of the remedial works contract, emergency work was carried out to stabilise the bridge after a partial socket pull out of the lower downstream cable led to significant deformation of the bridge deck.

4 REVIEW OF THE BRIDGE

In mid-March 1998 Connell Wagner were appointed by the Council to carry out an independent investigation into the causes of the Norbert Street Bridge cable failure. They were subsequently instructed to review the design of the bridge and recommend remedial measures.

4.1 Bridge Inspection

An inspection of the bridge revealed a number of areas of concern. These are described below.

4.1.1 Cable Anchorages

White metal extruded about 20 mm from the anchor sockets of the upper and lower upstream cables tying the pylon back to the western abutment. The length of extruded white metal indicated the amount by which the cables had pulled out of their sockets. The middle cable was a replacement to the original one that had unscrewed itself and sprung free during the construction of the bridge.

On the downstream side white metal extruded about 20 mm from the sockets of the middle and lower cables tying the pylon to the western abutment. The middle cable had two kinks along its length. This was the cable that was re-installed after its upper anchorage failed in late February 1998.

The steel plates attaching the cable sockets to the deck were narrower than the gap between the socket jaws in some instances (Figure 3). The absence of spacers between the plates and socket jaws gave rise to eccentric loading which could accelerate fatigue failure of the connections.

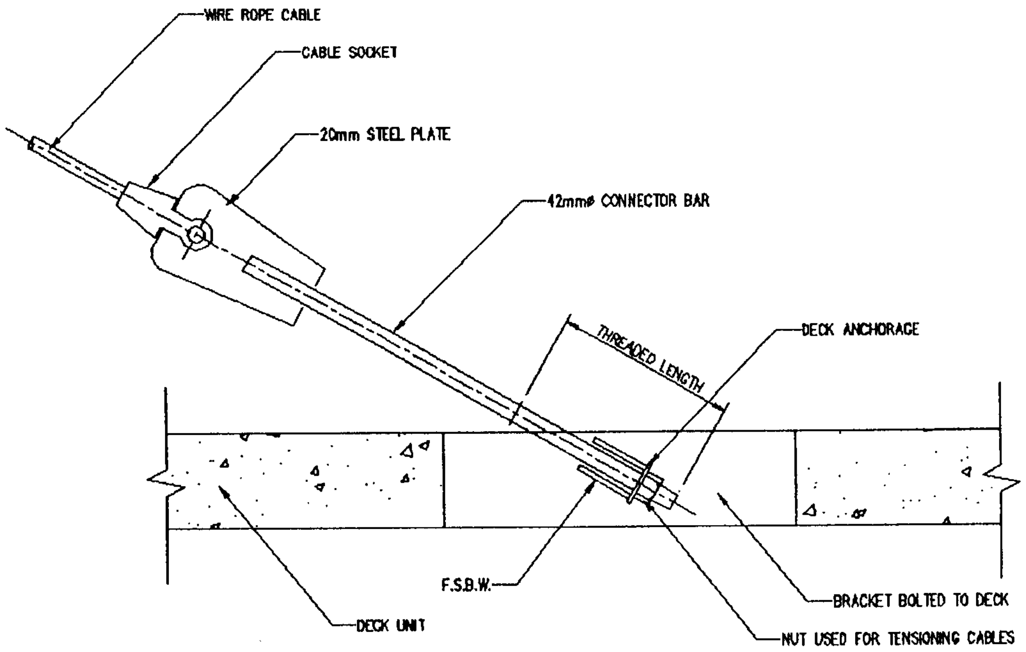


Figure 3: Elevation of Original Deck Anchorage

The additional pair of cables at the eastern end of the bridge was attached to brackets which were connected to the deck by three steel straps across the soffit of the deck. The brackets appeared to be flexing away from the deck.

4.1.2 Deck

During the site inspection the deck was observed to oscillate vertically under moderate gusts of wind. The maximum movement was in the order of 50 mm and occurred at the joint between the two easternmost deck units. The period of the oscillation was about two seconds.

There were downward kinks in the vertical alignment of the deck at the first joint between deck units from the western abutment, and at the first and second joints in the deck from the eastern abutment. The downward kink at the first joint from the eastern abutment was particularly pronounced and the steel angles joining the last two deck units appeared to be flexing away from the deck concrete. The opening up of the connection would be exacerbated by the oscillation due to wind loading.

At the eastern end of the bridge the deck tilted downwards on the upstream side where the water main is attached.

The downward kinks in the deck are probably due to there being insufficient length of threaded bar in the cable connectors to bring the deck units up to level during erection (Figure 3). Extension tubes have been placed between the nuts used to tension the cables and the anchorages on the deck in an attempt to alleviate the problem.

4.2 Structural Analysis

4.2.1 Structural Model

An analysis of the bridge was carried out using the as-built drawings as the basis for the geometry and section properties. A three dimensional model was used as this was essential for determining the complex response of the structure, particularly under wind loading, seismic loading and eccentric loadings on the bridge deck. The model consisted of approximately 60 joints and 70 members. The cables were modelled with cable elements to account for the reduction in their stiffness due to sag. The deck was modelled by a line of 3-D beam elements along the centroid of the bridge deck. The bending, shear and torsional stiffnesses of the beam elements were obtained from a detailed finite element model of a representative section of deck. Pile-soil interaction was represented by using equivalent springs to model the soil subgrade modulus.

4.2.2 Design Loadings

The loadings used were as follows:

- The dead load of the structure including the eccentric load from the water main.
- The specified live load of 2.0 kPa or an emergency vehicle axle loading. (The live load is low compared to most bridge codes which typically specify about 5.0 kPa)
- Wind loading, based on a wind velocity of 50 m/s, in the transverse horizontal and vertical directions modeled as static loads.
- Seismic loading based on the response spectrum for an elastically responding structure founded in deep soil. The complexity of the dynamic response of the bridge could be seen upon investigation of the modal analysis. It was necessary to include the first 16 modes of vibration before the effective mass included in the results approached the recommended 90% of the total mass of the structure. As expected, the fundamental mode of vibration for the bridge was found to be the lateral oscillation of the bridge deck between the abutments. However, the effective mass of this mode of vibration only accounted for 55% of the total mass of the structure. Some of the higher frequency modes of vibration were of significance, such as the fourth mode, which consisted of oscillation of the bridge deck and pylon along the longitudinal axis of the bridge.

4.2.3 Findings

A summary of the findings from the structural analysis are presented below.

4.2.3.1 Deflections

The maximum deflection under vertical loading occurred at the eastern most hinge in the deck. The vertical deflection at this point under live load was 585 mm and under vertical wind loading it was 360 mm. Both of these deflection values are excessive when compared to the 150 mm allowable deflection given in the ASCE Recommendations (Reference 12).

The steel angle connections linking the deck units could not accommodate the hinge rotations under transient loads.

4.2.3.2 Cable and Anchorage Forces

The forces in the cables connecting the top of the pylon to the western abutment are 0.55 of their breaking strength, fpu under dead load and 0.87 fpu under dead and live load. Elsewhere the maximum forces under dead and live load range from 0.32 fpu to 0.49 fpu. The recommended maximum permissible force in the cables under dead and live loads for steel wire rope in cable stayed bridges is 0.33 fpu (Reference 12). It should be noted that this assumes that the cable anchorages contain zinc which is normal practice for bridges and not white metal (References 10 and 11).

The loads in all the cable anchorage sockets greatly exceed the maximum permissible loads given in BS 463:1970 (Reference 8). The BS adopts a factor of safety of 5.0 for working loads. Under the design dead load and live load combination the anchorage loads for the pylon tie back cables are four times the permissible load and they are approximately twice the permissible load for the other cables.

The permissible anchorage loads are lower than the recommended permissible cable loads and they therefore restrict the maximum loads that can be permitted in the cables.

The cable failures in February and June 1998 may be attributed to inadequate cable socket capacity exacerbated by large load fluctuations due to vertical wind load.

4.2.3.3 Deck Bending Moments and Shear Forces

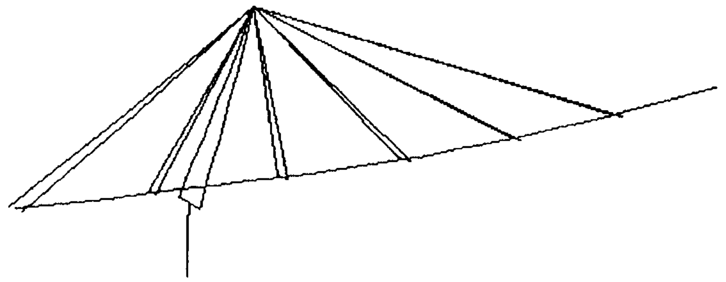
The lateral bending capacity of the bolted connections between the deck units was found to be inadequate under transverse seismic and wind loading.

4.2.3.4 Pylon Pile Capacity

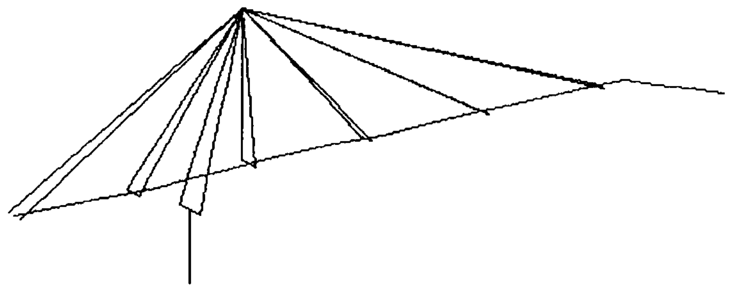
The bending moment capacity of the pile supporting the pylon was insufficient to resist transverse seismic and wind loading however the pile had adequate shear force capacity.

4.2.3.5 Dynamic Behaviour of Deck under Wind Loading

The vertical oscillation of the bridge deck was represented by the second mode of vibration of the bridge (Figure 4). The computed period was 1.8 seconds which was close to the observed period of oscillation in moderate wind. The New Zealand loading code for buildings (2) recommends that for design, dynamic wind analysis be undertaken for structures with a fundamental period exceeding 1 second.



(a) Mode No.1: Lateral Sway of Deck T = 2.3 s



(b) Mode No.2: Vertical Oscillation of Deck T = 1.8 s

Figure 4: Primary Dynamic Mode Shapes of Bridge Before Remedial Works

5 REMEDIAL MEASURES

Proposed remedial measures were incorporated in the bridge model and optimised by an iterative process.

The principal requirements were to strengthen the cables and their anchorages, stiffen the deck to reduce deflections under live and wind load and reduce the fundamental period of the deck to less than one second.

The following remedial measures were proposed:

- All cables to be replaced by 38mm and 32mm diameter Stressbar or equivalent. Additional temporary cables are required when replacing the cables tying the pylon to the western abutment. The diameter and grade of the shear pins in the cable connectors needed to be confirmed. Because the new cables have larger cross sections than the original ones they stiffen the deck considerably.
- The deck should be realigned to give a smooth profile. This is to be achieved by initially stressing the replacement cables to the computed force followed by re-stressing them to achieve a smooth profile.
- Deck stiffness to be improved by joining the gaps in the top chord of the steel trusses between each deck unit to make the deck continuous over its full length. The combination of the new cables and continuity of the deck will reduce the maximum deflection under live load to 150 mm. This is the recommended maximum deflection given in reference 12
- Two rubber buffers to be placed between the deck and the pylon legs to prevent lateral movement but permit longitudinal and vertical movements. The rubber buffers to be

attached to the pylon legs with steel plate connections. Once installed the transverse strength of the deck is adequate under seismic and wind loads.
- One pressure grouted minipile to be installed at either end of the hammerhead supporting the pylon legs and connected to the hammerhead.

The measures proposed above reduce the fundamental period of the structure to less than 1 second thus obviating the need for a dynamic wind analysis (Reference 2).

Two further measures were proposed to enhance the durability and provide a means to accurately monitor the bridge:

- The top and bottom surface of the deck to be cleaned and coated with a penetrative siloxane coating to limit the ingress of water into the concrete. This will improve the durability of the deck but will require re-application every few years.
- Two survey pins and two reference datum points to be established so that precision surveying can be undertaken on a regular basis to monitor the western abutment. This recommendation is based on the critical role of the Titan anchors in supporting the bridge.

6 CONSTRUCTION

As soon as sizes and quantities of Stressbars and anchorages for replacing the cables were known they were ordered by the Council in order not to delay the remedial works. The bars, couplers and clevises were coated with a zinc based metal spray and sealant prior to delivery to site. It should be noted that hot-dip galvanising is not a suitable method of treating the Stressbars against corrosion.

The contract for the remedial works was awarded at the end of June 1998 and the bridge was re-opened to the public two months later. Because of the risk of collapse the number of people that the Contractor was allowed to have on the bridge was limited to two prior to the replacement of the cables tying back the pylon. For safety reasons the Contractor's means of accessing the top of the pylon had to be independent from the bridge.

6.1 Cable Replacement

Prior to work commencing the bridge deck and the pylon apex were accurately surveyed. The survey was carried out within one hour of sunrise to minimise temperature effects.

Existing cables were replaced with either 38mm or 32 mm diameter Stressbars. The replacement of the cables was the major challenge for the Contractor. The bridge could not be supported from the riverbed because of the risk of floods and a detailed cable replacement procedure was specified to avoid overstressing any of the cables or inducing large deflections in the deck while cables were being replaced.

The replacement Stressbars for each cable were pre-assembled before being lifted into position. The couplings between lengths of new Stressbar were coated with Loctite prior to assembly to prevent them from working loose. The longest Stressbar cables were about 57m long and the Contractor had some difficulty in keeping them reasonably straight while lifting them into position. Each Stressbar was stressed at the bottom end with a centre hole jack.

The method for replacing the six pylon tie back cables was to install an additional pair of temporary Stressbars above the existing cables and tension them to a specified force. Thereafter the existing cables were destressed and replaced one at a time and tensioned to a specified force. Once the six cables were replaced the temporary Stressbars were removed and used elsewhere. The method adopted meant that the stress in the existing cables was not increased and that the replacement Stressbars ended up equally stressed.

The method for replacing each of the remaining cables was aimed at providing continuous support to the deck at each cable position and eliminating excessive deflection of the deck at any time. First a temporary Stressbar was installed immediately above the cable to be replaced. The temporary Stressbar was then stressed to the lesser of one fifth of the force in the existing cable or the force causing a 50mm lift in the deck. Thereafter the existing cable was destressed until the deck settled to its original level. The process was repeated until the existing cable could be removed and the replacement Stressbar installed. Using the same procedure, the replacement Stressbar was then stressed in increments alternating with the destressing of the temporary Stressbar cable. On completion of the cable replacement the stressing jack was used to bring the deck back to its original position.

6.2 Profiling the Deck

After replacement of all the cables the deck was resurveyed and the deck levels readjusted to give a smooth curve based on a cubic function. The curve adopted was such that existing levels did not have to be excessively modified since this could have risked damaging the deck joints. The downward kinks in the deck were corrected and the crossfall tilt due to the increased weight of the eccentrically mounted water pipe was eliminated. This required lifting the deck by up to 125mm on the upstream (watermain) side and 63mm on the downstream side.

The jacking forces in the cables were monitored as work proceeded. The following forces in kN were recorded on completion of the deck trimming operations:

CABLE No.	UPSTREAM			DOWNSTREAM		
	Actual	Theoretical	Difference	Actual	Theoretical	Difference
6	85	80	+6%	78	65	+20%
5	124	110	+13%	87	100	-13%
4	93	110	-15%	85	90	-6%
3	196	150	+31%	147	130	+13%
2 (Top)	178	155	+15%	133	130	+2%
2 (Bottom)	151	155	-3%	120	130	-8%
TOTAL	**827**	**760**	**+9%**	**650**	**645**	**+1%**

The total actual downstream cable force was close to that calculated, however the upstream is 9% higher than anticipated. This may be because the services supported under the bridge were heavier than allowed for in the analysis. Individual measured bar forces varied markedly from the theoretical values. This may in part be due to the hinges between deck units not behaving as truly pinned connections.

6.3 Stiffening the Deck

Once the deck had been profiled it was made continuous by welding in steel angles to close the gaps in the top chord of the deck truss at the joints between the deck units. This work was done on calm days within one hour after sunrise to avoid locking in internal stresses.

6.4 Installation of Rubber Buffers

Rubber buffers mounted on steel plates were installed between the deck and the pylon legs. The buffers allow longitudinal and vertical movement of the deck but provide horizontal restraint for the deck against wind and seismic loads.

6.5 Bored Mini-Piles

Holes were sunk using an air percussion rig over a total period of 4 weeks. The methodology stated in the Contractor's tender submission specified the use of a temporary casing to sink the pile holes which was then to be removed during grouting. On site, however, the piling sub-contractor considered that due to possible interlock with the coarse gravels/boulders he may not be able to withdraw the casing. For this reason the first pile hole (downstream) was sunk twice without casing but in both instances, on withdrawal of the drill string, the hole collapsed. The hole was grouted up and a further attempt made to drill to depth through the grout but unfortunately the hole collapsed again. Pile hole 1 was regrouted and pile hole 2 (upstream) was drilled with the Concentrix casing advancement system. Hole 2 was successfully drilled to a depth of 15.7 m below ground level (15m below base of pile cap) and the casing retrieved.

Due to the lack of room between the Concentrix casing and the proposed PVC sheathed tension anchor bar, it was considered too difficult to pass the grout tube down the hole. To overcome this a 36 mm diameter pre-galvanised Williams Reid all-thread bar with no sheathing was adopted instead of using ungalvanised bar grouted into a PVC sheath. This bar has an ultimate tension capacity of 1054 kN, well in excess of the contract documents which required an ultimate tension capacity of 624 kN.

Pile 1 (downstream) was successfully redrilled with the Concentrix equipment, however, on withdrawal of the casing from the hole it was found to have jammed at a depth of 4m below ground level. Despite attempts to lift it by crane and using hydraulic jacks, the casing could not be removed so the Concentrix casing together with the casing advancer head had to be left in the hole. It is believed that the casing snagged on the bottom of a 168mm OD outer casing grouted into the mouth of the pile hole during previous attempts to sink the hole. The length of casing extending into the pile cap was cut off and the remainder fully grouted. Given that there was still 12m of grouted pile beneath there will be no appreciable loss in the pile's tension/compression capacity.

The materials encountered in both holes can be summarised as:-

0 - 2.9m Brown silts with small gravels
2.9m -15.7m Gravels and boulders in a silt matrix

The standing water level in both holes was approximately 12m below ground level.

The holes were tremmie grouted under gravity pressure using a grout mix comprised of 18 litres water to 40 kg cement and 200 ml Sika 1000R. Tests have indicated that this gave a 28 day strength of 57 MPa.

6.6 Minor Works

The top surface of the deck was treated with water repellent and minor repairs were made to spalled areas of concrete.

7 POST CONSTRUCTION BEHAVIOUR

The enhanced stiffness of the bridge has been the most obvious observed improvement to the bridge's behaviour.

A report was received of wind induced vibrations in the cables during a gentle breeze. The amplitude of vibration in the longest cables on the eastern side of the bridge was reported to be between 5mm and 8mm and insignificant in the other cables. Wind induced vibration is not uncommon on cable-stayed bridges. The complexity of the phenomenon means that the wind speeds which cause vibration cannot be reliably predicted by calculation. The problem has been addressed on other bridges by various means including providing a dimpled surface on the cables, running a spiral rib up the cable surface or having damping wires connecting the cables. At this stage damping of the cables is not considered worthwhile.

The deck levels were re-surveyed 16 months after the remedial works were completed and small changes were recorded. The bridge is subject to on-going monitoring and maintenance and at some stage the forces in the cable stays may be re-measured.

8 CONCLUSIONS

Soon after construction the Norbert Street pedestrian cable-stayed bridge elicited public comment on its lively response to pedestrian and wind loading. Four years later one of the cable stays failed and a few months afterwards a second cable started to fail. A review of the bridge revealed, inter alia, that the bridge cables and anchorages were overstressed and that the lack of continuity in the bridge deck lead to the bridge having a low overall stiffness. The deficiencies were addressed by the replacement of the cables with larger ones, the introduction of continuity in the deck, the provision of lateral restraint to the deck at the pylon and the strengthening of the pylon foundations.

9 REFERENCES

1 NZS 4203 : 1984 - Code of Practice for General Structural Design and Design Loadings for Buildings

2 NZS 4203 : 1992 - Code of Practice for General Structural Design and Design Loadings for Buildings

3 NZS 3101 Parts 1 & 2 : 1982 - Code of Practice for the Design of Concrete Structures

4 NZS 3101 Parts 1 & 2 : 1995 - Code of Practice for the Design of Concrete Structures

5 NZS 3404 Parts 1 & 2 : 1989 - Steel Structures Code

6 AS 1250 : 1981 - SAA Steel Structures Code

7 NZS 3404 Parts 1 & 2 : 1997 - Steel Structures Code

8 BS 463 : 1970 - Sockets for Wire Ropes

9 BS 5400 Part 2 : 1987 - Steel, Concrete and Composite Bridges - Specification for Loads

10 PODOLNY W and SCALZI J B, "Construction and Design of Cable-Stayed Bridges" J WILEY & SONS 1986 Table 9.3 p198

11 WALTHER R, HOURIET B, ISLER W, MOIA P, "Cable Stayed Bridges" THOMAS TELFORD, London 1988 Table 5.2 p82

12 The American Society of Civil Engineers, "Tentative Recommendations for Cable-Stayed Bridge Structures", ASCE J. Struc. Div. Proc., 103, No. ST 5, 929-930, May, 1977

13 The American Society of Civil Engineers, "Commentary on the Tentative Recommendations for Cable-Stayed Bridge Structures", ASCE J. Struc. Div. Proc., 103, No. ST 5, 941-959, May, 1977