ANZAC Bridge Maintenance Project

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ABSTRACT

ANZAC Bridge was opened in 1995 and is the longest stay cable bridge in Australia. The Roads and Maritime Services (RMS) of New South Wales is currently undertaking essential maintenance works including improving the performance of the stay cable system and other infrastructure on the bridge to extend the service life of this key link in Sydney's transport network.

Towards the end of the original construction, larger than expected oscillations were observed in the stays under certain combinations of rain and wind. Design solutions were developed to address this problem and an assessment of the capacity of the stay cables for current and future traffic loading was also undertaken. The project team also reviewed the access provisions under the deck, at deck level and within the towers to assess their adequacy for ongoing maintenance, specifically relating to safety. The permanent access solutions were designed with safety and low whole of life costs being the primary criteria.

KEYWORDS

Aluminium; ANZAC bridge; Cable stay bridges; Dampers; Internal Radial maintenance; Wind rain induced vibrations/oscillations

INTRODUCTION

The ANZAC Bridge was designed and constructed in the early 1990's. This iconic bridge forms a vital link connecting the centre of Sydney with the western suburbs and carries approximately 125,000 vehicles each day including pedestrians and cyclists.

The Roads and Maritime Services of NSW (RMS) proposed to undertake essential maintenance works on the ANZAC Bridge at Pyrmont / Rozelle (the proposal). While the bridge was in good condition and routine maintenance had been undertaken since its opening in 1995, the RMS now considers that it is appropriate to take advantage of advances in bridge technology and undertake more substantial maintenance. Specific areas that were investigated included stay cable vibrations and durability improvements, stay cable capacity assessment, access improvements for maintenance, improved fencing along the northern side of the bridge deck and provision of new fencing along the southern side of the bridge deck.

In late 2010, RMS formed an alliance with Baulderstone, Freyssinet and Sage Automation named Bridge Solution Alliance (BSA) to deliver the project in two phases. Aurecon in association with Cardno was engaged as the design consultant for the project and subsequently engaged Leonhardt, Andra and Partner (LAP) for their specialist stay cable input. All parties worked as an integrated team under an alliance (BSA) to deliver the works. Through a collaborative approach between BSA and numerous stakeholders, the design solution was delivered to RMS in a very short timeframe of six months. At the time of writing this paper, construction of the works is progressing.

EXISTING BRIDGE CONFIGURATION

The bridge is a cable stay structure with two planes of stay cables connecting the towers to the deck via twin edge beams. The edge beams are 1.8 m deep and between 1.5 m and 1.35 m wide. The edge beams are either reinforced concrete or prestressed depending on the location across the bridge. The reinforced concrete deck is supported by cross girders spaced at 5.1 m centres.

The bridge has three main cable-stayed spans measuring a total length of 805 m and 32.2 m wide. These spans are supported by 128 stay cables that fan out from the top of two 120 m high towers at either side of the deck, each of which is founded on 56 reinforced concrete piles. The stay cables consist of parallel strand wire comprising between 74 and 25 strands with a maximum length of 195m.

The deck originally accommodated six design lanes of traffic as well as a pedestrian/cyclist pathway. The main span is 345 m in length with 142.5m long back spans. The bridge is currently operating with eight marked traffic lanes to meet the demands of Sydney's busy road network and to better accommodate upgrades to the road network on the approaches to ANZAC Bridge including Cross City Tunnel.

STAY UPGRADE

Towards the end of the original construction, larger than expected oscillations were observed in the stays under certain combinations of rain and wind. This phenomenon is known as rain-wind induced vibration (RWIV) (Bennet, Kodakalla, et al. 2011). The concern was that these large vibrations could, over time cause a premature loss of fatigue life in the strands.

Prior to the formation of the Alliance, RMS, after receiving proposals from different organisations who supply dampers, selected a Freyssinet damping solution for the works on ANZAC Bridge. Freyssinet proposed Internal Radial Dampers (IRDs) to be fitted to the longer stays, Internal Hydraulic Dampers (IHDs) to be fitted to the medium length stays and no dampers on a limited number of the shorter stays.

Once the ANZAC Bridge Maintenance Project commenced, BSA in consultation with RMS decided that for consistency and to standardise the maintenance of the dampers, the IHDs would be replaced with IRD's. For aesthetic reasons it was then decided to install IRD's on all stays.

Stay Cable Vibration Mitigation – Internal Radial Dampers

Internal Radial Dampers (IRDs) will be installed at 1.5 m above deck level in two parts to minimise vibration effects on the stays. A two-part steel guide tube will connect to the existing steel formwork tube to provide support to the IRDs and transmit the lateral loads from the IRD back to deck level.

Figure 1 shows the position of the IRD above deck level and its inter-relationship with the other stay components. Intricate two-part connections were detailed to connect these elements together and back to the original formwork tube protruding from the deck edge beam.

To gain the maximum effectiveness of the IRDs, the lever arm between the fixed point of the stay and the damper needed to be maximised. Unlike modern stay cable bridges where the deviator collars are generally located near the bottom of the formwork tube, on ANZAC Bridge, the original deviators were located at the top of the formwork tubes. In order to maximise the efficiency of the dampers, BSA proposed to move the deviator position downwards.

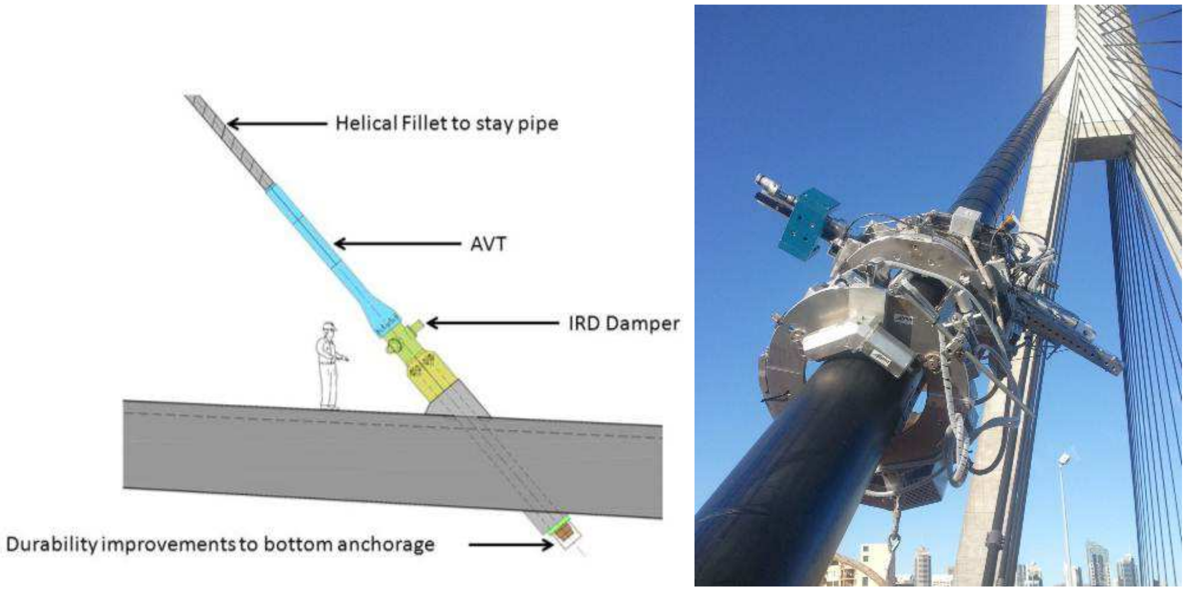

Figure 1. Extract from drawings showing additional stay components being installed at deck level and on-site helical fillet trial

Re-locating the deviator had a number of advantages including:

- The existing formwork tube was found to have adequate stiffness to resist the design loads from the IRDs without needing to be strengthened
- The fixed point for the stays at deck level would be moved, thus arresting any loss in fatigue life due to the bending action of the stays under vibration effects
- Allows the IRDs to be positioned at a lower level to minimise the visual impact and allow easier access for long term maintenance
- The new deviator would be profiled to eliminate fraying corners which will maximise the fatigue life of the stay cables

Stay Cable Vibration Mitigation - Surface Treatment

A key part of the proposed solution to prevent RWIV was to install a helical fillet on the existing smooth HDPE cable ducts.

In order to achieve this, BSA proposed to retrofit a helical fillet on each cable using a robot to weld a HDPE bead onto every HDPE stay duct. Since this type of retrofit had never before been undertaken on a cable stay structure, extensive trials were undertaken during the design phase by Alpin Technik including a full scale site test (refer to Figure 1).

Addition of Anti-vandalism Tubes

Anti-vandalism tubes (AVTs) will be installed above the IRDs to a height of 4.5 m above deck level. The AVTs comprise a two part pipe system and connects to the IRD below and the HDPE cable above.

DURABILITY

The bridge was originally designed with a provision for the stay cables to be grouted over their full length but subsequently redesigned with no grout and wax filled anchorages. As a result, the HDPE duct extended through the formwork tube to the bottom anchorage which included a drain-hole and an 8 mm diameter hole was provided though the anchorage plate. This drainage system was not adequate to prevent water build up within the formwork tube. In addition to the drainage issues at the ends of the stay cables, it was found by visual inspection that the lengths of the over-lap sleeves for thermal expansion/contraction at the tops of the stays were not sufficient and some of the sleeves have been exposed to the elements.

BSA proposed the following refurbishment works to improve the drainage of the stay system:

- Repair and increase the length of the over-lap sleeves near the top of the stay cables
- Remove the HPDE duct within the formwork tube to provide better access for maintenance
- Drill a new 30 mm diameter hole through the concrete anchorage block to allow water egress from the anchorage zone
- Incorporate additional drainage holes through the new guide tubes and IRDs above deck level

The existing strands are protected along their length with a HDPE coating extruded over the galvanised strand with a petroleum wax filling providing excellent durability. At the anchorage ends the HDPE coating is removed such that the steel wedges can grip the strand at each anchorage. The region in front of the anchor is covered with an anchor cap and the region behind is separated from the rest of the stay by a stuffing box. These two regions are then injected with wax to prevent corrosion of the strand.

Inspections of the wax in the bottom anchorages showed that quality of the wax had deteriorated and in locations, the wax was leaking from the anchorages. BSA has undertaken extensive on-site trials to develop a method to remove the wax without damaging the stay cables. The method involves inserting heating rods through the anchorage bearing plate into the anchorage zone and heating the wax in a controlled way to drain the wax. Once removed, each anchorage will be re-injected with Denso Void Filler No. 2 wax.

STAY CAPACITY ANALYSIS

An assessment of the forces in the stays was undertaken to understand the current utilisation of the stay cables. In assessing the stay capacity, the following inputs were assessed:

- Lift-off tests were carried out on 44 of the 128 stay cables to determine the current stay forces
- Fatigue tests were carried out on short sections of the strands to check that no premature fatigue loss had occurred
- A geometric survey was taken of the tower and edge beam
- Traffic data was collected to utilise for the fatigue assessment
- Concrete test results of the edge beam were taken

A three dimensional analysis model was set up using the ‘Sofistik’ software package as the primary modelling tool for calculating load and load combination effects on the bridge. The superstructure was modelled as a spine beam and incorporated the actual geometric shape of the varying cross section, i.e. including the widening in Span 2, the additional webs in Spans 2 and 4, and the box girders in the approach spans. All longitudinal post-tensioning in Spans 2, 3 and 4 was also included. A graphical output of the model is shown in Figure 2. A separate three dimensional analysis model built using Microstran to undertake an independent verification of the design actions which achieved a correlation of less than 5% for a number of load cases.

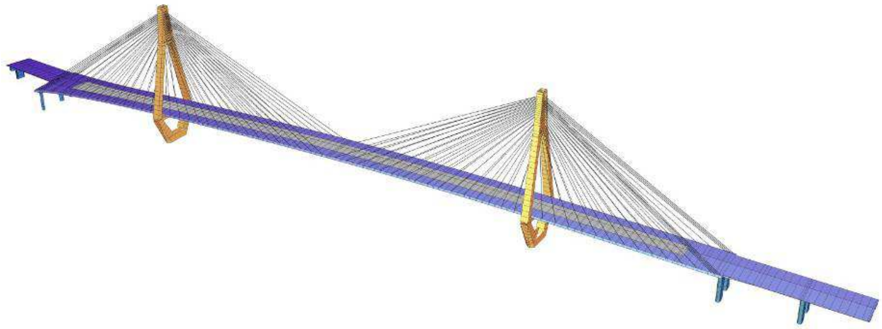

Figure 2. Model snapshot from Sofistik

Shear Lag was considered in the deck section properties when calculating the effects of short loaded length, such as truck loads. The effective width was determined based on Eurocode EN 1992-1-1:2004, clause 5.3.2.1 (Eurocode EN, 2004) since AS5100 (Australian Standards, 2004) was deemed too conservative in this case. Modelling of the torsional stiffness included both the St Venant torsion and warping torsion since for an open section this has a significant effect on the overall stiffness.

Based on the stay lift-off results the dead loads in the model were adjusted such that the total dead load of the deck matched the average vertical component in the stays and this was then used to determine the bending moments in the edge beam. This is important since the bridge has no vertical supports except at the tie down piers.

It is noted that the existing stay forces were compared to theoretical model outputs by carrying out lift-off tests under live traffic conditions. An assessment showed that this may have typically increased the stay forces by about 2% of the current dead load or about 0.8% of Guaranteed Ultimate Tensile Strength (GUTS).

The stay forces were checked for a number of load cases including 7 Lanes of NAASRA T44/L44 Loading, 8 Lanes of NAASRA T44/L44 Loading, 7 Lanes of SM1600 AS5100 Loading and NAASRA T44/L44 loading applied with AS5100 lane factors.

The design criterion was partly based on the Bridge Specific Assessment Live Loading (BSALL) developed for the Westgate Bridge in Melbourne (Cooper, 2009). Based on the traffic study data from the Sydney Harbour Bridge provided by RMS and from the BSALL study undertaken on the Westgate Bridge in Melbourne, it was determined that the 7 or 8 lanes of NAASRA T44 loading gave a reasonable if not slightly conservative approximation to the current traffic loading on the ANZAC Bridge. Consideration was also given to the fact that historical data shows that traffic loading in Australia has increased in intensity by an average of 10% every ten years.

With regard to fatigue, the stay forces were checked for two cases, namely A single T44 fatigue vehicle with 15% dynamic load allowance and 70% of an M1600 vehicle with no associated UDL and with 15% dynamic load allowance.

The application of the M1600 fatigue vehicle in accordance with AS5100-2004 was assessed for the ANZAC Bridge. The mass of this fatigue vehicle is about 100t which was considered unrealistic noting the fatigue vehicle described in AASHTO (AASHTO, 2007) and the PTI (Post Tensioned Institute, 2007) is approximately 32 t. Following extensive reviews with RMS, the T44 vehicle was used to check the remaining fatigue design life of the stay cables.

Stay Capacity Results

The capacity at Ultimate Limit State (ULS) was assessed for both the normal load factors given in AS5100 but also reduced load factors as described in AS 5100.7 clause 3.1. In conclusion under current traffic loads the stay forces are acceptable at ULS as required by the PTI. Some stays are slightly overstressed at Serviceability Limit State (SLS), however an SLS check is not a requirement of the PTI.

If future traffic increases are taken into account then this needs to be compared against the possible conservatism of the traffic assessment and further assessment of the live loading is being considered to be undertaken in Phase 2 of the project.

The live load fatigue was found to be acceptable since the stays have capacity for either the full number of cycles of T44 load or a reduced number of cycles of 70% M1600.

ACCESS IMPROVEMENTS

BSA engaged with the relevant stakeholders on the project including RMS Bridge Services to develop the access improvement solutions for the project. The development of the external access improvements required a significant interface with the urban design consultant for the project from Hassell (Hassell Ltd., in press) since any changes to the current bridge configuration may have adverse visual impacts. Hassell was engaged by the RMS to provide urban design input into the design solutions proposed by BSA.

Access to stay cable components at deck level

Due to the number of new stay components being added to the stay cables at deck level, there will be increased maintenance requirements compared with the current situation. Currently, maintenance personnel use a temporary static line to access each stay cable at deck level. This access method was considered the least desirable as personal protective equipment is the lowest form of safety control. For BSA's construction works, a temporary access walkway was required along the full length of the bridge. Through a value for money workshop, it was determined that changing the temporary access walkway to a permanent walkway would provide a safer access system across the bridge for long term maintenance.

Deck level maintenance walkways will be located along the outside of the edge beams for the entire bridge length. The walkways will be 600 mm wide and constructed using aluminium structural elements fixed to the side of the deck edge beam with a Fibre Reinforced Plastic grating. Aluminium was chosen for its low whole of life costs.

Access to anchorage zones in tower head

Currently access to the tower head is via an internal ladder system which will be locally modified to accommodate a new lift. A new industrial lift inside the northern tower leg of each tower will be installed to provide a speedy access from deck level to bottom of the tower head. New materials hoists will be placed within the tower head with a 350 kg safe load limit. The hoists will pull a custom designed tray system guided on tracks up and down the tower head. One of the key criteria for the hoists and lift system was to provide an emergency egress system in the event of a person being injured within the tower.

Access to under-deck anchorage zones

New under-deck maintenance walkways will be provided to access the last eight stay cable anchorages on each corner of the bridge to provide a fixed safe access system for inspecting and maintaining the stay anchorages. New under-deck walkways will be installed under the deck edge beams at the bridge ends, providing over 200 m of fixed access. These new walkways will comprise fabricated aluminium warren trusses that span approximately 10.3 m between supports. The trusses will be clad using a perforated aluminium sheeting to enhance the aesthetics of the new works. Through BSA's Design for Maintenance process which involved the asset owner, aluminium was chosen as the preferred material due to its low maintenance requirements long term and the access difficulties in maintaining the new walkways. Access to the new underdeck walkways will be via the underdeck gantry.

Underdeck gantry upgrade

A condition assessment and load rating of the gantry and runway rail was undertaken in accordance with AS1418: Cranes, Hoists and Winces (Australian Standards, 2002) and AS4100: Steel Structures (Australian Standards, 1998). It was found that the gantry was suitable for use however some minor modifications were recommended relating to improving the emergency egress system and maximising the space around the stay anchorages for ongoing maintenance which was previously very restricted.

NORTH AND SOUTH FENCE UPGRADE

A part of BSA's scope of works was to develop design solutions for a new fence on the southern side of the bridge and to investigate the possibility of re-using the original fence on the northern side of the bridge.

The original pedestrian fence on the northern side of the bridge was fully welded and constructed from aluminium. Cracking within the top flange of the bottom rail around the heat affected zone at the baluster connections had occurred at a number of random locations, however they were only observed on the stay cable spans of the bridge. No fence was present on the southern side.

BSA investigated the cause of cracking and concluded that there were two contributing factors. The first was liquation cracking that occurred during the original fabrication of the members. Liquation cracking occurs in the heat affected zone where micro-cracks form in the base metal due to the melting of the low melting-point grain boundary constituents (Huang, Cao, et al., (2005). These micro-cracks may never propagate into larger cracks, unless they are located in high stress areas. This was the case for the fence on ANZAC Bridge. The second contributing factor was the vertical balusters resonating or vibrating under high wind conditions with the wind direction perpendicular to the bridge. These vibrations caused high stress concentrations at the ends of the balusters with many stress cycles leading to fatiguing of the connection between the baluster and the bottom rail. This propagated larger cracks in the bottom rail in a number of locations.

Although the cracking to the bottom rail did not pose an immediate safety concern, BSA recommended that the fence be replaced and a new fence installed on both the northern and southern side. Following a comprehensive options study, a mesh panel fence was selected for its security and aesthetic attributes. The new fences will comprise aluminium frames supporting security weldmesh panels with the top rail set at 3.0 m above deck level.

Figure 3. 3D Visualisation of proposed fence on southern side of bridge

A significant amount of urban design was applied to the new fence shape. Connections were carefully detailed to eliminate the welding of aluminium in the high stress zones. The traffic facing flange that connects to the mesh panels will incorporate a slight inward curvature to provide an out of plane stiffness to the mesh panels to minimise warping or "pillowing" of the mesh panels.
The mesh panels will be constructed from steel with an aperture size of 72.2 mm wide by 8.7 mm high. This provides good transparency, even when viewing from oblique angles. The corrosion protection system for the mesh will comprise a zinc/aluminium undercoat with a powder-coated second level of durability protection.

CONCLUSION

The ANZAC Bridge has been in service for over 17 years. Although generally performing well, a number of maintenance issues had become apparent that needed to be investigated and refurbished where necessary, to ensure continued performance into the future and to avoid possible future durability issues.

Through the formation of the Bridge Solutions Alliance team, design solutions relating to the stay cable performance, access for future maintenance and upgrade works to the fence were resolved considering a number of project constraints including safety, whole of life costs, security, cost, traffic, constructability, urban design and the environment. Early and regular engagement of key stakeholders was the key to identifying the correct technical solution to provide the best value for money for the client.

The design phase of the project was delivered in less than six months and on programme. This was achieved through a collaborative approach adopted by each project participant. The final solution involved a number of innovations including the post-installation of a helical fillet weld to the stays on an operating bridge - a world first. Construction is currently underway and is due to be completed towards the end of 2013.

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REFERENCES

Bennet M., Kodakalla V. and Gupta V. (2011). Vibration Mitigation Measures in Cable Stayed Bridges, Proc. 8th Austroads Bridge Conference, Sydney, NSW, Australia.

EN 1992-1-1: 2004 Eurocode 2: (2004). Design of concrete structures - Part 1-1: General rules and rules for buildings.

Australian Standard AS 5100-2004: (2004). Bridge Design. Standards Australia, Sydney.

Cooper D. (2009). Highway Traffic Load Models for Bridge Design and Assessment, Proc. 7th Austroads Bridge Conference, Auckland, New Zealand.

American Association of State Highway and Transportation Officials, AASHTO LRFD. (2007). Bridge Design Specifications, SI Units, 4th Ed., Washington, DC, USA.

Post Tensioned Institute. (2007). PTI Recommendations for Stay Cable Design Testing and Installation 5th Edition. U.S.A. October 2007.

HASSELL Limited. (2011). ANZAC Bridge Maintenance Project – Landscape Character and Visual Impact Assessment Report dated July 2011 (Unpublished report prepared for the Roads & Traffic Authority of NSW).

Australian Standard AS 1418.1-2002: (2002). Cranes, hoists and winches—General requirements. Standards Australia, Sydney.

Australian Standard AS 4100-1998: (1998). Steel Structures. Standards Australia, Sydney.

Huang C., Cao G., and Kou S. (2005). Liquation Cracking in Full-Penetration Aluminium Welds: A Necessary Condition for Crack Susceptibility, Proc. 7th International Conference, Trends in Welding Research, Callaway Gardens Resort, U.S.A.

Australian Standard AS/NZS 1664.1-1997: (1997). Aluminium Structures Part 1: Limit State Design. Standards Australia, Sydney.