ABSTRACT

This paper provides an overview to selected construction challenges encountered during the building of the Ormiston Road cable stayed bridge, an iconic cable stayed composite steel and concrete bridge constructed in the Sir Barry Curtis Park, Manukau City, Auckland.

INTRODUCTION

The new Flat Bush township will be New Zealand’s largest urban development with 15,000 new houses and a town centre planned for construction over the next twenty years, eventually supporting a population of 40,000 people. The Manukau City Council wanted a spectacular landmark bridge to form a gateway into this new community. Beca Infrastructure Limited in conjunction with Craig Craig Moller Architects entered a Manukau City Council design competition for a new bridge, in mid 2004. The concept for the bridge design submission was to ‘touch the ground lightly’ and seemingly soar across the 70m span, with a slim profile bridge supported by cable stays off two asymmetrical pylons. Although cable stayed bridges are more usually adopted for longer spans, the form has been used very effectively for spectacular shorter span structures and was considered appropriate to meet Council’s objectives of a “gateway” and a “landmark feature” and the design was accepted.

The bridge incorporates dramatic lighting effects similar to the lighting of Auckland’s Sky Tower. This consists of feature lighting of the pylons, cables and decks together with the top section of the pylons, which are illuminated from within to resemble beacons, which are visible for some distance.

Thus the bridge provides an elegant element in the landscape both during night and day and is the first traffic carrying, cable stayed bridge constructed in New Zealand.
Fulton Hogan Ltd was awarded the construction contract to build the bridge and 1.2kms of 4 lane arterial road with supporting infrastructure. Construction Techniques Ltd was awarded the subcontract to manufacture, install and stress the cable stays and to provide post tensioning services. This paper discusses challenges with construction of selected elements, the pylons, the tension piles and the installation and stressing of the cable stays.

APPRECIATION FOR THE TASK

The construction of the cable stay bridge was technically very complex due to the asymmetric geometry and very tight tolerances specified. The bridge deck is on a radius of approx 37kms, which sounds very flat but results in variations in levels due to curvature of 66mm along the length of the bridge. The 45.5m pylons are made up of a 28m section of reinforced concrete tapered from 1.8m diameter at the base to 1.3m diameter at the top, with a 5.5m high structural steel box to provide anchorage for the stay cables and topped with a 12m lattice spire made of stainless steel and glass. To further complicate matters both pylons are inclined back longitudinally at 15 degrees and angled together at 5 degrees and were not self supporting. There was very little tolerance in ensuring the stay cables were correctly aligned between the pylon and the deck anchorages. The angular rotational tolerance of 0.25 degrees commonly specified for cable stayed bridges required the positional tolerance of the stay anchorages to be within 3mm. With this level of accuracy much of the construction effort and risk mitigation was focussed on survey integrity and conservation of construction tolerances.

TENSION PILES

The concrete pylons are angled in two directions providing a dynamic element to the bridge. They are also positioned closer to the western abutment than the eastern meaning the back span is considerably shorter than the fore span. This asymmetry generates considerable uplift on the western abutment which is resisted with deep tension piles. The four western abutment piles were designed as 37m long 900mm diameter reinforced concrete piles with large capacity post tensioning tendons through the centre to ensure that, under serviceability loads the piles would remain uncracked. These piles were very difficult to construct due to their length, the confined working space inside the pile caused by the presence of the tendon and the high hydrostatic concrete pressures at the base of the pile. Normal Drossbach ducting could not be used as the tendon sheathing after research showed that Drossbach could collapse at about 12m head of concrete. 100NB steel pressure pipe was used as an alternative, which could cope with the high hydrostatic concrete pressure during pouring and temporary loads during installation. Tendons were assembled on the ground prior to lifting and placing inside the piles which already had the reinforcing cage installed. It took a synchronised effort of 3 cranes using 6 snatch blocks and an excavator to successfully lift the 45 m long flexible tendons from horizontal to vertical, without kinking the tendon, so that they could be lowered into the pile casing.
Pouring of the piles was also difficult as there was only an 83mm void between the tendon and the reinforcing cage. Each pile was poured using a specially designed 75mm diameter tremie pipe with 10mm aggregate concrete.

The pile tendons pass through the western abutment and terminate in the deck. This meant that the tendons could not be stressed and grouted until the deck had been poured, some 9 months later. As a temporary measure to prevent corrosion of the strand, a sodium hydroxide solution was introduced to the pile tendons to create a protective alkali environment. Regular pH testing was used to monitor and maintain alkalinity.

This measure combined with temporary sheathing of the exposed strand allowed the tendons to be fully stressed, to the design load of 360 tonne without strand failure. The tendons were then grouted with a cementitious grout.

CONSTRUCTION OF THE PYLONS

The inclined pylons are constructed in three sections. All of different materials. The lower 28 m section consists of reinforced concrete circular section tapering from 1800mm diameter at the pylon base to 1300mm diameter at the top of the concrete section. Above the concrete section, the cable stay anchorage itself consists of a
fabricated steel box stressed to the concrete pylon. A non structural stainless steel and glass lattice frame in the form of a cone completes the top of the pylon providing a visual effect of the tower tapering to a point, which was a key architectural feature.

The pylons were too slender and at too raked to be constructed as self supporting elements and temporary support would be required until the transverse steel member between the top of the pylons and a number of the forestays had been installed.

Four different methods were identified for constructing the pylons. These were an insitu option and a precast option with three different methods of erecting the columns by pushing, pulling or lifting. The precast and pull into position option was preferred at the time of tender.

On contract award the project was re-evaluated with the preferred tender methodology of precasting the pylons and jacking, the combined 400t weight of the pylons and crossbeam into position, rejected in favour of casting the pylons insitu. The portal beam and pylon stay cable anchorages are stressed to the reinforced concrete section of each pylon with 12 x 40mm cast in hold down bolts. Placing these hold down bolts in the correct location was critical and identified as an extreme risk. With the precasting option the hold down bolts could easily be cast outside of the tight tolerances. By selecting the cast insitu option the pylon head and hold down bolts would be placed prior to base pumping self compacting concrete, eliminating the risk of casting the hold down bolts out of position.

The insitu method also allowed the pylons to be surveyed and adjusted (by jacking the back props) into precise position prior to casting the pylons, but meant that not only would the temporary works support the weight of the wet concrete, but also the 58t of structural steel sitting on top of the forms.

One complication was that the tapered steel forms into which the lower concrete pylon section were cast, would have to be painted to prevent rust staining of the concrete. The casting of the pylons was a relatively early activity and the steel formers could not be removed until the first cable stays had been installed, which occurred late in the programme, after the deck had been poured and the first two forestays installed.

The concrete pylon formers were made from 8mm steel plate, with each 1200mm plate rolled into the tapered section of the pylon. This meant each 1200mm plate was rolled with a 22mm taper. The pylon formers were supported by twin 900mm diameter strong back tubes and four 600mm diameter back stays. To provide the required reaction force to support the pylons the back stays were founded on a temporary pile and pile cap which was laterally stressed to the western abutment and longitudinally to the pylon pile cap using a strut and tie arrangement. Simple pin connections and sliding supports were used in the temporary works to facilitate jacking of the erected pylon forms into a precambered position prior to pouring. The precamber accounted for the anticipated 20mm deflection of the forms during the concrete pour.
An F5 finish to the pylons was specified with visible construction joints not allowed. With the pylons raked at 15 degrees longitudinally and 5 degrees transversely free falling concrete or the use of tremie pipes or pump hoses would not work to place the 55m3 of concrete in each pylon. There were also eight 25mm service ducts cast into each pylon to supply power to the many lights and CCTV system on the bridge, which further restricted access and could easily be damaged. There were also difficulties in placing and vibrating the concrete as the form would be fully closed by the stay anchorage structure at the top.

A decision was reached to base pump the pylons to prevent segregation. Self compacting concrete was selected so no vibration would be required. Base pumping to a height of 28 metres is not commonly attempted and there was some risk that this volume of self compacting concrete could not be pumped against the hydrostatic pressure. Measures would also need to be put in place to ensure the finish of the concrete pylons would achieve the specified F5 finish.

A 6m high trial pour was conducted to determine the pumping pressures during the pour and the best method to discharge and pressurise the form to reduce the number of bubbles in the finish. A range of form release agents and form seals were also trialled to determine the products that gave the best finish and reduce leakage of the highly fluid self compacting concrete. The gate valve to shut off the concrete pump was also performance tested.
The trial pour provided some interesting information on pump pressures, characteristics and behaviour of the base pumped self compacting concrete and how to effectively surcharge the form to achieve a better concrete surface finish.

It was considered that the pylons could be base pumped to the full height, in a single pour. However due to programming issues, it was decided to change the pour sequence of the pylons from pouring full height in one lift to forming a construction joint at the footpath level and pouring the pylons in two lifts. The construction joint is hidden by the footpath.

The pylons were poured up to footpath level a head of about 9m without any issues. A curved concrete cross beam connecting the pylons at deck level was then poured.

Once the concrete cross beam was constructed the reinforcing cage for the second stage pylon pour was placed. Also, the cable stay anchorages and the portal beam across of top of the pylons were placed, the forms sealed and closed. The remaining 19 metres of the pylons was then base pumped. Over 12 months of detailed planning to construct the pylons came to a successful two hour conclusion with this pour.

The staged stripping of the forms from the pylons revealed a high quality finish of the base pumped self compacting concrete. No segregated concrete or voids were found in the pylons and no patch repair was required.
CABLE STAYS

While the bridge span is short at 70 m, the effective tributary load area for the cables was of similar magnitude to a much larger cable stayed bridge because of the large deck width and resulted in similarly sized cable stays. The stays are constructed using the BBR Dina cable stay system which is of Swiss origin, and is used in many cable stayed bridges around the world.

The bridge is stayed by 20 stays, with ultimate capacity of up to 780 tonnes each, and with overall lengths between 20 and 53 metres. Stays connect to the pylons at the upper end via heavy cast steel clevises and pins. The lower ends pass through the main deck girder and terminate with a large threaded lock nut which allows the tension in each stay to be adjusted.

The stays consist of 7 mm diameter galvanised stressing wire, arranged in a compact parallel bundle within a thick walled HDPE sheath which is filled with a flexible corrosion-inhibiting compound. Individual wires are terminated at both ends with a cold-formed button head which transfers the full load to the anchor heads.
The stays were manufactured on site. The area behind the western abutment was set up with a 60 m long wire cutting and assembly facility. Various components used in the stays were manufactured in India, Germany, Spain, Switzerland, The United States and New Zealand. The twenty stays consist of between 78 and 144 individual extremely high strength steel wires and are up to 53 metres in length. In total nearly 70 km of wire had to be measured and accurately cut to length. To achieve satisfactory productivity a system of uncoiling the wire and mechanically shooting it along a guide for cutting to length was set up.

Once a set of wires for a pair of stays had been cut these were inserted through the various components at each end and through the length of the HDPE sheath. Each wire had to be inserted through a predetermined hole in the components at each end to ensure the wires remained as a parallel bundle, free from twists.

Once insertion of the wires was complete the wire ends are terminated with button heads. The button heads are cold formed by a specialist machine which grips the wire and pushes it into a dye. A circular profile is formed of consistent dimensions and which has been found to transfer the full wire capacity to the anchor heads. The process is similar to forming jolt heads on timber nails.
When the button heads had been formed the HDPE sheath was connected to the end anchorages and the corrosion-inhibiting compound injected to completely fill the HDPE sheath.

A number of options for cable installation was considered but it was decided that the stays would be installed complete and with the clevis fitted, using two cranes and a mechanical excavator.

![Figure 13: A stay being installed.](image1)

![Figure 14: Inserting the clevis pin.](image2)

The cables were laid out along the deck on trolleys and lifted at the clevis. The clevis was then attached to the connection plate, welded to the cable attachment structure and a 160 mm diameter hardened steel pin inserted. The lower end was then inserted into the guide pipe through the main girder.

The design required that cables opposite each other across the bridge, be stressed in pairs to avoid torsional forces in the deck structure or pylon. This necessitated twin sets of jacks and trestles.

As the cable sag was large, cable tensioning had to commence during installation and prior to release of the cranes. The load extension characteristics of stays are non linear and only light tensioning forces are required during the initial stage when sag is removed. A single pre-stressing strand was attached to the lower end of the pull rod connected to the lower end of the stay and a small capacity jack was used to pull the stay into a position where the pull rod engaged and the large capacity hydraulic stressing jacks could be utilised, for the final stress.

The main tensioning system consisted of 250 tonne centre hole hydraulic jacks tensioning the stays via a re-locatable 75 mm diameter, 450 tonne pull rod.
As the hydraulic jacks have small stroke compared with the required stay extension a two level trestle was utilised to ensure the stay could be locked at stages during tensioning. This ensured that the jack could be retracted, at the end of each stroke, without losing the force already applied to the stay. The hydraulic jacks were cycled until the required stay force was achieved.

During each stay stressing operation the pylon movement was monitored and a deck level survey was performed after each operation. Initially the first two sets of fore stays closest to the pylon were stressed to support the weight of the concrete pylons so that the temporary supports could be removed. After the temporary supports were removed the first back stays were installed and stressed. Stay installation progressed outward from the pylon, toward either end of the bridge, so that force on the pylon was balanced. As the stays were tensioned the temporary deck supports, on which the bridge deck was constructed became unloaded and were removed.

The force in any given cable is reduced as cables nearby are tensioned. The weight of a specific section of deck is redistributed between adjacent stays. A number of stressing cycles are usually required to achieve the design force in each stay. The consultants modelled the bridge and it was found that two cycles of stay tensioning would be required to avoid overstress of the stays and other bridge elements. At the completion of these two cycles the stay forces, pylon position and deck levels were very close to design predictions.

CONCLUSIONS

As the first significant cable stayed bridge constructed in New Zealand there were a number of challenges to be thought through and overcome. Generally it was found that, with the level of planning undertaken that anticipated complications did not arise and very few unforeseen problems were encountered. The bridge was successfully completed in May 2008 and opened by the Mayor of Manukau City, Mr. Len Brown, in a ceremony held in October 2008.
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APPENDIX
Figure 17. Stay cable details and arrangement

Figure 18: Bridge cross section.
Figure 19: Bridge deck plan