

ORMISTON ROAD CABLE STAYED BRIDGE

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ABSTRACT

This paper provides an overview to the design and construction of the Ormiston Road cable stayed bridge, an iconic cable stayed composite steel and concrete bridge constructed in Manukau City.

INTRODUCTION

The Ormiston Road Bridge project involved the design and construction of New Zealand's first trafficked cable stayed bridge and 1.2kms of 4 lane arterial road and supporting infrastructure for the new Flat Bush Township. 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 Ormiston Road cable stayed bridge forms an iconic gateway into this new community.

The idea of supporting structures using tension elements is not new. The first use of tension elements as the primary support to a bridge structure can be traced to Venice around 1600. In 1784 the carpenter C. J. Löscher designed a cable-stayed bridge with an approximate span length of 32 m and where the entire bridge was made out of wood, including the stays.



Figure 1. Löscher's Bridge built 1784

It was not until the 1950s that Dischinger designed the first true cable-stayed bridge. The Strömsund Bridge (Sweden, 1955) had a main span of 183 m and two side spans of 74.7 m.

In New Zealand cable stay bridges are an emerging technology, with the only other examples of cable stayed bridges being for pedestrians. Cable stayed bridges are usually selected for applications where superior span lengths are desired, or as in the case of Ormiston Road where their striking form is a key requirement of the design.

DESIGN

Background

Beca Infrastructure Limited (Beca) in conjunction with Craig Craig Moller (CCM) Architects entered a Manukau City Council (MCC) design competition for a new bridge located in the Barry Curtis Park in mid 2004. The competition required the bridge to be a landmark structure, providing a gateway appearance for travellers entering and leaving Flat Bush.

The concept for the bridge design submission was to 'touch the ground lightly' and seemingly soar across the 70m space, with a slim profile bridge supported on asymmetrical placed tower pylons using cable stays to the outer sides. The bridge needed structure elements above deck level for visual impact and a gateway appearance, which favoured a cable stay design.

The concrete pylons were angled in two directions in relation to the road alignment, providing a dynamic, element to the park landscape and a clear and uncluttered transition for the park to flow beneath the bridge.

Although cable stayed bridges are more usually adopted for longer spans than those considered here, 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".

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 ephemeral beacons, which are visible for some distance.

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Thus the bridge will provide an iconic and elegant element in the landscape both during night and day.

Design Criteria

The bridge is detailed to have a design life of 100 years. The principal design criteria for this bridge were:

- HN_HO_72 live loading in accordance with the Transit New Zealand Bridge Manual
- Wind loading derived from AS/NZS 1170
- Seismic loading in accordance with AS/NZS 1170 for an elastically responding structure
- Cable design in accordance with Post Tensioning Institute (PTI) Guide "Recommendations for Stay Cable Design, Testing, and Installation".
- Fatigue in accordance with AS 5100
- Deck vibration in accordance with Canadian Highway Bridge Design Code. AS5100 does not provide acceptable vibration levels for cable stayed bridges

Geotechnical Conditions

Subsurface materials comprise of Puketoka Formation Alluvium overlying East Coast Bays Formation Waitemata Group Rock. The alluvium comprises firm clays and loose sand layers with most SPT values in the range of 6 to 8 blows per 300mm. The depth to bedrock varies with almost 10m difference in rock levels between the two abutments, being approximately 23m and 14m below existing ground respectively.

Structure Description

Refer to Appendix 1 and 2 for an elevation and cross section of the bridge.

Abutments

The abutments and pylons are supported by bored piles socketed into competent Waitemata group mudstone. The piles are grooved within the rock socket to develop the required axial resistance by skin friction.

The western abutment piles are post tensioned due to the uplift forces generated by the asymmetrical nature of the bridge. The post tensioning is designed to maintain the pile in compression under all serviceability limit state loads. As the pile stiffness affects the structures deflection it was beneficial for the pile stiffness to remain uncracked under all serviceability load conditions.

The abutments are live load continuous with the superstructure. This assists with vibration control and removed the need for deck joints. The

abutments were constructed in stages so the dead load of the superstructure did not induce significant moments into the piles.

Pylons

The raking of the tower pylons in the longitudinal direction requires a tension tie beam between the pylon pilecap and the western abutment to transfer the horizontal axial component of the pylon. This was preferred to the pylon piles resisting the horizontal load in flexure, which would generate additional creep moments into the piles as the resisting soil crept over time. The tie beam was also post tensioned to remain in compression under all serviceability limit state load conditions for the same reason as the western abutment piles.

The reinforced concrete pylons taper from 1800mm diameter at the pylon base to 1300mm diameter at the interface with the cable stay steel anchorage box. A stainless steel lattice frame in the form of a cone completes the top of the pylon. The lattice cone has glass mounted inside and utilised as part of the bridge feature lighting. The lattice frame also provided a visual effect of the tower tapering to a point, which was a key architectural feature.

The cable stay anchorage consists of a fabricated steel box stressed to the concrete pylon. The anchorage box is inclined with the pylon and requires vertical steel plates projecting out for anchorage of the clevis and pin connections to the stay cables.

The pylons also incline transversely inwards over the carriageway. A horizontal steel portal beam is required to resist the horizontal axial load component generated from the cable stays.

Superstructure

The deck carriageway consists of two 3.2m lanes, a 1.4m cycle lane and a footpath travelling in each direction. The carriageway is separated in each direction by a 3.5m void running down the centre of the bridge allowing natural light underneath the bridge. The overall width of the bridge is 27m including the interior void.

The superstructure consists of slender box girders supported by the cable stays at 7m intervals. The box girders support transverse cross beams which in turn support intermediate stringers. This system was chosen rather than the option of only transverse cross beams so the deck thickness could be minimised reducing the overall dead load onto the cable stays and enhancing the architectural requirement for a slender deck.

The 170mm thick insitu concrete deck is designed to act compositely with the deck steel and carry a proportion of the horizontal axial compression load generated from the cable stays.

Cable Stays

The size of the bridge created challenges for detailing. 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 similar size cable stays. The challenge was to fit the cable stay anchorages in the small confined 1m deep steel box girders and also to anchor the cable to the top of the pylon. This resulted in the use of two back stay cables of smaller size rather than one large back stay.

Stay cables are galvanised parallel wires protected by a petroleum based product in a HDPE sheath with BBR DINA anchorages at the live end and proprietary clevis anchorages at the pylon anchorage location. The parallel wire system was preferred to the parallel strand system based on the vibration performance and fatigue characteristics. The cables are protected by the road barrier and a robust steel sleeve projecting from the deck.

The design of the cables is limited to 0.45fpu of the cable capacity at the serviceability limit state. Fatigue assessment of the cables was in accordance with the PTI Guide based on a single design truck occupying a single lane.

The constant axial load on the concrete pylons and concrete deck causes the concrete to creep over time resulting in a decrease in cable prestress in the long-term scenario. The short-term cable prestress load was assessed to ensure the cable stress limit was not exceeded when the bridge is first opened to traffic.

Two load combinations consisting of the loss of a cable were considered. The first situation was for the replacement of a cable under controlled conditions. This would require the closure of two lanes closest to the cable being replaced. The remaining two lanes would be contra-flowed with one lane for each direction of traffic. This load combination is treated as an overload combination.

The second situation is the failure of a cable due to vehicle impact or fatigue, with reduced traffic loading on the bridge. This situation is assessed to prevent progressive collapse of the bridge. Inelastic behaviour of all elements is acceptable provided that collapse is prevented. This is an extreme event and a reduced live load factor was applied similar to

the reduction used for live load combined with ship impact load in the AASHTO LRFD Bridge Design Specifications.

The stability of the bridge was also assessed for the loss of one crossbeam so no lateral restraint to the box girder is provided at this location. The critical element assessed was the box girder with a reduced concrete deck area for carrying the deck compression load.

CONSTRUCTION

Appreciation for the Task

The construction of the cable stay bridge was technical 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 top stay anchorages to be within 3mm, 33m above ground level. With this level of accuracy required much of the construction effort and risk mitigation was focussed on survey integrity and conservation of construction tolerances.

Construction Methodology Selection

Four different methods were identified for constructing the pylon column formers and support structure at the tender stage; 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.

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 an extreme risk, with the likely outcome the hold down bolts would be inadvertently cast in the wrong place.

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 columns insitu where the specified construction tolerances could be achieved with a lot more certainty. The pylon head and hold down bolts would be placed prior to base pumping self compacting concrete (SCC), eliminating the risk of casting the hold down bolts out of tolerance.

The insitu method allowed the pylons to be surveyed into the 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. The temporary works for the pylons were also designed so they could be moved into the precise final position prior to casting by jacking the back stays.

Piling

The ground conditions for piling were not too challenging with the underlying mudstone dipping 7 degrees to the west. The 900 diameter western abutment piles were 33m deep and 24m deep for the eastern abutment. The 1200 diameter pylon foundation piles were drilled from a lower height and were 27m deep.

The four western abutment piles were designed as 37m long 900mm diameter post tensioned piles. These piles were very difficult to construct due to their length, confined working space inside the pile and 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 ducting would collapse at about 12m head of concrete. 100NB steel pressure pipe was used as an alternative, which could cope with the high pressure demands and temporary loads during construction.

Tendons were assembled on the ground prior to lifting and placing inside the pile reinforcing cage. It took a synchronised effort of 3 cranes using 6 snatch blocks and an excavator to successfully lift the 40m long flexible tendons from horizontal to vertical without kinking the tendon. Pouring of the piles was also difficult as there was only an 83mm void between the tendon anchorage and the reinforcing cage former rings. Each pile was poured using a specially designed 75mm tremie pipe with 10mm aggregate concrete.

It was realised after the pylons were poured the unsheathed strand used in the pile tendons would require temporary protection from corrosion, as the

tendon could not be stressed and grouted until the deck was poured 9 months later. A sodium hydroxide solution was introduced to the pile tendons to create an alkali environment inhibiting corrosion of the strand. Regular pH testing of the tendon water was used to monitor and maintain alkalinity.

The extreme pressures at the base of the pile caused an issue with one pile, where grout forced itself up through the strand weave. This grouted about half of the wedges into place before the strand could be initially pulled. This problem was overcome by pulling on individual strands to check for seating of the wedges and reintroducing sheathed strand with an exposed tail into the tendon. The original unsheathed tendons that had anchorage were stressed and then the tendon was grouted. Once the grout in the tendon had reached sufficient strength the sheathed strands were individually stressed, maintaining the required capacity of the tendon.



Figure 2. Placing stressing tendons in the western abutment piles

Western Abutment

The western abutment was a very complex reinforced concrete structure in its own right. The abutment beam is of variable height to a maximum of 4m, dog boned shaped with a maximum width of 1.925m. Most of the complexity was introduced by the reinforcing layout with 5 rows of top and bottom reinforcing, horizontal and vertical double stirrup sets at 100mm centres. The pile reinforcing including four pile tendon anchorages and the eight raked tie beam tendons terminated over a 3.4m length in each end of the abutment. The reinforcing was extremely congested and took two months to fix into place. To further add to the complexity the

abutment was constructed with a horizontal construction joint, as the steel box girders and deck stringers were required to be cast into the top of the abutment.

Concrete Pylons

Column former and temporary works



Figure 3. Erection of the steel portal beam and pylon anchorage on top of the column formers prior to pouring

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 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.

Risks with concrete

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. There were also difficulties in vibrating the concrete as the form would be fully closed by the pylon anchorages at the top.

A decision was reached to base pump the pylons using self compacting concrete (SCC), as it would provide the best response to deal with segregation and compaction issues as no vibration was required.

There was some risk that this volume of SCC could not be base pumped to this pressure. Measures would also need to be put in place to ensure the finish of the concrete pylons would achieve the specified F5 finish. The back steel pylon forms were also required to be in place for 6 months after the concrete pour, while the deck was constructed prior tensioning stay cables. Rust staining of the concrete pylons from the forms was identified as an issue and were epoxy coated overcome this.

Trial pour

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 SCC. 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 SCC and how to effectively surcharge the form to achieve a better concrete surface finish.

Due to programming issues at the time 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.



Figure 4. Base pumping self compacting concrete into the 28m high reinforced concrete pylons

Curved concrete cross beam

The curved concrete cross beam was constructed after the bottom section of the pylons was poured. The crossbeam was difficult to construct from a geometrical point of view with the crossbeam also raked at 15 degrees to follow the pylons and had a curved soffit. The reinforcing connection to the concrete pylons was achieved using Reid Bar and couplers.



Figure 5. Curved concrete crossbeam which supports the bridge deck at the pylons

Second Pour Pylons

Once the concrete cross beam was constructed the reinforcing cage, pylon anchorage and portal beam were placed prior to the forms being sealed closed and the pylons poured. Over 12 months of detailed planning to construct the pylons came to a successful two hour conclusion with the pouring of the pylons to full height.

The staged stripping of the forms from the pylons revealed a high quality finish of the base pumped SCC. No segregated concrete or voids were found in the pylons.



Figure 6. Stripped top section of the pylons

Bridge Deck

Erection of the structural steel was the first task, with the positional placement of the structural steel of primary importance, due to the need for precise alignment between the top stay cable anchorages and the guide pipes and anchorages fixed inside the box girder. An insitu weld splice in the box girder was used to assist transportation and erection.

Temporary deck props were required to take the full load of the deck steel, concrete, precast footpaths and barriers prior to stressing of the stay cables. Due to compressible silt and organic soils overlying the mudstone, the temporary props were constructed using bored concrete filled steel casings founded into the mudstone.



Figure 7. Erection of eastern half of the box girder

Prior to construction alternative methods were considered to construct the concrete topping as a composite slab, using prestressed slabs as either a permanent form or part of the deck itself. The permanent form concept was rejected as it added an additional 900kN of deadweight to the bridge. Reinforcing continuity issues and clashes with shear studs in using a composite precast solution were too difficult to resolve. A decision was made to proceed with the insitu design utilising three deck pours.

Depth control of the concrete deck pour was very important to provide the required cover to the reinforcing without adding additional dead load to the bridge which would effect the cable stressing. A 10mm increase in overall slab depth would result in an additional 350kN of dead load.



Figure 8. Construction of the insitu concrete deck

Precast Footpath Units

The footpath units were precast and set on the steel outriggers into position before grouting the joints. Considerable effort was placed in achieving a high quality finish of the precast work and insitu grouting.

The use of clamped side lifters instead of standard face or edge lifters eliminated the need for patching any lifting eyes in the precast panels.

With the completion of the composite deck, footpath and vehicle barriers, the required dead loads were all in place prior to tensioning the stay cables.

CABLE STAYS

The bridge is stayed by 20 stays, with ultimate capacity of up to 780 tonnes, and with overall lengths between 20 and 53 metres.

At an early stage in the design process fatigue was identified as a significant issue. The BBR DINA system was specified because of its respected fatigue performance.

The DINA System

The DINA system is a development of the standard BBRv wire post tensioning system, which was used for post tensioning many of the concrete bridges built in New Zealand between 1960 and 1990.

The stays consist of 7 mm diameter wire, arranged in a compact parallel bundle within a thick walled HDPE sheath and filled with a flexible corrosion-inhibiting compound.

The individual wires are terminated at each end with a cold-formed button head which transfers the full load to the anchor heads.

Length adjustment and load transfer to the bridge structure is provided by lock nuts screwed onto the outside of the anchor heads. Grease filled corrosion covers are fitted to enclose the anchor heads following stay tensioning.

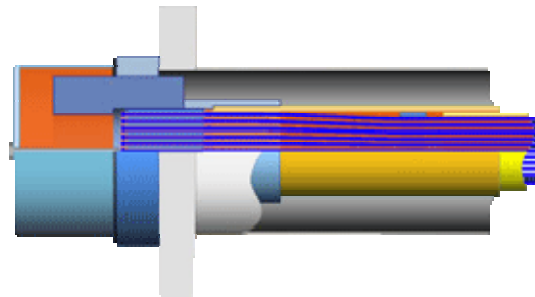


Figure 9. DINA stay cross section at end anchorage

Stay Manufacture

A decision was made to set up and manufacture the stays on site to avoid the complications of either coiling the stays onto large diameter reels or transporting excessively long loads by road. The area on the Western abutment was set up with a 60

m long wire cutting line and similar length assembly bench.

The first task was to butt fusion weld the HDPE sheaths to correct length.

The twenty stays consist of between 78 and 144 individual 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 to a pre set stop was installed. Wire was shot and cut to length within a tolerance of +/- 5 mm.

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

Once insertion of the wires was complete a comprehensive check is carried out prior to forming the button heads at each end of the wire. Assembly mistakes can not be easily rectified after forming of the button heads and usually the full set of wires need replacing to rectify.



Figure 10. Wire bundles inserted in stays prior to button heading. Note cutting line and pre-cut wires in the background and turbulence inducers on the HDPE duct in foreground.

The button heads are formed by gripping the wire and pushing 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.



Figure 11. Wires terminated with button heads at an end anchorage

When the button heads had been formed the HDPE sheath is connected to the end anchorages and the corrosion-inhibiting compound is injected to completely fill the HDPE sheath.

Cable Installation

The cables in completed form are extremely flexible and are susceptible to damage of the HDPE duct or the wires within, if bent excessively, and great care was needed during installation.

A number of options were 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 12. A stay being installed using a three point lift system

The cables were lifted and connected at the clevis, end first. The lower end was then inserted into the guide pipe through the main girder.



Figure 13. Inserting the clevis pin.

A single pre-stressing strand was attached to the lower end of the stay and a small capacity jack was used to pull the stay into a position where the main hydraulic stressing jacks could be utilised.

Cable Tensioning

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.

As the cable sag was large, cable tensioning had to commence during installation and prior to release of the cranes.



Figure 14. Stay tensioning arrangement. Note the small capacity jack being used to pull the pull rod through the large hydraulic jack.

The tensioning system consisted of 250 tonne centre hole hydraulic jacks tensioning the stays via a re-locatable 75 mm diameter, 450 tonne pull rod.



Figure 15. Stay tensioning arrangement. The 2 stage trestle allows the jack to be cycled and retain load in the stay.

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 cables 5 and 6 were stressed to relieve the weight of the concrete pylons off the temporary supports. The unloaded temporary supports were then removed before the first back stays were stressed. When the weight of the bridge deck was transferred to the stay cables during tensioning, the temporary deck supports were removed to avoid redistribution of weight back onto the deck props

The tension in any given cable is reduced as cables either side are tensioned as the weight of a specific section of deck is redistributed between adjacent stays. The consultants modelled the bridge and it was found that two cycles of stay tensioning would be required to avoid overstress to 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.

Acknowledgements

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Figure 17. Stay cable details and arrangement

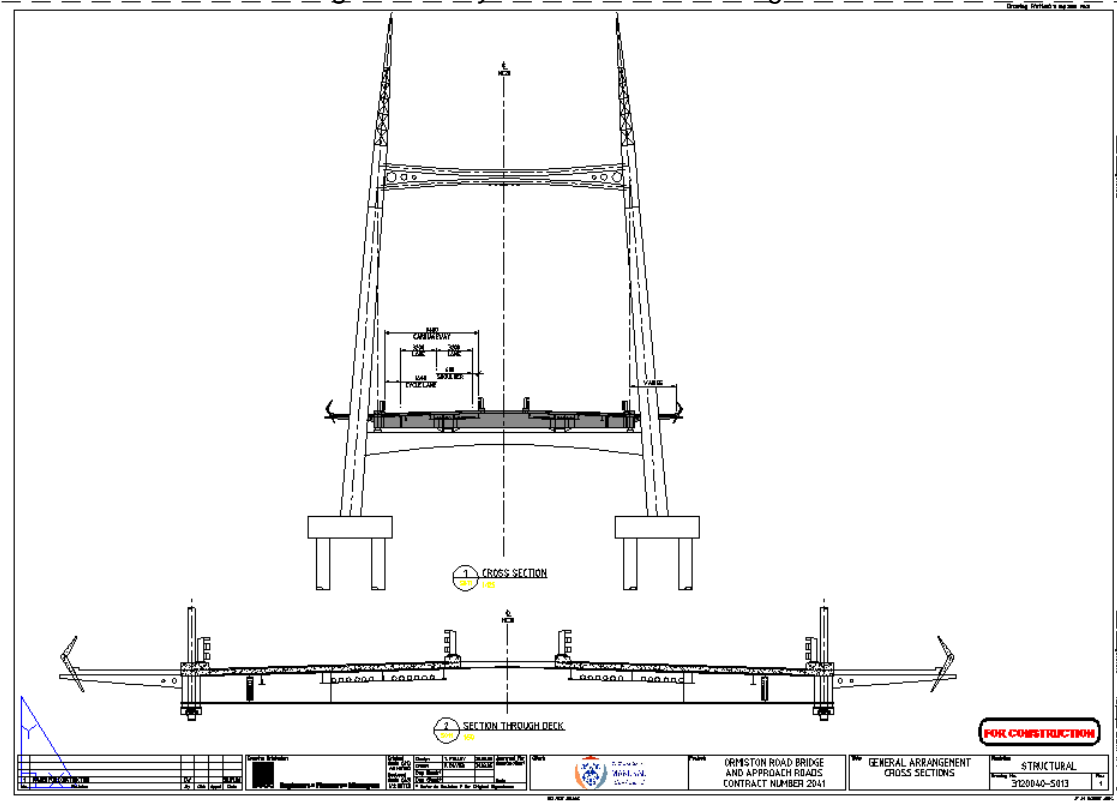


Figure 18. General bridge elevation

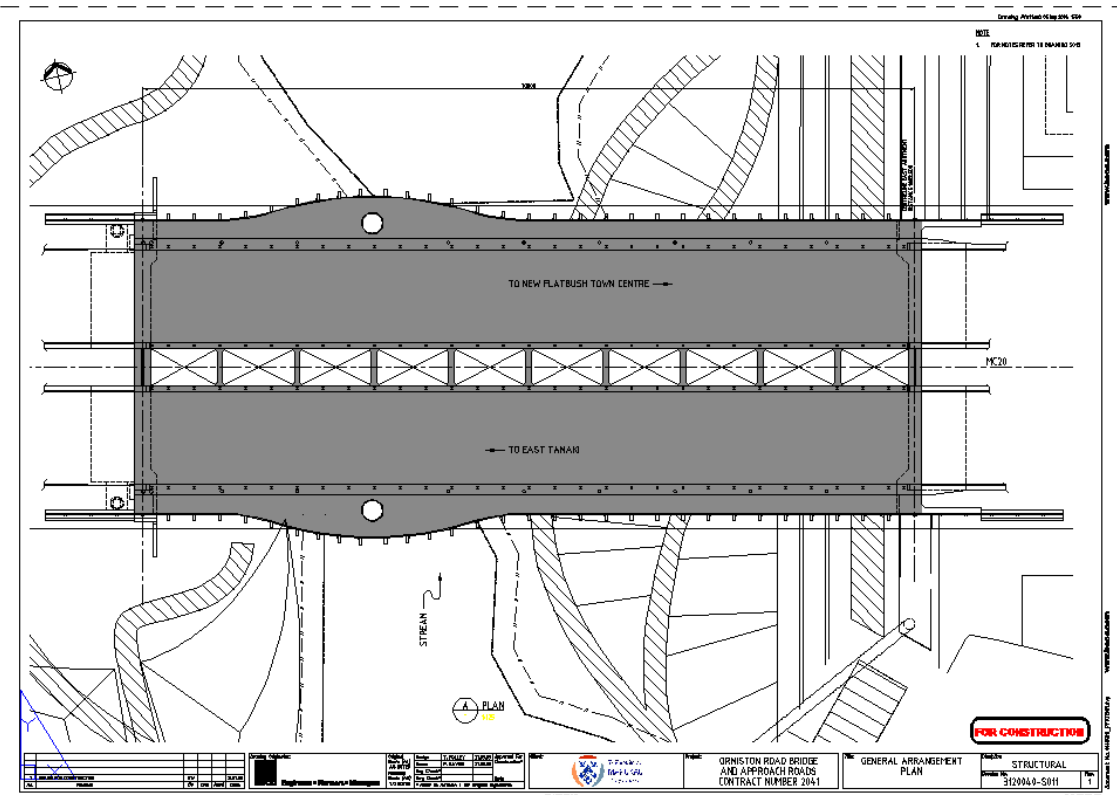


Figure 19. General Bridge Plan