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NEWMARKET VIADUCT REPLACEMENT PROJECT – END OF AN ERA

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INTRODUCTION

The existing Newmarket Viaduct was constructed in the 1960s and forms a vital link in Auckland's motorway network. The dual three lane viaduct carries over 160,000 vehicles per day and is part of the busiest section of road in New Zealand. The viaduct passes over a busy urban retail area and the North Auckland Railway line (refer to Figure 1).

Figure 1. Aerial photograph showing the new southbound viaduct under construction.

The combined effects of substantial traffic volume growth and the vital role the viaduct plays in the security of the motorway network, led to the decision for the viaduct to be replaced. The new viaduct provides additional lanes, improved seismic resistance, higher edge protection and increased vehicle load capacity. The key challenge was that the existing viaduct had to be deconstructed and replaced by the new viaduct with no impact on the peak motorway traffic capacity throughout the four-year project.

The critical link between the design and construction methodology resulted in the project being implemented as an alliance: comprising the owner (the NZ Transport Agency) three contractors (Leighton, Fulton Hogan and VSL) and four design firms (Beca, URS, T&T and Boffa Miskell).

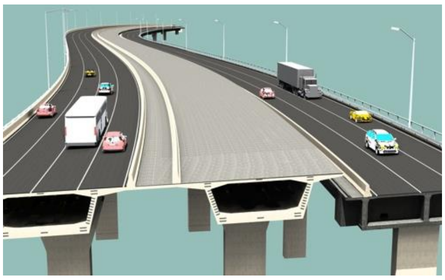
This paper describes the unusual challenges brought about by constructing a new viaduct in a multi-staged process while ensuring that the motorway remained fully operational during each stage. The deconstruction process is considered a world first and was aptly recognised by winning at the World Demolition Awards held in Amsterdam.

Key Constraint – Staging of the Works

The design of the new viaduct had to allow for a staged replacement of the existing viaduct; with the centre line of the new viaduct being shifted 13m to the east. This was to avoid major motorway realignment, land purchase, and to minimise disruption to the community. The construction and deconstruction work was carried out by a 810T overhead launching gantry. A detailed staging scheme was developed to enable the motorway to remain in full service throughout the project and the stages are illustrated below:

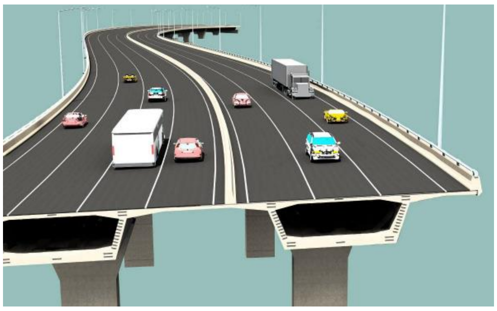
Stage 1 – Construct new southbound viaduct

Stage 2 – Commission new southbound viaduct and deconstruct old southbound viaduct



Stage 3 – Construct new northbound viaduct and pour insitu median stitch

Stage 4 – Commission new northbound, deconstruct old northbound and extend cantilever



Stage 5 – Final Lane Configuration

NEW VIADUCT

The new viaduct consists of two parallel post-tensioned concrete box girder structures; each constructed using precast match-cast segments and erected using the balanced cantilever method. The box girder is 3.2m deep with varying bottom flange and web thicknesses along the span. Each box segment is reinforced transversely and longitudinally. The longitudinal reinforcing is primarily for crack control since no reinforcing crosses the segment joints.

The box girder uses a combination of internal cantilever tendons and external continuity tendons to resist the applied loads. The continuity tendons are typically anchored over two spans. The cross section is shown in figure 2.

A cast insitu median stitch slab joins the two boxes together and forms a continuous deck 28.7m wide between the two high performance Test Level 5 (TL5) edge barriers. The viaduct will initially carry 4 lanes southbound and 3 lanes northbound, but has the capacity to run 4 lanes in each direction in the future. This can occur by shifting the median barrier and reducing shoulder and lane widths.

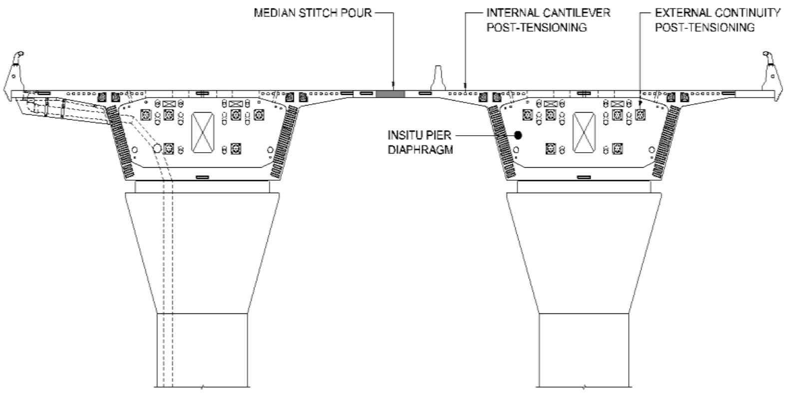


Figure 2. Cross section of the viaduct at a monolithic pier.

The viaduct varies in height above ground level from 8m to 23m. The 690m long continuous structure comprises 12 spans ranging from a 62m main span to 40m back spans as indicated in Figure 3. In plan view, the bridge is in the shape of an S with the smaller radius curve being 690m and the larger radius curve is 760m. Large steel finger expansion joints are located at each end of the viaduct.

The northern end of the bridge joins to a small Super Tee bridge due to limited vertical clearance over a main arterial route to Auckland's airport.

The site geology is complex and variable. Each end of the viaduct is founded on one of two separate basaltic lava flows that originated from volcanic cones on each side of the valley that the viaduct crosses. The two flows do not quite meet and the four piers between the lava flows are supported on piled foundations. These piled foundations are founded in competent Waitemata Sandstone which is approximately 25m below the ground surface.

Extensive grouting is required to fill fractures in the basalt for the shallow pad foundations located on the northern basalt flow. The southern end of the viaduct is supported on short piles bearing onto the southern basalt flow located approximately six metres below ground level.

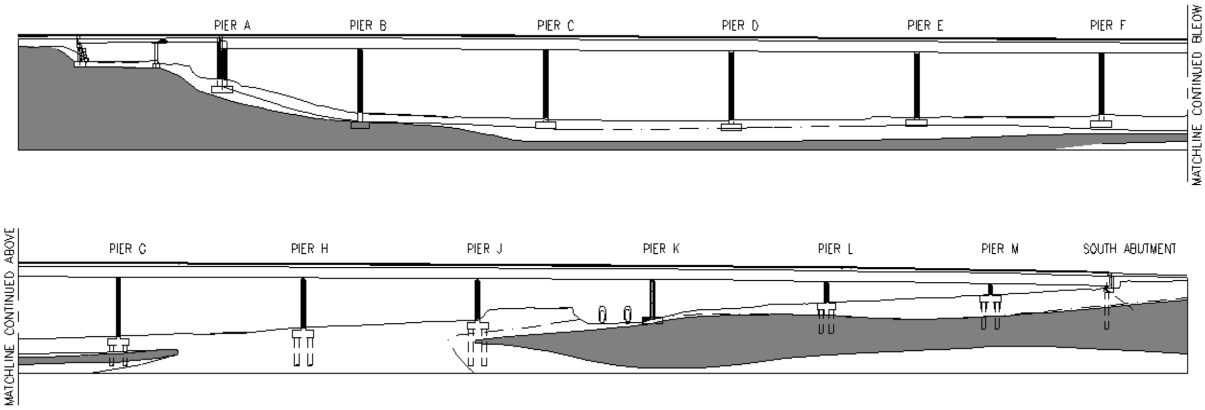


Figure 3. Elevation of viaduct showing extent and variation of basalt flows beneath.

Each box girder is supported on rectangular shaped reinforced column. The columns are flared at the top to match the width of the box girder soffit.

The varying column heights result in a combination of tall monolithic columns and short cantilever columns with longitudinally sliding bearings. Detailing the box girder continuous with the shorter columns would have required an expansion joint in the middle of the bridge to avoid longitudinally overstressing these columns. This was not preferred from a whole of life maintenance and an aesthetic point of view and would have resulted in shorter end spans midway along the viaduct.

The expansion joints are located at Pier A and at the South Abutment. Fixed pot bearings are on Pier B & Pier H. Sliding bearings are on Pier A and all piers from Pier J to the South Abutment.

Construction

The new viaduct was built by the balanced cantilever method using a 140m long 810 tonne launching gantry. The gantry was supported at each pier by placing the main support legs above the pier diaphragm and then used to construct the balanced cantilever span. The gantry launches to the next pier once completing a cantilever span.

During the launch the gantry's centre of mass passes over the cantilever span pier resulting in the largest axial demand on the substructure. The piers are designed to accommodate the transverse eccentricity of the gantry due to the curvature of the alignment and the significant gantry wind and seismic demands. The longitudinal fixity of the gantry was restrained by the previously constructed continuous spans. This avoids overloading the cantilever span pier that is being constructed.

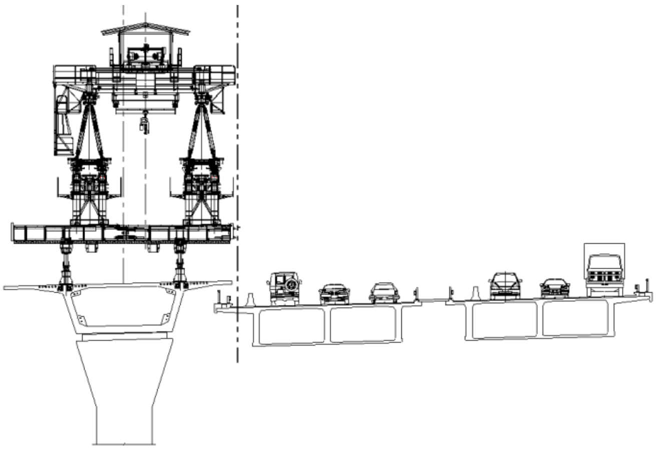


Figure 4. New viaduct (on left) with launching gantry

Due to the close proximity of traffic, transverse traffic impact had to be considered as a high risk possibility, impacting the erected cantilever span before it was made continuous, potentially collapsing the span with the gantry above. The original traffic barrier on the existing bridge could not resist this load due to strength limitations of the existing structure. This risk was minimised by adding a temporary barrier and adding impact struts between new and old bridge deck.

The bridge at 690m in length is close to the maximum length for this configuration of piers and specified form of expansion joint. Much longer and the bridge would require an intermediate expansion joint due to the large shortening demands.

The construction staging required the first box girder to be constructed with the second box girder being constructed one year later. The two box girders were then joined by a cast insitu median stitch. The median stitch restrains the second box girder which is trying to creep and shrink at a faster rate than the first box girder. The result of this effect is the second box girder prestress will tend to creep across to the first box girder due to the restraint provided.

This meant that the second box girder required more continuity prestress than the first box girder even though the effective cross sections are the same.

The two box girders are made as one by the insitu median stitch, which results in a deck 28.7m wide. The relatively large deck transverse temperature and shrinkage movements meant only one of the four bearings at a pier could be transversely guided. Using one guided bearing per box girder (ie two guided bearings per pier) would have developed significant transverse shears for the short piers since both columns would be resisting the transverse shortening effect. During construction, temporary transverse restraint were used on the northbound bridge since it was supported on two free sliding bearings.

Seismic Philosophy & Loading

The viaduct is such a critical element of New Zealand's motorway network that it is designed for much larger seismic loads than other bridges. The design Ultimate Limit State (ULS) elastic earthquake level has a return period of 2500 years for this bridge.

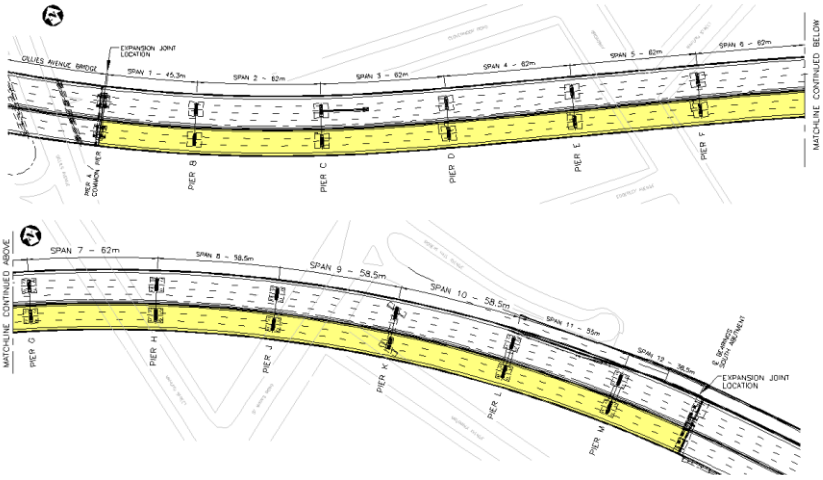


Figure 5. Plan View. Box 2 (shaded) will lose prestress, creeping across to Box 1.

The consequences of a failure during construction due to the close proximity of the existing viaduct led to consideration of increased loads. A fully constructed cantilever span with the launching gantry on top was designed to resist an elastic earthquake with a return period of 1000 years. This is the same seismic return period as most bridges have to resist in their completed structural form.

The bridge is designed for a ductility factor of 3 in the longitudinal direction and 1.25 in the transverse direction. A lower ductility level was used in the transverse direction due to the proportion of column size relative to the column height, ie dimensionally the shorter piers were more like a shear wall than a column in the transverse direction.

One bearing out of the four bearings at each pier (ie two bearings per box girder and one box girder per column) resists the SLS transverse loads. However this bearing is not strong enough to resist the ULS transverse lateral load of two box girders combined. This issue was overcome by detailing a steel shear key to be cast into each column for the piers with bearings.

The guided bearing has been detailed to yield in a ductile manner under ULS transverse loads. In this situation, the bearing transverse restraint guide yields and can no longer transfer transverse shear. This transverse guide had to be independent from the bearings vertical support capacity and needed to be detailed in such a manner that it would not damage the bearing's friction surface during yielding. The bearing can then continue to transfer vertical loads into the pier. The steel shear key will then transfer the ULS lateral loads. The rest of the bearing elements are detailed to be stronger than the over strength demand of the ductile restraint guide so no damage occurs to the bearing.

Insitu Median Stitch

The median stitch was the 1200mm wide insitu concrete section which joined the northbound and southbound box girders to become one viaduct. The median stitch pour occurred while traffic was already on the southbound viaduct but no traffic on the northbound box girder.

The effect of vibration on reinforcement bond strength was a concern since the median stitch was formed with lapped reinforcing from each box girder. Research found that vibration was no longer detrimental to concrete bond once concrete strength was greater than 10MPa.

Various concrete mixes were developed and tested until one was found to achieve 10MPa early strength within 8 hours but resulted in the concrete rapidly setting on site and becoming unworkable. Constructing the 690m long median stitch in one night required five concrete teams working rapidly with concrete trucks arriving every ten minutes to avoid a cold joint forming in the stitch pour. The planning and coordination for this critical pour took months to organise due to the potential impact this could have on the motorway traffic.

Pier Diaphragms

The combination of using a precast box girder shell, cast insitu concrete diaphragm and using external post tensioning resulted in the diaphragm being extremely complex and required a significant amount of reinforcing.

The diaphragm must be able to transfer large transverse and longitudinal seismic shears from the superstructure to the substructure. It must also be able to transfer the continuity prestressing anchorage force. The diaphragm design was critical during construction when the tendons are anchored on one side of the diaphragm. At this stage the tendons try to push the insitu diaphragm through the precast girder shell during the stressing. This action is resisted by shear friction reinforcement which crosses the interface of the precast and insitu sections.

The diaphragm also has a large access tunnel (1.2m wide by 1.6m tall) passing through it. This tunnel, combined with the voids from the external tendons represented a potential weakness in the diaphragm.

Self-compacting concrete was the only option for constructing these diaphragms. The congested reinforcing and limited access meant that constructing the second stage pour with normal concrete carried too high a risk that either segregated concrete or a concrete cold joint would occur.

DECONSTRUCTION

The deconstruction process involved making irreversible changes to the existing structure, systematically removing elements and weakening the viaduct. In addition, the remaining structure was required to carry live traffic as well as supporting the 810 tonne launching gantry used for the deconstruction. Therefore robust controls on deconstruction activities were essential to avoid a situation where the remaining parts of the structure could no longer safely support the required loadings.

Figure 6. Deconstruction of the southbound viaduct.

It was appreciated early on that flexibility would be required in the deconstruction sequence to mitigate the impact of site issues and allow any opportunities to be exploited. However, this aim was tempered by the need to have the deconstruction sequence adequately defined before the work commenced.

Programme and resources dictated that only one of the many possible sequence permutations available for deconstruction could be fully investigated to determine design loadings on the various propping systems and assess the release effects when span cuts were carried out.

This analysis was used as the basis for design of the temporary works elements. The critical load on any given prop depended on the deconstruction stage and location at which it was introduced into the remaining structure and the position of the launching gantry at any time. In addition, loads on the vertical prop/shield beam systems were further modified by using them as platforms to jack forces into the superstructure prior to making initial span cuts in order to minimise release effects.

The agreed deconstruction sequence was shown on a set of drawings in which each stage had a unique identifier. Hold points were specified before key events, such as gantry launching and initial span cuts. This approach provided both a framework for on site monitoring of progress and, also, a clear unambiguous ‘language’ for considering and communicating sequence changes in response to developments on site.

When considering requests for change to the specified deconstruction sequence, an assessment was made of the variation from the baseline analysis that would result from the proposal. After adjustment for actual monitored loads vs. predicted service loads, this variation was compared to the available ‘reserve capacity’ in the affected temporary works. A risk-based assessment was then applied to make the final decision.

For instance, if a proposal placed demands on a prop that left insufficient reserve capacity for a high wind ULS event, permission was granted on the basis that an adequate 48 hour ‘weather window’ existed for that operation only. Approval was communicated by updating the deconstruction sequence drawings and introducing a hold point, requiring sign off at a senior engineer level, prior to commencing the critical operation.

Temporary Works

One of the first operations carried out during the deconstruction of the viaduct was to cut the pier diaphragms that connected the southbound bridge to the northbound bridges. Whilst the 'global' stability of both superstructures was ensured by the temporary works systems illustrated in Figure 7, uncertainty existed around the performance of severed tendons re-anchored by the existing duct grouting.

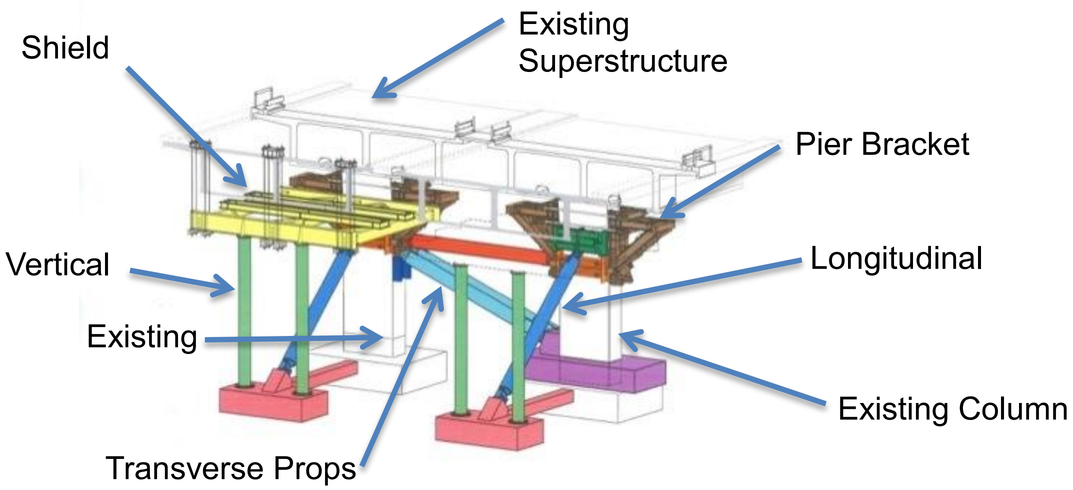


Figure 7 – Temporary works at bridge piers

The pier diaphragms formed a critical part of the load path from the superstructure into the columns. Pier brackets were stressed onto the column faces, as shown, to provide an alternative load path at the ultimate limit state and to allow jacking to remove the dead load effects from the pier diaphragms under service conditions. This approach reduced the risk of failure of a pier diaphragm and minimised displacements should such an event occur.

Investigation and Testing During Deconstruction

During the design phase and initial stages of deconstruction, extensive testing was carried out to justify design assumptions and provide reliable values for design parameters. For example, full-scale tests were performed to determine the friction capacity of the connection between pier brackets and columns using different combinations of grout and roughened steel plates. The results of these tests were incorporated directly into the design after the application of appropriate strength reduction factors.

The south end of the viaduct, being relatively close to the ground, was earmarked to be supported on falsework during demolition. This allowed proving tests[1] to be carried out on a number of critical techniques and systems before the launching gantry was irreversibly committed to working on the existing viaduct.

In view of the importance of the condition of the pre-stressing tendons and effectiveness of the original duct grouting to every aspect of the deconstruction, a scheme of investigations was implemented using a hierarchical approach, as follows:

- At the lowest level, ground-penetrating radar (GPR) was used to compare the actual locations of a representative sample of tendons with the as-built drawings.
- The exact location of a sub-set of this sample was confirmed by drilling 25mm diameter pilot holes. These holes also provided an indication of the condition of the post-tensioning sleeves and whether grout was present in the ducts.

- Enlarged intrusions were then made into a selection of ducts and to expose anchorages to confirm the condition of the grout and individual post-tensioning wires.

A basic statistical analysis of the results provided a measurement of tendon variance from assumed positions. This was used to bound the assessment of remaining structural capacity and load effects released during initial span cuts.

Supplementary investigations were carried out where the 'fully bonded' capacity of specific tendons was relied upon for any given operation. The investigations generally found that grouting of the post-tensioning ducts was effective. However, a significant number of ungrouted / partially grouted tendons were observed as segment removal progressed, requiring a range of remedial measures to be taken.

Span Cuts and Closure Segment Removal

Removal of the mid-span closure segments was generally carried out in advance of arrival of the launching gantry at that span. The 'Closure Segment Deconstruction System' (CSDS) was a combined arrangement of equipment and procedures. The CSDS was developed to safely release locked-in loads from the superstructure when the initial span cuts were made, and then lower the closure segment to ground level.

Figure 8 - Removal of closure segment over Broadway, Newmarket

The philosophy behind the system was fairly simple. Firstly, jacks acting in the plane of the top slab of the box girder were used to decompress the concrete in that region while the upper part of the superstructure was cut. The jacks were then gradually depressurised to release the bending moment from the bridge. At this point a set of 'stitching beams' were engaged to control the release of shear forces as the final cuts were made through the cross section. Finally, after a second cut was completed to release the closure segment, strand jacks mounted on the stitching beams were used to lower the closure segment to ground level.

In practice, a thorough understanding of the tendon geometry and possible upper and lower bound release effects at each cut location was required to ensure that sufficient deck jacks were installed to release the bending moments from the superstructure. Strict limits were also required on the amount of force applied by the jacks to ensure that decompression of the entire section was avoided as this could have caused an uncontrolled release of shear before the stitching beams were engaged. It was not possible to fully engage the stitching beams prior to 'moment release' as the resulting superstructure displacements would have over-stressed a number of key elements within the CSDS. Failure of the CSDS would have resulted in a dynamic load impact, overloading the temporary works and collapsing the pier and the gantry supported above.

Segment Removal

Segments were generally removed in opposing pairs in order to keep the piers in balance. After being stressed up to the launching gantry, or mobile crane, each segment was released from the superstructure by cutting along its interface with the adjacent segment using wire-saws before being winched to ground level.

The form of post-tensioning anchorage used in the viaduct was such that a single wedge anchored all 12 strands in each tendon. The anchorages were typically located in the box girder webs at the interfaces between segments; therefore the consequence of damage to an anchorage during cutting was a significant concern. Generally the risk of premature release of the segment adjacent to that being removed was mitigated by passively hanging it from the gantry. Where this was impractical, the risk was, either, eliminated by installing guides for the wire-saws, or, providing redundancy by other means.

Monitoring of Prop Forces

Monitoring of forces in the temporary props was carried out at critical deconstruction stages, such as during gantry launching and before and after cutting of each span.

Ultimate limit state events including the live load, earthquake, wind and temperature cases generally governed the design of the combined structural system of remaining structure and temporary works at each stage of the deconstruction. It was not practicable, however, to base the monitoring regime on these extreme events. The method adopted was to track the actual demands on the props against the envelope of service loads, extracted from the baseline analysis, for each prop at each deconstruction stage.

Figure 9. Deconstruction gantry supported on vertical props during pier segment removal.

This approach provided early warning of situations where inadequate reserve capacity might occur for the design events, so that corrective action could be taken.

As the project progressed, opportunities were identified to maximize use of the deconstruction gantry in order to reduce reliance on mobile cranes. For example, a method was developed of using the gantry during its launch cycle to remove the pier-cap segment that had, up to that point, supported its rear leg.

This was achieved by supporting the gantry on the shield beam/vertical prop system while cutting the pier-cap segment (up to 120 tonnes) free of its column and lowering it to the ground using the gantry winch. Implementing this involved significant changes to the gantry kinematic sequence that pushed both the gantry and the propping system to their limits of safe operation (80% of ultimate capacity in the case of the propping system).

Strict controls were placed on this activity including proof loading the pair of vertical props before cutting the pier-cap segment free to confirm that both props would attract equal load during segment removal. Continuous live loading of the props was also carried out during the lift to allow corrective action to be taken in the event that uneven loading of the props started to develop.

Temperature Measurements

Differential temperatures within the existing bridge decks were also monitored, particularly during critical deconstruction operations such as the initial cuts to each span, where temperature constraints were necessary to ensure that released forces could be controlled by the CSDS.

Temperatures were recorded at a number of locations on the bridge. At each location two thermocouples were embedded in the top slab and one in the bottom slab of the box girder. This arrangement permitted the temperature gradient in the top slab to be extrapolated as well as measuring the ambient and differential temperatures in the bridge superstructure.

During the summer months, good agreement was noted with the differential temperature profile given in the NZ Transport Agency Bridge Manual. The peak temperature difference typically occurred around 4PM. A significant reverse differential temperature profile was also recorded, which tended to reach its peak at 7AM. Analysis of the bridge, carried out for the deconstruction indicated that the latter effect, combined with the prestressing tendon arrangement near the bridge piers, was responsible for some of the initial problems with opening joints in the original viaduct.[2]

Monitoring of Structure Movements

Survey prisms were attached to the bridge superstructure during deconstruction in order to monitor movements during critical operations such as span cuts and the gantry launching cycle. Typically these were located adjacent to the span closure segments, above the vertical props and at the column locations. Measurements were taken using a total station theodolite at designated points in each operation and compared to pre-defined alert and alarm criteria and baseline readings taken at the start of deconstruction.

In addition, monitoring of the foundation pads was carried out during the gantry cycle. This was considered prudent in view of the highly variable nature of the geology below the viaduct and because the bridge foundations were expected to receive up to 40% more loading than they had previously encountered. Accordingly, monitoring points were established on each column base pad, primarily to provide early warning of gross settlement so that mitigating action could be taken as necessary.

CONCLUSION

The success of this exciting and unprecedented project is a testament to the skill and integrity of the participants. The authors wish to acknowledge the support and contribution of the New Zealand Transport Agency in the success of this project, as well as the other project participants – Leighton, Fulton Hogan, VSL, URS, Beca, Tonkin & Taylor and Boffa Miskell.

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