DESIGN AND CONSTRUCTION OF THE GREAT NORTH ROAD INTERCHANGE RAMP VIADUCTS

TERRY CHENG, WILL DOBSON, LAWRENCE BLACKMORE, ANDREW DICKSON

Well-Connected Alliance, New Zealand

SUMMARY

One of the dominant features of the NZ Transport Agency’s Waterview Connection project in Auckland is the Great North Road motorway to motorway (SH20 to SH16) interchange (GNRI). The interchange ramps include four separate significantly curved viaducts varying in height (up to 20m above SH16) and ranging in length from 225m to 500m. The viaduct superstructures utilise precast Super-Tee girders made fully integral with all substructures apart from abutments and a portal pier at the mid-length of one viaduct.

With design complete and construction of the viaducts well advanced, the first part of this paper discusses the design of the ramps and offers a designer’s perspective on integral Super-Tee bridge design in the New Zealand environment, outlining key aspects of analysis and design methodology. The second part of this paper describes the construction of the ramps, including construction challenges, construction methodology and construction details developed to optimise constructability.

The paper concludes with lessons learned from the design and construction of these significant structures and thoughts on how New Zealand practice, including the requirements of the NZ Transport Agency Bridge Manual, could be improved to more effectively make use of this durable and resilient construction form.

OVERVIEW OF GREAT NORTH ROAD INTERCHANGE

The NZ Transport Agency’s $1.4 billion Waterview Connection is the largest road project ever undertaken in New Zealand. The project involves construction of 4.8km of new motorway which links SH16 and SH20 together to complete the Auckland Western Ring Route. It will relieve regional congestion and provide a direct link between Auckland CBD and the International Airport. The project work includes two parallel 13.1m diameter tunnels 2.4km long, two cut and cover tunnel approach trenches, and four interchange ramps where the motorway emerges at Great North Road. The project is being delivered by the Well-Connected Alliance which comprises the Transport Agency, Fletcher Construction, McConnell Dowell Constructors, Beca, Parsons Brinckerhoff, Tonkin and Taylor and Japanese construction company Obayashi Corporation.

The Great North Road Interchange ramps comprise four separate viaducts connecting the SH20 tunnels with SH16. Figure 1 shows more detailed information of the bridge layout.
DEVELOPMENT OF STRUCTURAL FORM

The form and location of foundations of the ramps was limited by the existing SH16 alignment and the adjacent coastal marine area. A single pile - single column pier type foundation was the prescribed solution in the coastal marine area and near existing carriageway, so it was adopted throughout for aesthetic and construction consistency.

Geometric constraints dictated by the tunnel vertical alignment and the proximity of SH16 meant that the headroom envelope available for the Great North Road Interchange Ramps was highly constrained. This precluded the use of deeper structural forms such as ladder beam decks and post-tensioned box girder construction. Multi-girder steelwork options were considered but ultimately a Super-Tee girder construction was adopted as the most cost effective solution.

Having made the decision to adopt a Super-Tee beam deck an exercise was undertaken to compare the merits of simply supported and integral construction through the preliminary design process. It was found that the integral option did not offer significant improvement in span length and reduction in girder size. Accounting for the relevant construction factors at the preliminary design stage, from a cost perspective, neither solution gave a significant saving over the other. However, because the Project Requirements and Minimum Standards indicated that integral construction should be adopted wherever practical, a fully integral solution was chosen.

The superstructures comprise spans of approximately 33m with 5 or 6 precast, prestressed Super-Tee beams with cast in-situ concrete slab. The precast Super-Tee beams are 1525mm deep and vary in width across the top flanges to suit the geometry of each location. The superstructures are cast integrally with reinforced concrete pier cross heads supported on single reinforced concrete columns and single bored piles. At the abutments, Super-Tee girders are supported on pot bearings sitting on the abutment beam to allow movement along the longitudinal axis. Shear keys are provided at the abutment as transverse restraint to the superstructure. The abutments are supported on groups of bored piles with MSE walls or batter slopes forming the approaches. Typical structural form of the bridges is shown in Figure 2.
Integral pier construction leads to the need to temporarily support the Super-Tee girders before the integral connection between the pier and superstructure is made. Due to the height of the superstructure above ground and the limited space available for construction, it was not feasible to support the girders from the ground. Steel temporary work attached to the pier column was considered but ruled out as being too expensive and risky, especially when considered in conjunction with a gantry erection methodology. A two staged crosshead construction was adopted. The first stage down-stand acts as a temporary support to the girders. The second stage in-fill pour stitches the superstructure together with the pier to complete the integral connection. One of the crosshead down-stands has been made shallower than normal due to headroom constraints. As a result, temporary posttensioning was put in to provide the required strength in the temporary state.

Constraints of pier location as Ramp 4 passes over SH16 resulted in two 44m extra-long spans, well beyond the limit of 1525mm Super-Tee girder (1800 Super-Tee's were not readily available at the time). Three table-top pier crossheads (13m & 8m in length rather than the standard 2.6m) were introduced to allow the superstructure to span the 44m length. In the case where the existing or new motorway is directly below the ramps, sometimes it is not possible to put a single column pier at the location needed. As a result, two special portal piers have been designed to support the superstructure, one at Ramp 2 and the other one at Ramp 4. The Ramp 2 superstructure is supported on bearings on a post-tensioned portal beam spanning across the SH16 on-ramp. Similarly, a reinforced concrete portal beam cast integrally with the superstructure carries Ramp 4 over Ramp 3 bridge underneath.

Long integral bridges like the GNRI Ramps experience large demands due to creep & shrinkage shortening and thermal movements, particularly in the end piers. The long construction period of the whole project provides an opportunity to take steps to minimise shortening demands. Selected piers have been left un-stitched with the superstructure over a period of time. Instead of shortening as a 500m long bridge, the presence of un-stitched piers allows the creep and shrinkage shortening to take place in several short pieces over a period of time and reduces overall shortening demands significantly. Also, some of the piles towards the ends of the structures are formed with a permanent annulus around the upper portion of the pile to increase their flexibility and reduce shortening demands on the pile, column and crosshead.

**DESIGN CONSIDERATIONS FOR INTEGRAL CONSTRUCTION**

The adoption of integral construction for bridges incorporating prestressed precast Super-Tee type girders is becoming increasingly common in New Zealand. The GNRI ramps make
extensive use of integral Super-Tee construction. Despite the recent prevalence of this construction form in New Zealand, there are no standard details or design guidelines available. It has only been since May 2013 that the NZ Transport Agency’s Bridge Manual (NZTABM) has provided some limited guidance specific to the design of such structures. Some variability in practice is apparent.

Integral construction eliminates bearings, bearing shelves, shear keys and their associated inspection, maintenance and replacement. It also offers aesthetic benefits from the elimination of the above items. Structurally, integral construction appears to offer the benefits of continuity for live load effects, plus the advantages of the superstructure participating with the piers and/or abutments in longitudinal frame action to resist seismic loads and braking and traction forces. On the other hand, analysis and design of an integrally constructed bridge is considerably more complex than that of a bridge using simply supported spans, which means that it takes longer and costs more. Key aspects of analysis and design methodology associated with integral Super-Tee bridges are discussed further in the design section below.

**STRUCTURAL ANALYSIS**

The Great North Road Interchange ramps are long, curved prestressed concrete bridges with integral piers. Compared with simply supported bridges with superstructures free to float on bearings or semi-integral bridges where girders do not have moment continuity, but are pinned to the supports, the integral feature introduces more complexity in the structural analysis. A more comprehensive structural model of the entire bridge is required. Integral bridges generally need to be analysed as a whole to correctly model the interaction between superstructure and substructure. More potential critical load cases need to be considered which might be ignored for simply supported structures, for example, long-term creep and shrinkage effects. Due to the restraint of the superstructure at the integral connection, secondary demands develop from residual creep, differential shrinkage and differential temperature loads. Integral staged construction using precast prestressed components such as Super-Tee girders with a cast in-situ deck introduces a further level of complexity.

**Modelling**

Modelling of the interchange ramps was carried out using software package CSi Bridge. A full 3D model of the entire bridge was created for each ramp with the basic geometry imported from CAD models. Variations of the main model were generated to analyse various effects such as staged construction. Substructures were modelled using frame elements in CSi Bridge. Bearings and transverse shear keys were modelled using link elements oriented to suit the movement direction. Winkler springs at 1m intervals were used to represent the horizontal interaction between the structure and the founding soil. A screen-shot showing part of the full CSi Bridge model is shown in Figure 3 next page.

**Loadings**

Loadings were derived in accordance with the NZTABM and the Project Requirements and Minimum Standards. Analysis for most load effects was little or no different to that for any other long curved bridge of integral construction. In addition to gravity loads, creep and shrinkage, settlement, temperature and seismic loads can generate significant flexure and shear demands in both superstructures and substructures of integral bridges. Particular consideration is given in this paper to the analysis of creep and shrinkage effects which is of particular importance for an integral construction Super-Tee girder bridge. Significant effort has also been put in to the analysis of construction load effects.
Dead load and superimposed dead load

Dead loads were calculated based on the weight of the structure applied in stages to account for the construction sequence. Dead load of the precast girder and in-situ deck was carried by the precast girder unit when it was simply supported before being made integral with the pier crosshead. Superimposed dead load including barriers, surfacing and services were applied in a similar way as dead load and were applied after the super-T girders were made integral at the pier.

Creep and shrinkage

The effect of creep and shrinkage on composite Super-Tee girder bridges includes global shortening, residual creep effects of dead load and prestress, and differential shrinkage effects. These effects were analysed in stages to allow for the staged construction effect.

Global creep and shrinkage shortening

For long integral bridge structures like the Great North Road Interchange ramps, shortening effects generate large moments and shears in the piers near the abutments, as well as significant sagging or hogging moments in the Super-Tee girders adjacent to the end piers. Also, due to the curvature of the bridge, global shortening of the superstructure causes transverse bending in the central piers which is additive to load effects such as transverse seismic and eccentric live load. Because of the efficiency of the Super-Tee shape, the average compression due to prestress is relatively high compared with other prestressed concrete bridges and hence creep shortening effects are significant.

Effects of residual creep and differential shrinkage

Residual creep and differential shrinkage are the creep and shrinkage effects that occur after a precast member is made composite and/or continuous. These effects were considered in accordance with AS 5100.5:2004 Appendix E, using creep and shrinkage parameters calculated as per RRU Bulletin 70 (used as the basis for creep and shrinkage in NZ bridges until mid 2013). Residual creep and differential shrinkage effects in a composite continuous structure comprise two parts:

- internal stresses developed within the composite section due to the redistribution of stresses between the precast prestressed girder and the cast in situ deck. These stresses are applicable to both simply supported and flexurally continuous structures.
- external continuity restraint effect which is the secondary effect due to support restraint in a continuous structure. This effect only applies to flexurally continuous structures.

For the Great North Road Interchange ramp superstructures, the effects of differential shrinkage were found to be relatively small. The effects of residual creep on shear force and axial load were also found to be relatively minor. However, the flexural effects of residual creep were found to be significant. The effect of residual creep in a structure that initially carries load in a simply supported state, but is made continuous, is that over time the effects of both dead load and prestress tend towards those for a continuous structure. In a structure consisting of precast girders and a cast-in-situ deck that are made composite, the effects of residual creep include those of differential creep, meaning that stresses also tend towards those that would occur if the full load was applied to the composite section. Secondary continuity restraint moments from residual creep of dead load and prestress on a typical end and internal span are illustrated in Figure 4.

![Figure 4: Typical long term dead load and prestress moments](image)

The effects of residual creep on dead load moments will not surprise most designers. However the effects of residual creep on prestress forces will be less familiar, generating positive secondary moments similar to those caused by differential temperature. For the Great North Road Interchange bridges, and probably for most integral construction Super-Tee bridges, the net result of residual creep of dead load and prestress was the development of positive (sagging) secondary moments. These both detract from midspan moment capacity (offsetting
the advantages of continuity for live load) and mean that design for sagging as well as hogging moments is necessary at the piers.

This outcome is intuitively correct for girders that are not ‘dead load balanced’, meaning that the prestress moment in the bare section is greater than the dead load moment and thus the girder wants to continue to hog upwards, but is restrained from doing so by the continuity. It is initially less apparent for a dead load balanced girder – but even in this case the effects of differential creep between the girder and deck will force a hogging upwards which will cause positive restraint moments.

It is worth noting that the magnitude of the secondary moments that develop is directly related to the assessed residual creep factor for the period commencing at the establishment of continuity. The NZTABM 3rd edition has departed from previous NZ practice by referring to AS3600 for creep factors instead of to RRU70. This appears to give lower creep factors.

It is in the assessment of residual creep moments that the analysis of integral construction bridges utilising Super-Tee girders differs from that of other Super-Tee bridges and other prestressed concrete bridges.

Construction Load

Significant design effort has been put in to analysis of and design for construction load effects on the incomplete bridge structure during construction. The majority of construction load effects were a result of using a self-launching gantry to erect the Super-Tee girders. The substructure including pier crosshead down-stand, column and pile needed to carry the self-weight of Super-Tee girders, wet deck concrete and erection gantry loads. The construction sequence required the heavy Super-Tee delivery truck to run on the complete span deck to deliver girders to the gantry. The girder delivery truck was also required to run across incomplete crossheads where the crosshead infill stitch pour was delayed to reduce shortening demand. Since access from the ground was very limited, some of the construction activities were carried out from the bridge deck which required cranes or other construction plant to be set up on the incomplete structure. As a result, many additional construction live load cases have been checked. As the constructors continuously “value engineered” their construction methodology, analysis and design review of construction load effects became an ongoing activity as construction progressed.

In some cases, temporary falsework was needed to carry construction loads. Design of temporary falsework required inputs from both temporary works and permanent works design teams. The temporary steel falsework at ramp 4 reinforced concrete portal was one of the most significant temporary works of the GNRI ramps. This twin steel I beam system was supported on steel corbels stressed on to two permanent pier columns. It was designed to carry the self-weight of Super-Tee girders, the deck in adjacent spans, wet concrete of the portal beam and the erection gantry.

Another significant temporary falsework was the prop to the large table top pier at Ramp 4 pier 10. Considerable analysis and design input was required due to the complexity of erection sequence at the table top spans. The props were designed to work in both compression and tension to limit pier rotation during girder erection and deck concrete pour. They were also required to stabilise the pier under extreme wind and earthquake events.

**DESIGN METHODOLOGY, CRITERIA AND OUTCOMES**

This section discusses key aspects of the design methodology and outcomes of the Great North Road Interchange bridges with a particular focus on issues related to integral construction Super-Tee bridges. Making the bridge superstructure integral with the supports
introduced more complexity in the design than for a simply supported structure. Firstly there were more critical load cases which made the load combination process more complicated. Both short term and long term loading situations needed to be considered because one situation might be more critical than the other for the design of a particular element. Tension force could also develop in the superstructure which needed to be considered with bending, shear and torsion. Secondly, there were more design items to consider - in particular, the design of the integral joint between substructure and superstructure and the design of the region of the Super-Tee girder near the support for both hogging and sagging.

**Substructure design**

The Great North Road Interchange substructures were designed as reinforced concrete elements in accordance with the relevant SLS and ULS design requirements in NZS3101:2006 and the NZTABM. A large number of pier columns and piles, particularly the ones towards the ends of the bridge, were found to be governed by static load combinations. This was mainly because of the significance of shortening effect acting together with eccentric traffic loadings. Selected piers were left as non-integral for a period of time to reduce creep and shrinkage shortening demands. Annulus sleeving of piles through the rock was also introduced at critical locations to provide more flexibility and reduce shortening effect on the piles. Seismic and construction load combinations tended to govern some of the tall piers towards the centre of the bridge. The effect of column slenderness was assessed in accordance with the requirements of AS 5100.5:2004, Section 10.

Pier crossheads were constructed in two stages with the stage one down-stand acting as a temporary supporting beam to the precast girder, wet concrete of the deck and the erection gantry load. The design of the crosshead downstand was governed by the construction load combinations. Starters which formed part of the complete crosshead vertical reinforcement were cast into the down-stand. The rest of the pier crosshead was cast together with a portion of deck to complete the integral connection.

The design of the integral joint between Super-Tee girder, pier cross head and pier column was complicated because of the interaction between the elements under various critical load combinations. Moment, shear and axial loads at the end of the Super-Tee girder were transferred into the crosshead then down to the pier column. Joint design was carried out using a strut-tie approach. Joint reinforcement was detailed to adequately transfer loads across the joint as well as satisfying constructability requirements.

**Superstructure design**

The Super-Tee girders were designed as either a Class C (cracked) prestressed member or reinforced concrete member at various locations along the span. SLS and ULS design was in accordance with the NZTABM, NZS3101:2006 and the Project Requirements and Minimum Standards. Key SLS design criteria included limitations on crack width, steel stress range under transient load effects, maximum SLS steel stress and SLS concrete compressive stress. The key design criteria for Class C prestressed members and reinforced concrete members are summarised in the following table 1. The girders were also designed to have no tension at mid span under permanent load effects.

NZTA Bridge Manual load combinations are quite different to those in AS 5100. Combination 1A is the primary traffic load case, with only traffic load and permanent effects. Of the other load combinations, the most important for continuous superstructures is usually combination 2A, which combines traffic load effects with global and differential temperature effects. In the SLS combinations, all three are taken at their full nominal values. Stress range is checked not using a single lane fatigue loading as used in AS 5100, but the combined transient load effects from the SLS combinations.
Table 1: Key SLS design requirements

<table>
<thead>
<tr>
<th>Design item</th>
<th>PS</th>
<th>RC</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS compression comb 1A</td>
<td>$\leq 0.52f'c$</td>
<td>N/A</td>
</tr>
<tr>
<td>SLS compression other combs (no DTP)</td>
<td>$\leq 0.65f'c$</td>
<td>N/A</td>
</tr>
<tr>
<td>SLS compression other combs (with DTP)</td>
<td>$\leq 0.75f'c$</td>
<td>N/A</td>
</tr>
<tr>
<td>SLS crack width comb 1A</td>
<td>$\leq 0.20\text{mm}$</td>
<td>$\leq 0.35\text{mm}$</td>
</tr>
<tr>
<td>SLS crack width other combs</td>
<td>$\leq 0.30\text{mm}$</td>
<td>$\leq 0.40\text{mm}$</td>
</tr>
<tr>
<td>SLS strand stress limit in service after loss</td>
<td>$\leq 0.8f_{py}$</td>
<td>N/A</td>
</tr>
<tr>
<td>SLS non-PS steel stress limit in service</td>
<td>$\leq 0.8f_{y}$</td>
<td>$0.8f_{y}$</td>
</tr>
<tr>
<td>SLS stress range in strand and non-PS steel comb 1A</td>
<td>$\leq 150\text{MPa}$</td>
<td>$\leq 150\text{MPa}$</td>
</tr>
<tr>
<td>SLS stress range in strand and non-PS steel other</td>
<td>$\leq 200\text{MPa}$</td>
<td>Assume as PS</td>
</tr>
</tbody>
</table>

Both short term and long term in-service cases were considered in the design of the Super-Tee girders. The short term case was taken at the time when the bridge first opened to traffic. In this case, the girder had the highest in-service prestress in it and only a small portion of residual creep effect had occurred. The short term case was found to be critical for the design for hogging moment at the region near the pier. The long term case governed the design of sagging moment at both midspan and pier regions as all prestress losses and residual creep effects had occurred.

Design of sagging region

The Super-Tee girders were designed as a Class C (cracked) prestressed member for flexure sagging moment within the span. A sagging moment could develop along the whole span because of the presence of continuity restraint moment and other load effects discussed earlier. A tension force caused by global shortening was also present in the superstructure which needed to be considered in conjunction with bending. The critical location was generally at mid span where sagging moment was the largest. However, locations towards the end of girder where some of the strands were de-bonded were also found to be critical for spans near the end of the bridge. This was mainly because of the significant sagging moment generated by thermal, creep and shrinkage shortening near the pier as well as the reduction in prestress at these locations.

Load combination SLS 2A governed the design of most girders for flexure sagging moments at SLS. Steel stress range under transient load effects was found to be the governing criteria. As described above, traffic live load, global temperature and differential temperature transient effects were required to be considered together in the stress range check. Design for ULS sagging moment was governed by load combination ULS 1A at mid span and combination 1B near the pier for most of the girders. It was found that the ULS case was usually less critical than the SLS case mentioned above. Bottom non-prestressed reinforcement was provided together with the prestressing strands to satisfy the SLS and ULS design requirements.

Design of hogging region

The hogging region near the support was designed as a reinforced concrete section for flexure hogging moment for SLS and ULS requirements. Additional longitudinal deck reinforcement was provided in the hogging region. In addition, because the precast Super-Tee still had
prestress in this region, concrete compressive stress at the bottom of girder within the hogging region was checked as a prestressed section.

**Design for opening moment at pier**

Special consideration was given to the design for sagging moment at the integral joint between the Super-Tee and the pier cross head diaphragm and for the region of the girder near the support. It was found that this region could be in sagging even under permanent load effects. The section at the end of the girder had no prestress so it was designed as a reinforced concrete section for SLS and ULS requirements. Because the reinforcement could be in permanent tension, both sagging and hogging transient moments could generate tensile stress range in the bottom reinforcement. All strands that were not de-bonded were extended from the end of the Super-Tee and anchored into the pier cross head diaphragm to provide the required ULS sagging moment resistance. However a significant number of additional reinforcing bars were also required to control SLS stress range in the strands under transient load effects. The additional bars were placed at a position to optimise their effectiveness as well as making sure the maximum allowable SLS stress in the bars was not exceeded. The typical reinforcement projection detail is shown in Figure 5 & 6.

![Figure 5: Typical Super-Tee reinforcement projection detail elevation.](image)

![Figure 6: Typical Super-Tee reinforcement projection detail end section.](image)
INTEGRAL JOINT DETAILING

The requirement for moment continuity at each integral pier was provided by extending the fully bonded strands beyond the end of the Super-Tee girder and anchoring them with swaged plates at the opposite side of the crosshead. Although the seismic load case did not provide the governing moment demands, it was considered necessary to provide full anchorage for all positive moment reinforcement to cater for seismic overloads, in accordance with usual NZ design practice. Also, providing anchorage for the strands at the far side of the crosshead assisted with the internal joint mechanisms for the transfer of moment from superstructure to piers. In addition to the strand projections, 10 to 16 RB32 bars were also provided, these projected halfway across the crosshead with loose lapped RB32 bars provided for continuity. These were anchored with footplates due to the limited anchorage length available.

This arrangement is quite congested in practice so several measures were incorporated into the detailing to assist with constructability as follows:

- The width of the crossheads (2.6m) was dictated by the column size of 1.8m. In order to make placement of beams with opposing strands and reinforcement easier, the distance between the Super-Tee girder ends was maximised by using the minimum practical seating on the given crosshead width.
- The strands and RB32’s which projected from the girder ends were arranged in columns to maximise the gaps between them thus allowing easier placement of the girders over the crosshead stirrup reinforcement. The stirrups at beam seating locations were detailed with couplers to allow the beams to be placed on an uncluttered seating. This option was, however, not pursued beyond the first few crossheads as the constructors felt coupling the shear links after beam placement was more of an issue than landing the beams with shear links carefully set out in place.
- The RB32 projection bars were not lapped directly but were terminated at the midpoint of the crosshead with a loose RB32 (with footplates) provided for continuity.
- At the column location the vertical bars projecting from the column into the crosshead which would have clashed with the girder strand and reinforcement projection were displaced locally or terminated within the down-stand to provide a less congested seating.
- Swages were staggered longitudinally to allow the cluster to sit lower in the crosshead second stage pour.
- An option to provide continuity by means of bars projecting transversely from the beam end blocks and anchored with hooked reinforcement was examined but rejected due to the additional complexity caused in forming the beam end block.

Figure 7 shows a typical construction detail at pier crossheads. Clashing of reinforcement was minimised by careful setting out of the crosshead vertical starter bars and grouping the strands and RB32s extending from the girder.

Although this form of construction was relatively new to many of the construction team, with careful planning of the reinforcement placement and refinement of the detailing, the pier crosshead construction has progressed well with nearly three ramps completed to date.
CONSTRUCTION CHALLENGES

The most significant construction challenge was constraints on the working area available. The varying topography of the GNRI construction area meant that although the geometry of the bridges was fairly standardised for the majority of ramp spans, the methods and ease of construction varied depending on the site conditions. For example, construction of areas which were situated within the coastal marine area of environmental and archaeological importance was carried out on temporary staging. Construction of this staging itself required bored and driven 710mm diameter steel piles supporting 610UBs and concrete deck panels. Piles of temporary staging were installed with a launch frame and a 100 tonne crane mounted drill rig.

Constructing the bridge near or over existing carriageways was also a challenge. Piers and spans which were situated in close proximity or above the live SH16 motorway, its on and off ramps and or Great North Road were constructed during night shifts with traffic closure. At all times work planning was completed with the aim to minimise stakeholder disruption, whether it be the local residents of the Waterview area or the Auckland commuters utilising the live SH16 motorway and main arterial routes such as Great North Road. Figure 8 shows some of the piers located around the coastal marine area and existing carriageways.
Another challenge was the high density of reinforcing required within some structural concrete elements. Common areas which encountered this problem were the two stage crossheads. This density of steel required extra care in the detailing and fixing of the reinforcement to ensure steel was placed in the correct positions to miss strands protruding from the Super-Tee’s while still achieved its required height in the diaphragm. Landing of the Super-Tee’s was hindered by this density also, with the extra time required to ensure the strands were situated correctly amongst the vertical starters protruding into the diaphragm.

With the Great North Road Interchange linking into the SH16 motorway, it therefore interfaced with neighbouring Western Ring Route projects. This interface created programme constraints where areas of work were shared with the neighbouring projects. This required careful planning to ensure all deadlines set years prior were hit to allow handover of the sites.

CONSTRUCTION OF TYPICAL ELEMENTS

Piles and Pier Columns

The foundations of the GNRI ramps consisted of 76 bored reinforced concrete piles. The pier piles were all 2.1m diameter while the abutment piles were 1.2m and 1.5m diameter. The piles have been typically founded into Waitemata Group sandstone with a grooved rock socket. Twenty six piles were drilled through basalt layers with permanent steel casing coated with Sika Blackseal to de-bond the pile from basalt to avoid potential down drag. Due to the strength and thickness of basalt and the limited torque from available piling rigs, it was necessary to pre-drill four to five 900-1050 diameter cores within the bore of 2.1m diameter pile before shaping with a larger core barrel and removing basalt fragments with a grab. Of the 76 piles, 11 were within the coastal marine area of archaeological importance so they were constructed from the temporary staging described earlier.

Among the many other challenges, the generally accepted piling tolerances of ±75mm plan, ±25mm height were abandoned for the 2.1m diameter pier piles to obtain a plan position of ±20mm for installing a column plunge cage into the top of pile which functioned as column starters. This starter cage was plunged with a tolerance of less than 20mm into wet concrete by a jig attached to the top of the pile following completing the concrete pour. To allow sufficient time for the plunge operation, retarding the concrete near the top of pile was required.
Tight areas are not uncommon with piling but with the live SH16 motorway traffic on either side, at times each slew of the crane or piling rig had to be planned with attenuation arranged due to the counter weights slewing over the live traffic envelope. Temporary works piling and crane platforms were also required in many locations to install piles on motorway batters. For piles located around the coastal marine area and also in areas of prior land fill, most of these piles were permanently cased with an outer 2.4m diameter annulus casing creating a need to pile at different diameters, somewhat telescopically, in order to hold up the overlying soil and rock layers.

The 1.8m diameter columns up to 23m high were all constructed full height in a single stage. As described above the piles were constructed with a column starter cage which enabled full height, prefabricated column cages to be pitched and stood on the pile and secured to the plunge cage using U bolts. The column cages weighed up to 14 tonnes and it were required to be temporarily supported on the top of the pile before installation of column form work. 8 additional longitudinal bars in the column cage were added to act as temporary support to the column cage on the top of the pile concrete. Some of the columns have very dense steel of up to 52 No.DH40 vertical bars with DH25 spirals at 100 centres. Adjusting cage orientation in plan was critical to ensure the crosshead reinforcing could be passed through the top of the cage at the correct orientation. Column cages more than 13m high were propped laterally to ensure no overturning prior to formwork installation. The column formwork consisted of fabricated steel half round sections in 6m, 3m, 1.8m and 1.2m lengths. These were bolted together and sealed on the ground horizontally in sections up to 12m prior to being lifted over the column cage. Adjustable props at 90 degrees to each other and connected to concrete mass blocks at ground level were then bolted to and used to plumb the formwork. All columns were poured via 4” to 6” boom pumps which due to their heights required a longer than the standard 6m length of dropper hose to minimise concrete free fall. These dropper hoses longer than 6m were required to be supported by a crane at all times during the concrete pour. Standard concrete vibration methods were used via the use of pneumatic concrete vibrators to achieve concrete compaction. After 48 hours of the concrete pour, the formwork was then removed by unbolting and lifting off formwork sections in halves.

Pier Crosshead Down-stand

The crossheads were constructed in two stages, the down-stand first then the infill diaphragm after Super-Tee erection. The primary purpose of the down-stand was to support the Super-Tee girders and launching gantry during construction, prior to pouring the diaphragm. Due to
multiple pier locations over the coastal marine area and over live roads, a typical falsework system bearing on the ground below the crossheads was not possible. A friction clamp system was therefore adopted supporting construction off the permanent columns. The falsework set consisted of two frames which were stressed together around the column using 8 No. 36mm diameter Freyssibars. Due to each crosshead having a specific cross fall and longitudinal fall, Megashor jacks were used on top of the clamps to achieve the falls specific to each crosshead. Megashor rocker heads were then used in the cross-fall direction and wedged steel shims in the longitudinal fall direction to make up the angle difference between the clamps and the soffit. The soffit, consisting of two fabricated steel platforms was then lifted up and bolted to the Megashor to form a hand railed working platform and soffit form for construction of the crosshead.

Unless severely constrained by lack of space such as sites within the median of the motorway and Great North Road, access to the crosshead platform was gained via scaffold stair access. This was found to give the safest and most efficient form of access. In the site where space was restricted, a scissor lift was used to access the platforms.

Due to each crosshead having a different cross-fall or longitudinal fall to the next, the ply on the soffit form was adjusted to the required shape and stop ends installed to achieve the required length of the particular crosshead. This was completed once the friction clamp and soffit platforms were installed to allow for any correction of any out of tolerance falsework installation or column position. Figure 10 shows a typical crosshead falsework.

Figure 10: Typical crosshead falsework.

The reinforcing cage was tied in-situ onto the soffit form. Although prefabrication of the reinforcing cage was explored in the early planning stages, it was not adopted due to the significant reinforcing density in both the column and the crosshead which would have made lifting and installation of the pre-fabricated cage difficult. Once the reinforcing was tied in place, ply faced steel side forms incorporating walkways were installed each side of the crosshead and held in place using form ties running below and above the design surfaces of the concrete. The concrete was then poured using a boom pump in one continuous pour to the top of the down-stand level. At this point Sika Rugasol was applied to the top surface to form a construction joint. Prior to the removal of the formwork and falsework the gantry plinths used for supporting the launching gantry were formed up and poured as a secondary pour the day after the main crosshead down-stand pour. After the gantry plinths were poured the side formwork and falsework were stripped in reverse order to installation. A typical pier crosshead down-stand is shown in Figure 10.

Three sets of falsework were used in the construction of the 43 standard crossheads. Work on the three falsework sets was completed by three crews, falsework installation and removal, reinforcing fixing and formwork and concreting. This enabled three working faces to be open at all times and continuity of work for each crew. By the end of the standard crosshead
construction, an average duration for completion of an individual crosshead down-stand was 18 working days, reduced from 25 days at the start of construction.

![Figure 11: Typical crosshead down-stand.](image)

**Superstructure**

Construction of the superstructure was carried out using a twin truss self-launching girder erection gantry supplied by DEAL in Italy. With 1.7km of viaducts to erect, the choice of erection methodology required careful consideration. A detailed costing exercise was undertaken to compare the merits of crane erection and use of an overhead gantry. This exercise demonstrated that there was no real winner between the two options from a pure cost standpoint. As a result a decision was made to pursue the gantry option as this offered perceived safety, traffic disruption and repeatability benefits. The key downside of this choice was the resulting linearity of the construction programme which would make acceleration only possible by the employment of crane erection of some parts of the interchange, thus reducing the amortised cost of the gantry. The up-front cost of purchase and design lead time was also a downside of gantry use. Figure 12 next page is a photo of the erection gantry transporting a Super-Tee girder.

At the time of the decision to go for the gantry option (and before the tender and design process had run its course) the bridge design was quite well advanced to the point that loadings from the gantry were needed to finalise the design. In order to work round this timing issue the procurement of the gantry included a restriction on the eccentric loading that the gantry could apply to the substructure. A limit on the transverse loading was imposed such that this did not exceed the maximum live load eccentricity which could be imposed in the permanent case. On this basis the design of the substructure was able to proceed without the final gantry specification and loadings. This restriction resulted in an innovative solution from the chosen supplier (DEAL of Italy) whose gantry sequence included provision to ‘side-shift’ the edge girders using one or the other of secondary cranes under each truss to lift and replace the girder whilst limiting the eccentricity of the whole system.
Four concrete plinths were cast and stressed on each crosshead down-stand to support a steel run-way beam which the gantry sits on. To erect a new span, the gantry used its main crane between the twin trusses to pick up the central Super-Tee girder from delivery truck located on previous completed spans. It held the girder in the current position and launched itself one span forward. Then the girder was moved one span forward by two cranes on the gantry while the gantry remained stationary during girder movement. Gantry and girder moved forward alternately until the girder reached the span currently being constructed. The Super-Tee was lowered down on temporary plastic shims on the crosshead down-stand. The central girder was always installed first. Longitudinal restraint was installed between the central girder and crossheads at both ends acting as a prop to limit crosshead deflection before more girders were erected. Figure 13 shows gantry movement along the bridge under typical Super-Tee erection sequence.

Once the central girder was erected, the gantry repeated previous operations to transport other girders to the span being erected. The gantry moved transversely on the run-way beams to deliver other Super-Tees to the required transverse location, following a special transverse sequence. Figure 14 shows typical transverse sequence of a five girder span. The gantry used its main crane to put the two edge girders temporarily at location 2 and 4. It then used the
secondary crane located under each truss to side shift the edge girders to the edge location 1 and 5. This operation was a key innovation from the gantry designer to minimise the eccentric load of the whole gantry on the pier during erection of the edge girder. Finally the gantry erected internal Super-Tee in location 2 and 4 using its main crane. Figure 14 also illustrates the side shift and internal girder erection operation.

When Super-Tee girders temporarily sit on Ramp 4 table top down-stand, the large longitudinal eccentricity about the pier generates excess rotation of the pier before the table top was fully completed. Two temporary steel props were installed from the ground at the large pier 10 table top to limit rotation of this pier. Span 8 and 9 girders were placed alternately on either side of the small table top pier 9 to limit pier rotation. Similarly, span 10 and 11 girders were placed alternately on either side of the other small table top pier 11. Figure 15 shows the gantry erecting girder in the Ramp 4 table top spans and the temporary props installed at Ramp 4 pier 10.

Deck pour was carried out span by span together with the pier cross head infill stitch pour. To minimise eccentric loading on a pier, deck slab of a span was not allowed to be poured until girders in both spans adjacent to this pier were all erected. In this case the self-weight of the girders were roughly balanced on both sides of the pier. Deck pour started at 3.2m from the pier centreline and run backwards to the pier stitch to avoid locking in rotation at the end of the girder. Figure 16 next page illustrates the typical deck pour sequence.
CONSTRUCTION OF SPECIAL ELEMENTS

Post-tensioned Crosshead

Due to the vertical alignment constraints on Ramp 3, the clearance to the underside of the crosshead situated within the median of SH16 meant that it was required to be shallower than the standard crosshead. As a result, temporary post-tensioning was put in to the shallower down-stand to provide the required strength during construction of the superstructure. Also, due to the crosshead overhanging the live motorway lanes, it was not possible to create a working platform to construct the crosshead in-situ as per all the other ones. The decision was made to precast the crosshead down-stand in a site adjacent to the motorway.

The crosshead was cast with drossbach ducts in the positions of where the column verticals were required to pass through the crosshead down-stand. The column was then cast within the median of the motorway with the location of the verticals matching the location of the drossbach ducts within precast crosshead. Under full motorway closure the 90T crosshead was transported out onto the motorway where it was lifted over the column starters and landed on four jacks which were used to adjust the pre-cast crosshead to its final orientation. The drossbach ducts and the gap between the column and crosshead was then grouted fixing the crosshead within 5mm of design vertically and 25mm in plan. Finally, the post-tension stress bars were stressed.

Ramp 4 Table Top Piers

Three table-top pier crossheads (13m & 8m in length rather than the standard 2.6m) were introduced for the two 44m long spans in Ramp 4. All three were constructed in two stages using the same method. Due to the space constraints at all three table top sites, and because a significant portion of the table top footprint was situated over the live carriage way, the falsework needed to be supported centrally around the permanent works column. The falsework was designed to support the wet concrete of the large table top at pier 10. The same falsework was used for the other two small table tops with minor modification. To install the falsework, firstly the base clamp was bolted around the base of the column beared on top of the permanent pile. Four 710mm diameter steel casings of four different lengths, to apply the longitudinal and cross fall to the soffit were then stood onto the base clamp and secured to the top of column by a ‘top clamp’. To enable stripping of the soffit platform, sand jacks were then welded to the top of the casings. The soffit, fabricated in two halves, was bolted together at ground level prior to being lifted onto the top of the sand jacks and steel casings. Installation of the soffit was undertaken during full closures of the motorway and Great North Road. This was to enable the mobile crane required to lift the soffit into place, to set up on the road. Due to its plan area of 15.5m by 16.0m, with walkways attached, a 450T crane with Mega-wing attached was required. This operation was heavily time critical due to the restricted road closure times on SH16 and Great North Road and the time required to set up and counterweight the crane.
Once the soffit was installed the reinforcing for the down-stand was tied in-situ, with reinforcement bundles being lifted onto the soffit by a service crane. The concrete pour for all three table tops was completed by two concrete boom pumps pouring from each end. These were required to enable the load to be applied onto the falsework evenly to minimise eccentric loading in the falsework and the permanent works column. Pouring with two pumps also increased the speed which enabled the pour to be completed within the timeframes allowed by the road closures. After curing of the concrete and creation of a construction joint, the falsework was removed from the column. Figure 17 shows the Ramp 4 pier 10 large table top during the down-stand concrete pour.

Figure 17: Ramp 4 pier 10 table top down-stand during concrete pour.

As mentioned in the superstructure construction section, two temporary props to the down-stand work in both compression and tension were installed from the ground before superstructure construction. The foundation of each prop was made from 1050mm diameter concrete pile while the prop itself was a 710mm diameter steel tube. The props were connected to the top of the pile via a hinged connection bolted to the underside of the crosshead. After all girders were erected on the table top spans following the sequence described in the superstructure construction session earlier, the second stage infill crosshead pours were carried out pier by pier. Props were then removed after the concrete gained sufficient strength.

**Ramp 4 reinforced concrete portal beam**

A 17.5m long portal beam spanning between two 1.3m square columns was required to support ramp 4 superstructure at the point it crossed over the top of Ramp 3. The two 16m high columns were constructed square by utilising the same steel formwork used to construct the 1.8m diameter circular columns. This was achieved by constructing square timber formwork within the circular steel forms. Using these forms enabled the columns to be poured full height and thus eliminating two construction joints which would have been required if a proprietary formwork system was used. This was not only beneficial in terms of aesthetics and structural integrity of the columns, but also beneficial to cost of construction. This was due to reducing the programme and thus labour hours required to build the columns in one lift as opposed to having to jump the formwork to complete multiple lifts.

The columns were constructed in a similar method to the standard columns, although due to the high density of reinforcing within the column reinforcing cages, a self-compacting concrete mix was used. This gave reassurance that all areas of the column would be well compacted and not reliant on being accessed by a vibrator. The disadvantage to the use of this mix meant
that the vertical pour rate was required to be no more than 3m an hour and the forms had to be made water tight prior to pouring.

Falsework made of two 1.88m deep steel I beams and a number of transom beams was required to carry the self-weight of Super-Tee girders and deck in adjacent spans, and the wet concrete of the portal beam. The temporary beam also supported the erection gantry while it was travelling over the portal. This twin steel I beam falsework system was supported on steel corbels stressed onto the two permanent portal columns using 12No 36mm stress bar. Sand jacks were placed on the steel corbel to set the steel beam to the right level, longitudinal fall and cross fall. The steelwork was delivered to site in pieces and required bolting together on site. A plywood working platform was installed over the transom beams to enable safe access within the steelwork. Once ready for installation, the falsework was lifted into position onto the sand jacks via a 450 tonne mobile crane. Once the falsework was secured in position and access gained, the plywood working platform was surveyed and then decked out to construct a soffit which incorporated the 40mm pre-camber to be applied to the portal beam.

![Figure 18: Ramp 4 portal falsework.](image)

Four steel gantry plinths were installed on the falsework to support the erection gantry. Super-Tee girders were placed onto the falsework I beams using the launching gantry. The two central girders on each span were fitted prior to installation, with brackets stressed to the underside of the beam. These brackets fixed the Super-Tee to the top flanges of the falsework main beams which then enabled these Super-Tee’s to act as a longitudinal restraint to the falsework and the portal pier columns. The gantry erected spans adjacent to the portal beam following the typical sequence as discussed earlier. After girder erection, the portal beam concrete was poured following the adjacent span deck concrete pour. Figure 19 shows the gantry travelling over the portal beam.
Ramp 2 Post-tensioned Portal beam

Ramp 2 superstructure is supported on bearings on a post-tensioned portal beam spanning 24m across SH16 westbound carriageway. The portal beam spans between two standard 1.8m diameter columns on free float pot bearings and restrained by a solid steel shear key at each end. There is a concrete up-stand on the top of each column as a bearing shelf for the pot bearings. Figure 20 shows the general arrangement of the portal beam. At the time of writing this paper, the columns and up-stands have been constructed while the portal beam and Ramp 2 superstructure have not been constructed yet.

Due to the location of the portal over the live motorway, pre-casting was always the most optimal construction method. However, pre-casting the entire 25m long portal beam was not feasible because it would be too heavy to lift. Minimising the weight in the design phase was critical to make lifting and installing the beam feasible and cost effective. A two staged construction design solution was developed. The bottom half of the portal beam will be pre-cast on the ground with some of the post – tension tendons stressed. This pre-cast beam weights about 150 tonne which is manageable. It was established that the most cost effective method for installing the beam was via a tandem lift utilising a 350 tonne and a 450 tonne mobile cranes as opposed to a single crane lift. Due to limited positioning of these cranes.
within the motorway median site and within the reduced lanes of the motorway carriageway, the position of the precast site was critical and was selected based on the crane lift plan. This plan set the crane radius for each crane which minimises the requirement for luffing of the cranes and allows one crane to operate under free slew at any one time.

After pre-casting and the first stage of post tensioning, the bearings are to be fixed to the underside of the beam and it will then be lifted into position under motorway closure. Once in position the solid steel shear keys are slotted through the portal beam into the column up-stand at each end and grouted into position. The top half of the portal beam is then able to be poured in-situ on the pre-cast section to form the full cross section of beam. After the remaining tendons are stressed, the portal beam is ready for the arrival of gantry and the construction of Ramp 2 superstructure.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the Waterview Connection experience, the following conclusions can be drawn.

Creep Effects

- An important aspect of the analysis and design of integral construction Super-Tee bridges is the development of positive (sagging) secondary moments throughout the span due to residual creep. These moments offset the advantage of continuity for live load effects, and can require significant amounts of reinforcement to be extended from the girder ends and developed into the support.
- Variability in the methods used for assessment of residual creep moments could lead to significant variation in design outcomes. The fact that the NZTABM 3rd edition now refers to AS 5100.5 Appendix E for this aspect of design should reduce such variability.
- The assessment of creep factors, especially those applying to the period after the establishment of continuity, is more important in integral construction bridges than it is in the equivalent simply supported bridges, and it is important that robust methods for the assessment of such factors are available.

Design Criteria

- In the design of fully integral bridges, load combinations that include the effects of traffic loads (both vertical and horizontal) together with temperature effects (global and differential) assume more importance than they do in the design of simply supported bridges. The combined effect of the NZTABM SLS load combinations and the NZS 3101 prestressed concrete design criteria, together with the lack of a true fatigue design loading, seems unduly conservative in this regard. This currently works against the adoption of integral construction.

Seismic Resistance

- Integrally constructed bridges do offer seismic resistance advantages. However, for long bridges such as the GNRI Ramp bridges, the increased shortening demands arising from integral construction are significant – to the extent that the resulting additional reinforcement may result in such bridges costing more than equivalent simply supported bridges. Which effect is more important will depend on the level of seismic hazard present.
- Also, if careful planning and detailing is not used, the necessary integrity of girder-pier joint might not be achieved in practice, which could be detrimental to long-term performance.

Aesthetic and Maintenance Considerations

- Elimination of bearings and associated shelves and shear keys is certainly of benefit aesthetically as well as from a maintenance viewpoint
Complexity and Effects on Cost

- The analysis and design of integral construction bridge using Super-Tee girders is considerably more complex than that for bridges utilising simply supported girders; partly because of residual creep effects, but also because the design of any fully continuous and integral bridge is more complex. Because it is more complex it takes longer and costs more.
- Compared with simply supported spans on bearings, it appears that the increased complexity of the crosshead and integral pier detailing may potentially take longer to build and hence cost more, offsetting the savings on bearings, plinths and shear keys.

Erection Methods

- Although the initial purchase cost of a gantry is significant, for long Super-Tee bridges like the GNRI Ramps with limited construction space available on the ground, using a self-launching gantry instead of cranes to erect superstructure offers overall construction cost, program and safety benefit.
- However, gantry erection requires significantly more design input which should also be taken into account in the initial optioning exercise. This is particularly true for bridges similar to GNRI Ramps with single column piers and cantilever crossheads, where construction load cases could end up governing the permanent works design.
- It should also be noted that gantry erection itself restricts the number of work fronts to one which results a relatively linear construction program. If program acceleration is required during construction, additional cranes need to be introduced to open up more work faces. This will reduce the cost benefit of gantry erection and might make the overall construction cost higher than the pure crane erection option.

Recommendations

- It is recommended that NZ designers, constructors and bridge owners take note of the conclusions above.
- With the increasing prevalence of integral Super-Tee construction form, it is recommended that those aspects of the NZTA Bridge Manual and NZS 3101 that have particular significance for the design of such bridges are reviewed and revised as necessary to ensure that future designs are not penalised by unduly onerous requirements, but are also sufficiently robust. These aspects include:
  - Adequate allowance for residual creep effects
  - Load combinations, especially combining traffic and temperature demands
  - A suitable fatigue design loading
  - Prestressed concrete SLS design criteria
  - Creep factors suitable for NZ concrete mixes.
- It may be desirable that some guidance is given with regard to detailing of Super-Tee girders used in this form of construction, just as it is available for simply supported girder spans. An examination of the current UK and US practice for design of such bridges may be useful.
- We suggest that it may be desirable to carry out some physical testing in order to determine the extent to which theory and practice coincide for this construction form, both with regard to the development of residual creep demands and the performance of details that are being used in integral construction.
- For long bridges with limited construction space available, gantry erection offers potential program, cost and safety benefits although it does require significant initial cost and more design effort. Gantry erection also results a linear construction program. It is recommended all factors should be taken into account in the optioning of construction methodology.
- Finally, because there are both advantages and disadvantages to integral construction, the answer regarding whether integral construction is more expensive than simply supported (not just from the Design and Construct contractor’s viewpoint) needs to be based on a true “apples for apples” comparison for each project. This should start with designs with
similar constraints that achieve the same standards, and properly accounting for all design, construction and whole-of-life costs.

REFERENCE
## AUTHOR BIOGRAPHIES

<table>
<thead>
<tr>
<th>Photograph</th>
<th>Short Biography</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Terry Cheng" /></td>
<td>Terry Cheng is a structural engineer with Beca Ltd, based in Auckland. He joined Beca's Civil Structural group after graduation and now has over seven years experience in bridge design. Terry is the senior bridge design engineer on the Waterview Connection project responsible for the Great North Road Interchange bridges.</td>
</tr>
<tr>
<td><img src="image" alt="William Dobson" /></td>
<td>William Dobson is a Section Engineer with Fletcher Construction. He has over 5 years of experience working in the construction industry, working on motorway projects involving earthworks, drainage, pavements and bridge structures. On the Waterview Connection project, William is a section engineer for the Great North Road Interchange bridges.</td>
</tr>
<tr>
<td><img src="image" alt="Lawrence Blackmore" /></td>
<td>Lawrence Blackmore is an Associate Structural Engineer with Beca Ltd, based in Auckland. He has 26 years’ experience as a bridge engineer and has been with Beca for the last 3 years having transferred from the UK. Lawrence is the Discipline Lead for bridges on the Waterview Connection project.</td>
</tr>
<tr>
<td><img src="image" alt="Andrew Dickson" /></td>
<td>Andrew Dickson is a Senior Technical Director with Beca Ltd, based in Auckland. He has 29 years of structural design experience, the last 18 of which have been spent as a bridge engineer with Beca’s Civil Structural group. Andrew is the Surface Work Structures Design Verifier on the Waterview Connection project, responsible for the overview and verification of the surface structures design.</td>
</tr>
</tbody>
</table>