The Waterview Connection – Great North Road Interchange Bridges

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Abstract: The NZ Transport Agency's $1.4 billion Waterview Connection in Auckland is the largest roading project ever undertaken in New Zealand. One of the dominant features of the project is the Great North Road motorway to motorway (SH20 to SH16) interchange (GNRI). The interchange ramps incorporate four separate significantly curved viaducts varying in height (up to 20m above SH16) and ranging in length from 225m to 500m. The substructures typically consist of reinforced concrete hammerhead type cross heads supported on single pier columns and single piles. However, a number of variations were necessitated by restrictions on pier positions, including some portal type piers and some 'tabletop' piers. 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. Delayed stitch pours at some piers were utilised to reduce creep and shrinkage demands on end piers to acceptable levels. The superstructures were constructed using a self-launching gantry, over one of the busiest motorways in Auckland.

With the construction of the bridges successfully completed, the first part of this paper offers the designer's perspective on the challenges posed by the design and construction of long integral Super-Tee bridges utilising gantry-erected girders, outlining key analysis and design considerations. The second part 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.

Keywords: Waterview Connection, Super T, Integral, Launching Gantry

Overview of Waterview Connection & 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 ramp bridges 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 (GNRI) ramps comprise four separate viaducts connecting the SH20 tunnels with SH16. The lengths of the bridges range from 225m to 500m, with horizontal radii as tight as 240m. Figure 1 shows more detailed information on the bridge layout.

Development of Structural Form

As shown in Figure 1, there were a number of site constraints affecting the choice of substructures. The form and location of foundations was limited by the existing SH16 alignment, existing Great North Road and the adjacent coastal marine area (CMA). Resource Consents placed a limit on the number of piles able to be installed within the CMA at the southern end. In order to minimise the number of piles, a single pile - single column pier type foundation was the prescribed solution in the CMA and near existing carriageways, so it was adopted throughout for aesthetic and construction consistency.

Figure 1. Plan of Great North Road Interchange Ramps.

All the pier piles were 2.1m diameter reinforced concrete bored piles founded in East Coast Bays Formation (ECBF) rock. The size of pile was governed by both vertical end bearing capacity and stiffness requirements. Pad type foundation was also considered at the preliminary stage where thick basalt layer was present. However, it was decided the pile solution was of lower cost and risks. All pier columns were 1.8m diameter circular and supported directly on the piles. The columns were up to 23m high. A hammer head type pier crosshead became logical following the choice of a single pier column.

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 longer span deeper structural forms such as ladder beam decks and post-tensioned concrete 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 pier construction through the preliminary design process. It was found that the continuity restraint effect of creep and differential temperature offset the benefit of live load continuity. Also, the NZTA Bridge Manual (2nd edition at the time) load combinations which include the effects of traffic loads together with temperature effects seem quite conservative as compared with some other international bridge codes, which works against the adoption of integral construction. As a result, the integral option did not offer significant improvement in span length and reduction in girder size. Although integral construction eliminated the cost and time of setting out bearings and shear keys, the construction of integral joints was expected to add a level of complexity to the construction. 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 comprised spans of approximately 33m with 5 or 6 precast, prestressed Super-Tee beams with cast in-situ concrete slab. The precast Super-Tee beams were 1525mm deep and vary in width across the top flanges to suit the geometry of each location. The superstructures were cast integrally with reinforced concrete pier cross heads supported on the single reinforced concrete columns and single bored piles. At the abutments, Super-Tee girders were supported on pot bearings sitting on the abutment beam to allow movement along the longitudinal axis. Shear keys were provided at the abutment as transverse restraint to the superstructure. The abutments were supported on groups of 1.2m diameter bored piles with MSE walls or batter slopes forming the approaches. Typical structural form of the bridges is shown in Figure 2.

After discussion with the pre-casters, all girders within the same span were reinforced and prestressed the same way to provide consistency in fabrication. Since the edge girders tended to attract more live load and had to carry the barrier dead load, the flanges of edge girders were made narrower than the internal girders and central girders. For a typical 5 girder span, the edge girders were typically about 2.5m wide while the internal girders and central girders were 2.65m and 3.0m wide. This approach resulted a more even distribution of demand between girders and optimised the structural efficiency of each girder. The outer flange width of edge girders were varied along the length of girder to accommodate bridge horizontal curvature in plan. Where the flanges on each side of an edge girder were of unequal width, flange slots at 2m intervals were provided to make the effective flange width on both sides equal. As a result, no transverse bending stress was developed during stressing.

Figure 2. Ramp 3 (completed) & Ramp 4 (under construction).

Integral pier construction led to the need to temporarily support the Super-Tee girders before the integral connection between the pier and superstructure was made. Due to the height of the superstructure 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 acted as a temporary support to the girders and launching gantry. The girders were initially supported on the down-stand with a 320mm wide seating, then the second stage in-fill pour stitched the superstructure together with the pier to complete the integral connection. Temporary girder longitudinal, transverse and torsional restraints were installed as required before integral connection was made. Figure 2 above shows girders temporarily supported on the crosshead down-stand.

Design Challenges

Complex Structural Analysis

The Great North Road Interchange ramps are long, curved prestressed concrete bridges with integral piers. Compared with simply supported bridges or semi-integral bridges where girders do not have moment continuity, 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 the 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. Integral staged construction using precast prestressed components such as Super-Tee girders with a cast in-situ deck introduces a further level of complexity (refer paper Dickson, Blackmore & Cheng, 2014 for more detail). Modelling of the interchange ramps was carried out using software package CSi Bridge. Winkler springs at 1m intervals were used to represent the horizontal interaction between the structure and the founding soil.

Design for Erection Gantry and Construction Load Cases

Construction of the superstructure was carried out using a twin truss self-launching gantry supplied by DEAL in Italy. Because of the tight piling program, 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 foundation design. The bridge designer was heavily involved in the gantry procurement process and interacted with various gantry suppliers to work a way through this timing issue.

Gantry erection is commonly seen on twin column portal type pier bridges, constructed with relative small gradient and radii. Due to potential large eccentric load on the substructure, using gantry erection on bridges with single column and long hammer head crosshead was a challenge. It was decided to estimate the demand due to in-service load cases and 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 eccentric live load which could be imposed in the permanent case. On this basis the design of the piles and columns was able to proceed without the final gantry specification and loadings. The Alliance received an innovative twin truss solution from the chosen supplier DEAL of Italy. DEAL's gantry sequence included provision to 'side-shift' the edge girders using secondary cranes under each truss to lift and replace the girder. This operation limited the eccentricity of the whole system. Figure 2 shows a photo of the twin truss gantry.

Significant design effort was put into the 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 completed spans 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 were checked. Wind and seismic load cases could also be critical for the incomplete structures supporting the gantry.

The above load effects needed to be considered by the bridge and temporary work designer to ensure the permanent works and the required temporary works could cope with the demands. In order to achieve consistency between bridge, temporary work and gantry design, intensive interaction between the bridge designer, temporary work designer and gantry designer was required during the gantry design process. This cooperative design approach minimised design and construction risks.

Shortening Demands

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 (governed by the tunnels) provided an opportunity to take steps to minimise shortening demands. Selected Ramp 1, Ramp 3 and Ramp 4 piers (two each ramp) were left un-stitched with the superstructure over a period of time. 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. It was important to carry out the final stitch pour when the bridge was as cool as possible so night time pours were required. Also, plans needed to be put in place to minimise differential temperature effect during the day after the pour. The project program dictated that Ramp 2 being the last ramp to be constructed could not be left to shorten in short segments. To address this, a finger type expansion joint has been introduced around mid-length of the bridge to keep shortening demands down to a manageable level.

Stiffness of the end piers also had significant impact on shortening demands. It was preferred to keep the end piers same as other piers due to aesthetic reason and construction consistency. Therefore, some of the piles towards the ends of the structures were formed with a permanent annulus around the upper portion of the pile to increase their flexibility and reduce shortening demands on the pile, column, crosshead and integral pier joint.

Extra Long Spans and Table Top Piers

Where Ramp 4 passes over existing SH16 and Great North Road, the location of Ramp 4 piers 9, 10 and 11 was highly constrained by the presence of existing carriageway as well as the bridge horizontal alignment. As a result, the only possible option was to place pier 10 and 11 right next to the existing carriageway, and place pier 9 in the narrow existing SH16 median. This pier arrangement resulted in two 44m extra-long spans, well beyond the limit of 1525mm Super-Tee girders (1800 Super-Tee's were not readily available in New Zealand 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. Similar to the typical spans, the girders were initially simply supported on the edge of tabletop down-stands, and spanned about 33.5m. The table tops allow the girders to span a distance much less than 44m while they carry the dead load as simply supported girders. After the span is made integral, the relatively stiff tabletop attract more live load hogging moment and the mid span live load moment can be kept at an manageable level for 1525 Super Tees.

A special erection sequence was developed for the spans adjacent the table tops. When the Super-Tee girders temporarily sit on the Ramp 4 table top down-stand, the large longitudinal eccentricity about the pier can generates excessive 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, and restrain the pier under seismic conditions. 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 3 shows the gantry erecting girder in the Ramp 4 table top spans and the temporary props installed at Ramp 4 pier 10.

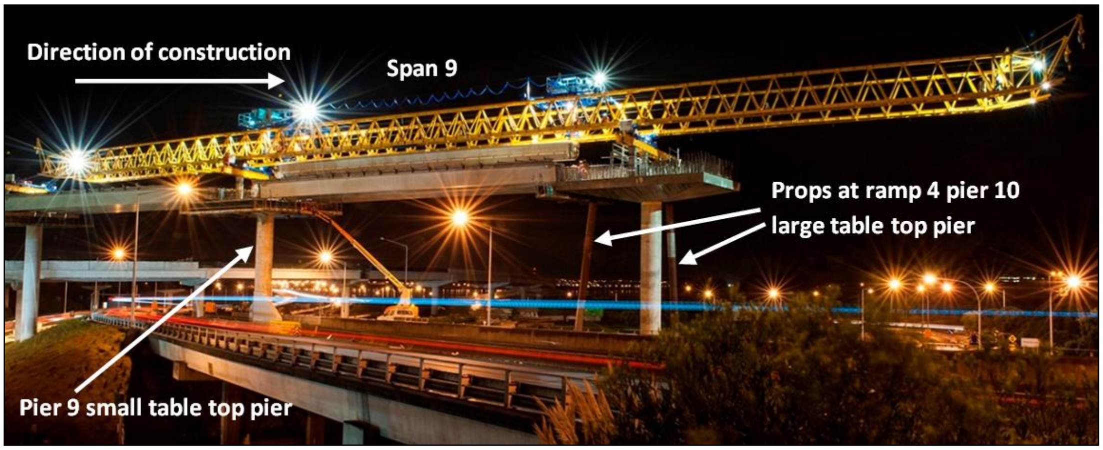

Figure 3. Ramp 4 table top spans & props installed at Ramp 4 pier 10

Portal Pier – Ramp 2

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.

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 is restrained by a solid steel shear key at each end. It was logical to place the required ramp 2 intermediate expansion joint at this location. There was not enough clearance above the live carriage way to install false work to cast the portal in-situ. Therefore, pre-casting was the optimum 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 was pre-cast on the ground with some of the post – tension tendons stressed. The weight of the precast section was kept to a manageable level by tapering the cross section width towards the end, and by making the section a U shape. After lifting the beam onto the columns, the top half of the portal beam was then poured in-situ on the pre-cast section to form the full cross section. After the remaining tendons were stressed, the portal beam was ready for the arrival of the gantry and the construction of Ramp 2 superstructure. Figure 4 below shows more detail of the Ramp 2 portal beam.

Portal Pier – Ramp 4

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. Again clearance to the new motorway below was a key constraint. In this case it was feasible to install false work on the underside of the beam so a reinforced concrete portal beam cast integrally and flush with the superstructure was the optimum solution. 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. Once the portal beam was cast integrally with the superstructure, the falsework was removed and the dead load was re-distributed into the complete portal. Figure 2 earlier shows the gantry travelling over the Ramp 4 portal beam.

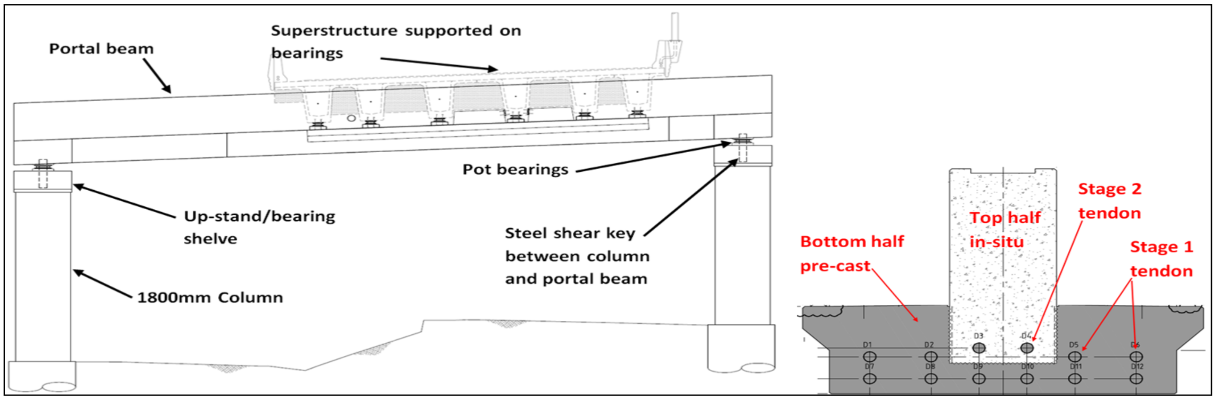

Figure 4. Ramp 2 post tensioned portal beam

Construction Challenges

Superstructure Construction Methodology

As noted previously, the construction of GNRI ramps was constrained by the topography and site conditions. With major arterial routes splitting the site into eight satellite areas, two adjacent NZTA Projects, a coastal marine and archaeological area located underneath three of the ramps, selecting the appropriate construction methodologies was critical for successful delivery. This section of the paper describes the construction challenges, successes and learnings gained in delivering the four bridges within two weeks of the original three and half year baseline construction program.

With 1.7km of viaducts to erect, the choice of erection methodology required careful consideration. A detailed optioning exercise was undertaken at preliminary design stage to compare the merits of crane erection, use of an in-house overhead beam shifter (moved by light crane) and a self-launching gantry (SLG). Some of the key aspects are listed in table 1 on the next page. This exercise demonstrated that although there was no real winner between the options from a pure superstructure construction cost stand-point, the cost of site set up for beam shifter and mobile crane option was a significant risk. As a result a decision was made to pursue the gantry option as this offered perceived safety, site access and traffic disruption and repeatability benefits. The post construction review further confirmed the cost of site set up and staging for crane would be much greater than the SLG option.

The biggest advantage of this method was having the beams delivered via one access through the main site and on to the southern abutment of the bridge being constructed. Both other options required beam delivery trucks (up to 50m long) to access the area immediately adjacent to the span being constructed. This would have caused considerable disruption and additional cost to build access routes capable of taking these vehicles. Another benefit of the SLG was it minimised road closure while working over motorways. With the proximity of the adjacent St Lukes and Causeway Projects, full motorway closures were at a premium. Using the SLG meant that construction traffic underneath was only stopped for short periods, as opposed to the other two options, where mobile cranes and beam delivery trucks would be blocking all lanes for much longer period. This was a small but crucial part of the successful relationships with the adjacent projects.

The key downside of this choice was the resulting linearity of the construction program which would make acceleration only possible by the employment of crane erection for some parts of the interchange, thus reducing the amortised cost of the gantry. However, the various constraints on the beam age at placement, plus the production capacity of the precast yard, meant that the driving factor for multiple workfronts was beam production – not gantry operation.

The SLG was designed and fabricated in Italy, by DEAL. Responsibility for the design certification of the system remained with the Supplier, with the GNRI Design team providing a producer statement that the structure could accommodate the loads imparted. The subsequent commissioning procedure – again supervised by DEAL – was witnessed by the same company who carried out the design certification. Maintaining DEAL's ownership of the design certification process removed a significant amount of risk from the Alliance, with the only notable stumbling blocks being contradictions between the Euro (EN) and New Zealand Standards, and the definition of a Gantry crane in the Approved Code of Practices. A "Gantry Crane" is normally taken as the type that operates on rails in a manufacturing environment, which places prohibitive tolerances on the levels. The SLG is subject to maintenance inspections in line with a normal crane, and a full assembly inspection after completion of each ramp.

Table 1. Erection method comparison (preliminary optioning exercise).

Item	**Self-Launching Gantry (SLG)**	**Crane Erected Beam Shifter**	**Mobile Cranes**
Upfront Cost.	$1.8m	$734,000	0
Flexibility	Linear span by span	Moderate flexibility between spans	Fully Flexible
Operating cost	$41k/Span	$45k/span	$70k/span
Beam Delivery Access	1 access per bridge	1 access per span	1 access per span
Site Setup Cost	Nothing	Minor	Significant.
Estimated Resale	200k	80k	0

The gantry was operated by a team of 9 people, supported by a dedicated site engineer and electrician on call 24/7. The maximum operating speed was 12m/minute, although this was only without load on the hook. Generally when carrying the beams, the gantry moved at between 3 and 6m per minute. Beam placement was generally 1 beam per night, with an additional shift for setup, and one shift for side shifting. The majority of the 277 beams were placed on nightshifts. This was mainly due to road closures and to minimize impact on dayshift operations. Furthermore, in most case the beams could only be delivered between 10pm and 5am due to oversized vehicle permit restrictions.

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.

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 5 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 5 also illustrates the side shift and internal girder erection operation.

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 side 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 6 illustrates the typical deck pour sequence.

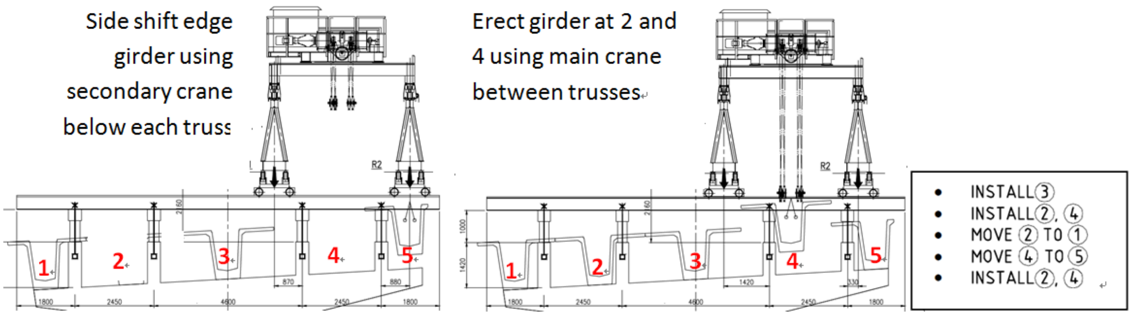

Figure 5. Typical transverse erection sequence for 5 girder span.

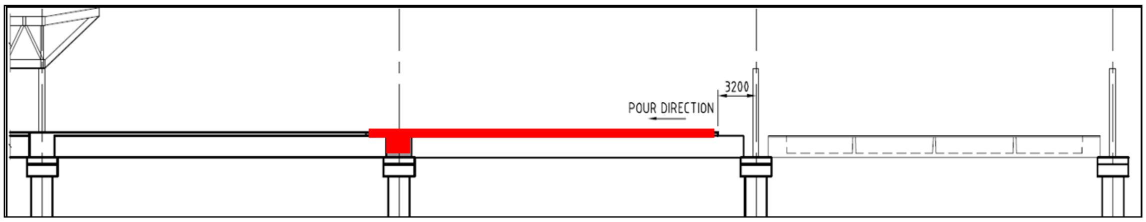

Figure 6. Typical deck pour sequence.

Integral Pier Construction

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. In addition to the strand projections, 10 to 16 RB32 bars were also provided. The column and vertical starter bars were also present. This arrangement is quite congested in practice so careful detailing and construction planning was required.

The strands and RB32's which projected from the girder ends were arranged in vertical columns to maximise the gaps between them thus allowing easier placement of the girders over the crosshead stirrup reinforcement. 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. Swages were staggered longitudinally to allow the cluster to sit lower in the crosshead second stage pour. Column and crosshead vertical starter bars at Super T girder location were carefully set out so they missed the reinforcement projecting from the end of the girder. Figure 7 shows a typical construction detail at pier crossheads.

Figure 7. Typical detail at pier cross head (partially completed).

Construction of Special Elements

While the construction of typical elements such as piles, columns, and abutments could be considered business as usual in infrastructure construction, construction of special elements like the portal beams, and table tops did create some challenges. Some details regarding construction of special elements were discussed in earlier sections. A couple of lessons are discussed below.

These elements were relatively heavily reinforced, particularly near the anchorage zone of the Ramp 2 post tensioned portal beam. There were a few congestion issues encountered during the construction, which caused construction delays and required re-detailing and revisiting of the design to optimize constructability. The drawings produced by the designer were in a conventional 2D form. Also, fabrication of the reinforcement was performed based on a series of 2D sections. In hindsight, this approach did not provide enough opportunity for the construction team to visualise potential construction issues. An option on future projects would be to carry out 3D modelling for all complex reinforcing details in design stage which can be used by the steel fabricator, to avoid rework and construction delay.

Lifting can be a challenge for heavy precast elements such as the Ramp 2 portal beam. The design team was working to a lifting limit of 150t which was based on the maximum allowable crane capacity for the particular available site set up. However, the actual weight turned out to be 175t due to under estimating the density of concrete and post tension work, pushing it against the limit of the cranes. The lift had to be very carefully engineered by the crane engineer and performed by our most experienced crane operators. Using a unit weight more representative of the heavy reinforcement content would result in a more accurate weight estimate in the design stage.

Key Outcomes

Now that the construction of the ramps have been completed, the following outcomes can be observed. Firstly, adopting integral construction not only offers maintenance benefits to NZTA, but also offers aesthetic benefits, particularly for bridge structures adjacent to the heart of a city. Through feedback received from the community, the aesthetic benefit is already highly valued by the general public. Figure 8 shows the complete bridges.

Secondly, early interaction between the constructor, permanent works, temporary works and gantry designers was the key to the successful design and implementation of a gantry erection method. The cooperative approach gave opportunities to refine gantry design and construction sequence so the efficiency of the permanent works could be maximised. The major structural elements were either governed by, or close to be governed by in service load cases rather than temporary construction load cases. The required temporary work for gantry erection was also kept to a minimum. The four bridges were delivered within two weeks of the original three and half year baseline construction program.

Thirdly, the costing exercise on superstructure construction options carried out at the preliminary stage appeared to be valid. The conclusion that self-launching gantry option was of lower cost and risk was backed up in the post construction review. The post construction review further confirmed the cost of site set up and staging for cranes would push the cost of the other two option much greater than the SLG option.

Figure 8. Typical detail at pier cross head (partially completed).

Finally, although the integral pier joint detail was complicated, with careful planning of the reinforcement placement and refinement of the detailing, the integral pier construction progressed well and did not cause any major difficulty.

Conclusions & Lessons Learned

- Super T is more cost effective than multi steel girders in NZ when used in long bridges with constraints on structural depth.
- Integral bridge offers and aesthetic benefit which could be of high value to the community.
- Integral bridge did not offer significant improvement in span length and reduction in girder size because the continuity restraint effect of creep and differential temperature offset the benefit of live load continuity. Also, 2nd Ed NZTA Bridge Manual load combinations including the effects of traffic loads together with temperature effects seem quite conservative as compared with some other international bridge codes, which works against the adoption of integral construction.
- Standardised girder reinforcement detail can reduce design and fabrication effort.
- Having the internal girders wider than the edge girders resulted in more even distribution of demand between girders and optimised the structural efficiency of each girder.
- The analysis and design of integral construction bridge using Super-Tee girders is considerably more complex than that for bridges utilising simply supported girders
- Shortening effects generate significant demands in continuous bridges. Delaying the continuity stitch pour and putting annulus sleeve over top section of piles can effectively reduce the amount of shortening demand. The first is highly dependent by the overall construction program.
- Although the initial purchase cost of a gantry is significant, for project with large length of 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 benefits. It is expected such benefit is reduced when the total length of bridge being constructed is shorter.
- 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 the GNRI ramps with single column piers and cantilever crossheads, where construction load cases could end up governing the permanent works design.
- Effective interaction between bridge permanent works, temporary works and gantry designers right from the start of the design is important so design and construction risk is minimised. The cooperative approach also provides the opportunity to maximize the efficiency of temporary and permanent works.
- With careful planning of the reinforcement placement and detailing, the complex integral pier construction can be trouble free.
- Use of 3D modelling at the reinforcing detailing stage of complex elements provides the opportunity for the construction team to visualise potential issues during construction.

Acknowledgement

The authors acknowledge the NZ Transport Agency and the Well-connected Alliance for the permission to publish this paper.

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