DESIGN OF INTEGRAL AND HIGH SKEW BOX GIRDER BRIDGES – SH18 HOBSONVILLE DEVIATION, AUCKLAND

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John has over 14 years experience specialising in the design of bridges. John’s experience includes the design of a variety of civil and bridge projects located in New Zealand, the United Kingdom, Asia and Australia. He leads Connell Wagner’s Auckland bridge team and was the design leader for bridges and structures on the Hobsonville Deviation Project.

Artist impression of Trig Road Underpass
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SYNOPSIS
The SH18 Hobsonville Deviation and SH16 Brigham Creek Extension Project forms part of the Upper Harbour Corridor extension for the Auckland Western Ring Route. The project includes a total of six vehicle traffic bridges which consist of five integral super-t bridges and a concrete box girder bridge.

The five super-t bridges are designed as fully integral structures with no articulation or movement joints at either piers or abutments. This was done to reduce maintenance costs, in recognition of whole-of-life benefits, by eliminating girder bearings and expansion joints which both have a significantly shorter life expectancy than the bridge structure themselves.

The highway alignment of the concrete box girder bridge requires a two span structure on a skew angle of approximately 67 degrees with a varying deck width. The deck was designed as two separate cast insitu post tensioned concrete box girders. The length of the concrete box girders sections vary with bridge skew and splay from 35m to 45m. The deck is integral with the central pier and has pot bearings and expansion joints at the abutments.

INTRODUCTION
The Hobsonville Deviation is the final project for the SH18 motorway and is valued at NZD$220 million. It includes a 6km section of four-laned highway (SH18) between Greenhithe and Westgate. The project also includes a 3km extension of dual-lane highway of SH16. There are four interchanges as part of this project, Buckley Avenue, Brigham Creek Road, Trig Road and Hobsonville Road and a new interchange between SH18 and SH16 including a flyover.
There are a total of seven bridges on this project, six road bridges and a pedestrian bridge. The road bridges include three single span and two double span, precast super-t integral bridges and one cast in-situ post tensioned box girder bridge. The cable stayed pedestrian bridge at Clarks Lane is not part of the focus for this paper, but will be discussed in another paper.

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Span Arrangement</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buckley Avenue Underpass</td>
<td>Single span 31m, no skew</td>
<td>Integral super-t bridge</td>
</tr>
<tr>
<td>Clarks Lane Footbridge</td>
<td>Two spans, 2x30m. no skew</td>
<td>Cable stayed bridge</td>
</tr>
<tr>
<td>Brigham Creek Overpass</td>
<td>Single span 25m, 13° skew</td>
<td>Integral super-t bridge</td>
</tr>
<tr>
<td>Trig Road Underpass</td>
<td>Two spans, 2x32m, no skew</td>
<td>Integral super-t bridge</td>
</tr>
<tr>
<td>Hobsonville Road Underpass</td>
<td>Two spans, 34m and 24m, 9° skew</td>
<td>Integral super-t bridge</td>
</tr>
<tr>
<td>Eastbound Flyover</td>
<td>Two spans, 37 to 43m, with 67° skew</td>
<td>Post tensioned box girder</td>
</tr>
<tr>
<td>Totara Creek Bridge</td>
<td>Single span 32m, 30° skew</td>
<td>Integral super-t bridge</td>
</tr>
</tbody>
</table>

Table 1. Summary of Bridges

The project is a New Zealand Transport Agency (NZTA) project delivered as a Design and Construct tender by HEB Construction with Connell Wagner DC (CWDC) as the lead designer. Work on this project started in April 2008 and construction is due for completion in 2012.

This paper will discuss the design of the six road bridges, focusing on the design of bridges in New Zealand, maintenance and design life issues, integral bridge design and the design of box girder structures for high skew bridges.

DESIGN LIFE REQUIREMENTS

The required design life on this project was 100 years. Major components that require regular replacement are the girder bearings and expansion joints. Typically their life expectancy could be 25 to 30 years. This means they would need replacement 3-4 times over the design life of the bridge requiring road closures and significant disruption to the motorway. Recognising this during the tender design, NZTA put a strong emphasis on a whole life cost saving and asked to reduce the number of these components within the project where possible.

In consideration of this, during the tender design options to remove bearings and expansion joints from the design were explored. The adopted solution was to make the bridges integral meaning that the girders are locked into the headstocks removing the need for bearings. The expansion joints are replaced by an asphaltic plug joint at the end of the abutment headstocks. This option was used for five super-t bridges which had skew angles no greater than 30 degrees. The Eastbound Flyover had a high skew at 67 degrees and the use of bearings and expansion joints could not be avoided.
DESIGN LOADING IN NEW ZEALAND

Live traffic loading has been designed for loads specified in Section 3 of Transit New Zealand Bridge Manual (TNZBM) which includes HN-HO-72 Traffic Loading. This loading comprises of a combination of both uniformly distributed loads and axle loads for both normal (HN) and overload (HO) cases. Structural analysis software SAM has been used to produce a grillage model of each bridge and determine the worst case loading on each structure incorporating the HN-HO-72 loading module.

Earthquake loading has been determined using the response spectrums from the TNZBM. These spectrums were developed for different types of soils, the stiffer the soil the higher the loading during an earthquake. These values were then modified by factors for location, ductility and risk. As these bridges are located in Auckland the values are not as severe as in other areas of New Zealand. The bridges have been designed for an elastic response under an event with a return period of 1 in 2500 years. Although this increases the loads applied to the structure, it provides the simplicity for structural detailing and limited the prospect of the structure being damaged during an earthquake event for its design life. This allows the structures to maintain service after a significant seismic event.

Thermal effects have been considered for both a global case and a differential effect. The asphaltic plug joints which connect the bridge deck and the approach road surface can accommodate 15 to 20mm thermal expansion and contraction. Differential temperature effects have been designed for a by considering a temperature profile throughout the structure with hot top and cold bottom. This causes the girders to rotate at the ends and as they are locked in loading is induced into the piles.

Permanent effects such as creep and shrinkage are considered with special consideration, especially the differential effects. Construction sequence and methods are critical in calculating the differential effects of both creep and shrinkage. To model the worst case for differential creep and shrinkage between the girders and in-situ deck, the deck was assumed to be poured 180 days older than the precast girders. A separate load case for girders to be erected and cast in after 20 days was also to model a worse case for global creep and shrinkage. Due to the bridge integral nature, the rotation at the support is restrained and induced secondary loadings onto the piles.

GEOLOGICAL CONDITIONS

The typical geological condition of the site is shallow, medium to weak rock of East Coast Bay Formation (ECBF) overlayed by Tauranga Group Alluvium. The general depth of the ECFB is found at shallower depth towards the western side of the project and deeper at the Greenhithe side.

The geotechnical design parameters adopted for the ultimate bearing capacity and the skin friction are 5 to 8 MPa and 200 KPa respectively.
DESIGN OF SUPER-T BRIDGES

Super-T Precast Girders
During the tender design stage various girder systems were considered for the six road bridges. It was then decided that super-t girders were the preferred option because they have been proven to be an economical solution up to 35m spans. In this project integral connections were utilised to eliminate bearings at supports and allowed for the super-t girders to span further due fixity at supports.

![Figure 2. Trig Road Underpass Elevation](image)

For economical and detailing simplicity all the road bridges were reviewed in an attempt to use 1200mm deep super-t girders. The additional benefits of this for the contractor, HEB Construction, would allow them to utilise their existing 1200mm super-t girder mould and provide a better control over the logistic supply of the precast girders. Through our preliminary design process we determined that 1200mm super-ts can accommodate spans up to 32m for bridges with integral connections. The final designs resulted in three bridges: Trig Road Underpass, Brigham Creek Overpass and Buckley Avenue Underpass using 1200mm Super T girders. The remaining bridges: Hobsonville Road Underpass and Totara Creek Bridge have spans longer than 32m or had significant skew angles and required 1500mm super-ts.

![Figure 3. Cross Section of Trig Road Underpass](image)

Structural Modelling
Two main analysis software packages were used during the detailed design stage: SpaceGass and SAM. In each programme a grillage model of the complete structure was built including integral piers and abutments. The influence of the fixity at each bridge was dependent on the relative stiffness between the girders and pier/abutment system with the effect of soil stiffness modelled using Winkler soil springs provided along the embedded length of the piles to account for different soil parameters at each location.

Space Gass is a general structural engineering software which was used to analyse and determine seismic design actions and displacement of of bridge substructures. In Space Gass the structural models were set up assuming that all prestressed members and deck slab had uncracked section properties. All other members (piers, piles, headstock beams) were assumed to have cracked section properties based on approximately 40% of the gross section stiffness. Torsional stiffness of each girder was taken as 20% of the gross torsion section and 40% for headstocks and piers. Parapets and footpaths were applied as super-imposed dead loads only and assumed they were not contributing to the stiffness of the superstructure.

Seismic design of the bridges was based on Section 5 of the TBM. Moments and shears induced in superstructure elements from horizontal seismic loading were generated using three-dimensional models in SpaceGass as shown in Figure 4.

Our seismic design approach assumed an elastic response for the design earthquake. Analysis to ensure the bridge will not collapse after an earthquake with a return period significantly larger than the design earthquake was undertaken, as per the requirement noted in Clause 5.1.1 of Transit Bridge Manual.
SAM is an integrated bridge design software which predominantly used to model the bridge superstructures using the HN-HO-72 live loading module. Using SAM, “built-in” effects such as temperature change, differential temperature, long term creep and shrinkage, differential shrinkage and residual creep were included in order to assess their impact on superstructure and substructure members. Creep and shrinkage were modelled as equivalent negative temperature changes based on approximate long term strains calculated based on Section 6 of the AS5100.5-2004. Differential temperature loading was applied across the depth of the section in accordance with the TNZBM. Differential shrinkage and residual creep were assessed in accordance with AS5100.5-2004 and RRU Bulletin 70 “Creep and Shrinkage in Concrete Bridges”.

Superstructure Design
The Super-t girders were designed as Class C prestressed beams in accordance with Clause 19.3 of NZS3101:2006 Concrete Structures Standards. For the project NZTA placed a specific serviceability limitation on the stress increment after the girder reaches decompression moment and maximum crack width which superseeded the limitation noted in NZS3101. The allowable stress increment to the reinforcement and tendons after a decompression moment was limited to 150MPa and the crack widths limited to 0.2mm. See tabel 2 below.

<table>
<thead>
<tr>
<th>Stress Case</th>
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<tr>
<td></td>
<td><strong>II</strong></td>
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<tr>
<td></td>
<td>Permanent loads plus variable loads of long duration, or</td>
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<td>permanent loads plus frequently repetitive loads</td>
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<td>Tension in</td>
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<td>0.5√f'c</td>
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<tr>
<td></td>
<td>0.25√f'c</td>
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<tr>
<td>Steel stress range</td>
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<tr>
<td>(MPa)</td>
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i. Maximum tensile stress
### Load Category

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<td></td>
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<td>B1, B2</td>
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<td>Prestressed Concrete</td>
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<tr>
<td></td>
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<td>C</td>
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<td></td>
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**ii. Maximum crack width**

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### Girders

Girders were designed as partial prestressed members for the flexural sagging moments and as non-prestressed reinforced concrete elements for hogging moments at supports.

### Substructure Design

Dead load, live load, earth pressure, temperature, creep and shrinkage loads were applied and a series of load combinations created to cover the critical combinations (1A, 1B, 3A and 4) in accordance with Section 3 of the TNZBM. Cracked section properties were assumed for piles and headstocks with an approximated cracked stiffness of 40% that of the gross section property.

A non-linear static analysis was carried out for these load cases and combinations to take p-delta effects into account. A minimum embedment length of 5m was adopted for all bridges to assure sufficient capacity for the lateral force embedment.

All the abutment piles in this project were isolated from the ground pressure by either placing oversized casings around the piles or locating the piles in front of retained abutments. For the abutment headstock however, a strain racheting effect is expected to occur due to the repetitive thermal expansion and contraction of the bridge deck pushing against the soil behind the headstock. Strain racheting increases in the stiffness properties of the ground due to repetitive compaction of the soil. This produces soil loading onto the headstock that in turn is transferred into the deck via the integral abutment.

Precast fascia panels are utilised at pier locations and exposed abutment piles. These panels reflect the urban design concept which has been adopted throughout this Hobsonville Deviation project.

### Detailing of Reinforcement for Integral Connections

The integral connection at bridge supports was achieved by providing both positive and negative moment capacities for different load combinations. Positive moment capacity was required for seismic load combination and provided by anchoring 6HD25 bars into
headstocks. Two layers of closely placed deck reinforcement were utilised to meet the negative moment capacity demand for deadload and liveload cases.

Each super-t girder was connected to adjacent girders with in-situ diaphragms at support locations. A typical detail of two rows of reidbar couplers were cast into ends of the girders to allow bars to be inserted on site to form laps in the diaphragms.

![Section at Pile Location](image1)  ![Section at Girder Location](image2)

**Figure 6. Typical Abutment Connection**

**Construction Issues**
Some of the significant aspects which needed to be considered during construction stages were:
1. Alignment of abutment and pier piles reinforcement to avoid clashing with diaphragm reinforcement. A significant number of pile reinforcement protrudes into abutment and pier headstock resulted in tight tolerance between reinforcements. To mitigate these issue pile reinforcement is bundled into pairs and aligned prior to piles being poured.

2. Confirmation of girder hogs at the time of erection to ensure that 160mm minimum deck thickness is achieved throughout the span. Significant variation of hogging between girders within the same span may occur due to the different age of girders. Anticipation for these variants was taken by adjusting the thickness of shims placed under each girder prior to decking being poured.
3. Erection and installation of pier cladding panels under previously erected bridge deck. The height of precast fascia is approximately 300mm less than the vertical clearance underneath the bridge. Panels are proposed to be installed using a tilt up process utilising heavy duty forklifts or suitable mobile cranes.

HIGH SKEW BOX GIRDER BRIDGE DESIGN

The SH18/SH16 Eastbound Flyover Bridge is a high speed motorway interchange for east bound traffic from SH16 to SH18. It is a two span continuous post tensioned twin box girder on a high skew angle.

Alignment
The flyover bridge, carries the SH18 eastbound on ramp over the SH16 dual carriageways. The bridge is aligned on a 550m radius with a constant 6% super elevation.

The bridge includes 2 x 3.5m wide traffic lanes with a varying width merging lane. A 3.5m wide shoulder is provided on the verge side of the carriageway and on the median side a varying width shoulder is provided including additional setback for stopping sight distance. The lane arrangement under the bridge comprises 2x3.5m wide traffic lanes northbound with a 4.2m verge side bus shoulder, and a 1.8m median shoulder. The southbound carriageway provides a 3.5m traffic lane with a 4.2m verge side bus shoulder, and a 2.2m median shoulder. The central median is 10.2m wide allowing for the future provision of an additional 3.5m southbound lane whilst retaining sufficient median width to allow for sight distance past the bridge piers. The resulting bridge deck has a skew angle of approximately 67 degrees and a varying width due to the lane configuration.

Deck drainage of surface water is accommodated with surface flows across and along the bridge to drainage located beyond the bridge abutments.
Alternatives Considered
The geometry of the flyover structure presented a challenge during the development of the design. Several options were considered including a structure comprising three square spans with pier headstocks spanning across the traffic lanes of the SH16, similar to the specimen design. This could have been constructed as a fully integral structure thus offering whole life cost saving benefits. However in comparison to the two-span skewed option, this option had a significantly higher deck area together with long-span headstocks which may have required post tensioning.

A two span bridge with trapezoidal steel box girders spanning over 40m was also considered. Escalating construction costs associated with steel prompted a review of the structural form resulting in the adoption of the two span continuous post tensioned concrete box girder super structure cast integral at the pier.

Span Arrangement and Articulation
The bridge is a two span continuous structure. The bridge deck is continuous and cast integral with the pier, whilst pot bearings and bridge expansion joints are provided at the abutments. Transition slabs are provided for the approaches.

![Figure 8. Eastbound Flyover Elevation](image)

As a result of the high skew and varying width due to the lane configuration the span length varies across the width of the structure. The length the concrete box girders sections vary from approximately 37m on the west structure to approximately 43m on the east structure.

Knock off corbels are provided at the abutments to allow for excessive longitudinal superstructure deflections under seismic loading. All bearings are free float bearings an anti-lifting bearing provided at the acute the south-west corner. Lateral restraint blocks are provided between the box girders at each abutment to transfer transverse forces into the abutments.

Substructure
The abutment crosshead beams are supported by 1500mm diameter bored piles. These are sleeved through MSE abutment walls constructed prior to construction of the
superstructure. The sleeves will extend to the base of the MSE walls to ensure that no bridge loading is passed to the MSE walls.

The central pier comprises two 2000mm diameter concrete columns located in the SH16 Motorway median. Each column is seated on a pile cap and supported by 4-1200mm diameter bored piles.

**Superstructure**

The superstructure comprises twin cast insitu post tensioned 2.0m deep concrete box girders. The two structures converge along the length of the structure and are connected by an insitu deck stitch that tapers with the bridge geometry. Deck cantilevers vary along the length of the structure to accommodate the curved road geometry.

An internal diaphragm is provided at the abutments and at the piers. At the piers the diaphragm is orientated perpendicular to the span rather than diagonally between the columns.

Each box girder has 8 No. tendons (two in each web) of 19 No. 15.2mm diameter strand, the profiles are draped along the deck length. Stressing of the tendons is designed to be undertaken from each end to balance friction losses. The option to stress from both ends of the tendons was investigated but not adopted as the construction preference was to undertaken stressing with a single jack where possible.
Edge Protection on Bridge Deck
Rigid concrete barrier edge protection is provided along the outside edges of the bridge deck. The barriers comprise pre-cast panels up to 6m long and an in-situ concrete stitch to connect the precast element to the deck. A 15mm gap filled with sealant is placed between each unit (along the precast and insitu stitch) to prevent the barrier contributing to the edge girder stiffness. The barriers will incorporate urban design pattern on the outer face. The barriers provide performance level 5 containment to NZTA requirements.

Design and Construction Considerations
The twin box girder arrangement was adopted to alleviate the torsional effects generally associated with high skew bridges. By adopting this arrangement the bridge is forced to span longitudinally along the length of the box girders as opposed to spanning diagonally between obtuse corners of the bridge deck. The deck stitch between the two converging longitudinal elements has been thickened and heavily reinforced to accommodate seismic design actions as it is twisted and turned during a design seismic event.

The use of the twin box girder arrangement results in a heavier bridge deck which poses additional design issues by increasing the structure dead load and corresponding seismic actions. Combining the heavy superstructure with a high skew also results in some significant lateral loads at the abutments under torsional seismic loading. As the deck elements are released to move longitudinally with lateral restraint provided orthogonal to their line of movement the resulting lateral loads are passed onto the abutments at a relative skew. These lateral forces are then resolved into in plane and out of plane components. Furthermore as the abutment piles are isolated from the surrounding MSE abutments they have a significant free length and acting as cantilevers to resist the out of plane forces are subjected to high bending moments.

The construction sequence requires the pier piles and columns to be constructed conventionally with the abutment piles cast to road level and formed as columns to the underside of the abutment headstocks. The abutment piles are isolated from the MSE walls with precast concrete rings and MSE walls constructed around these. The deck is to be cast on imported fill which is to be stock piled on site under the bridge. This avoids the need to erect false work and also provides a large platform which avoids the need to work at heights.

The deck box sections are to be formed and cast and stressed separately to avoid stress in the slab between each box. The stich slab is to be cast on completion for the tendon stressing sequence.
CONCLUSION

The integral super T bridges and high skewed box girder are designed for New Zealand seismic loads provided a number of challenges through the design process. The integral nature of the Super T bridges create stiff portal frame actions which induce high flexure and shear forces requiring careful detailing of the integral connection between the precast beams and the superstructure. The high skew for the box girders generate torsional effects on the bridge superstructure and substructure when resisting seismic force.

At the time of this paper submission the construction is well underway for the bridge substructures. The SH18 Hobsonville Deviation and SH16 Brigham Creek Extension project is expected to be finished in 2012.

ACKNOWLEDGEMENT

We acknowledge the parties involved in the project: the project principal: NZTA, the contractor: HEB Construction and the urban designer of the project: Jasmax Architect.

REFERENCES

2. NZS3101:2006 Concrete Structures Standards
3. AS5100:2004 Bridge Design
4. NZRUU Bulletin 70 – Road Research Unit Bulletin No 70: Creep and Shrinkage in Concrete Bridges.