DESIGN AND CONSTRUCTION OF NEW ZEALAND’S FIRST 1825 SUPER TEE BEAM – MACKAYS TO PEKA PEKA EXPRESSWAY PROJECT

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SUMMARY

Pre-cast Super T Bridge beams were first introduced into New Zealand in 1995 during the construction of the Sylvia Park Viaduct and have been used by Kiwi engineers ever since. 1025 and 1225 Super T beams were introduced into “New Zealand Standard Bridge Beams”, for span ranges of 20 m to 30 m, in 2008. For Central Motorway Junction Stage 2 and Tauranga Harbour Link (THL) projects bridge engineers introduced 1525 Super T beams to push the spans of precast concrete bridges out to 35 m.

The Resource Consent for MacKays to Peka Peka (M2PP) Expressway Project came with a condition of providing a 35 m clear waterway channel under the Waikanae River Bridge. This required a span length of about 39 m, which was beyond the span limits of all available precast bridge beams in New Zealand. This led to the design and construction of New Zealand’s first 1825 Super T beams.

INTRODUCTION

The MacKays to Peka Peka (M2PP) Project

A new four lane expressway is currently under construction through the Kapiti Coast of New Zealand. The MacKays to Peka Peka (M2PP) Project is part of the new Kapiti Expressway, which passes through Kapiti, 60 km north of Wellington. The project is being delivered under an alliance contract model, a consortium formed of NZ Transport Agency (the Agency), Fletcher Construction, Higgins Group and Beca (the MacKays to Peka Peka Expressway Alliance). When completed the expressway will provide a modern and reliable route which crosses over local roads and waterways, and bypasses the local town centres over a distance of 18 km. The project includes seventeen bridges comprising six multi-span, nine single-span bridges over local roads, the expressway and streams, and two pedestrian bridges over the expressway.

The MacKays to Peka Peka expressway is a lifeline road link to the capital, which passes through both urban and rural areas. Pre-cast Super T beam systems have been adopted on the project due to their structural efficiency, stability, serviceability, economy of construction and pleasing aesthetics.
**History of Super T Bridge Beams**

The Super T bridge beams were first developed in Australia during the 1990’s. In New Zealand Pre-cast Super T Bridge beams were introduced in 1995 during the construction of the Sylvia Park Viaduct (Khan and Brown 2014). During the design stage the designer, Duncan Peters from Connell Wagner, carried out a comparative evaluation of the structural efficiency of commonly available standard precast beams in New Zealand at that time and the closed top Super-T beam, developed by VicRoads, for a span range of 21 m to 26 m to see which would be the most economical type to use for the project. The Super T beam was found to be the most efficient and it also had the advantage of providing a complete working surface for casting the in-situ deck slab on. This was an added advantage for the viaduct spans over the busy road and railway line. Based on the investigation, Super T beams were adopted for the viaduct. This was the first use of Super T beams in the New Zealand Bridge Industry (Khan and Brown 2014).

After successful use of Super T beams for Sylvia Park Viaduct, Super T beams were used for the highly skewed bridges like Southern Motorway Underpass Bridge, Mungavin Road Bridge in Porirua and Puhinui Interchange Bridge built on the SH16 near Auckland airport in 2001. In 2005 Super T beams were also used for the 413 m long, 15 span Hewletts Flyover in Tauranga, where the design was based on the partial prestress approach. In 2008 with collaboration between precasters and designers, 1025 and 1225 Super T beams were introduced into the “New Zealand Standard Bridge Beams”, for span ranges of 20 m to 30 m.

Good torsional resistance, flexibility in flange width, structural efficiency and an aesthetically appealing shape made Super T’s popular precast bridge beams. The history of bridge construction in New Zealand has shown that Kiwi engineers love new ideas, embrace new technologies and are not afraid to take on the challenge of pushing the boundaries. For the Central Motorway Junction Stage 2 (Khan and Brown 2014) and Tauranga Harbour Link (Joseph 2009) (THL) projects Bridge engineers introduced 1525 Super T beams to push the span of precast concrete bridges to 35m.

**Evolution of 1825 Super T Beams**

The Resource Consent for MacKays to Peka Peka (M2PP) Expressway Project came with a condition of providing a 35 m clear waterway channel under the Waikanae River Bridge, requiring a span length of about 39m. A desk top study was conducted to see what types of bridge beams had been used for this span range.

<table>
<thead>
<tr>
<th>Table 1: Bridges in New Zealand with large span precast beams</th>
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<tbody>
<tr>
<td><strong>Bridge Name</strong></td>
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<tr>
<td>Rosebank Patiki Interchange (Alan Powell and Nigel Snoep 1998)</td>
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<tr>
<td>Central Motorway Junction Stage 2</td>
</tr>
<tr>
<td>Tauranga Harbour Link (Joseph 2009)</td>
</tr>
<tr>
<td>Ngaruawahia Bypass – Lack Road</td>
</tr>
<tr>
<td>Waterview – Great North Road Ramp</td>
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<tr>
<td>Waikato Expressway Otanerua Viaduct</td>
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The study showed that a 39m span was beyond the span limits of all available precast bridge beams in New Zealand unless a wider table top crosshead is adopted as used for the Great North Road ramp at the northern corridor. This led the Alliance team to think outside the square and develop some innovative ideas to overcome the span restriction problem. The Alliance team carried out a preliminary study and evaluated the following three options (Khan and Brown 2013):

- 1525 Super T beams with 5 m wide cast-in-situ table top crosshead beams
- 1825 Super T beams with 2.5 m wide pre-cast crosshead beams.
- 2200 Steel composite beams with 2 m wide pre-cast crosshead beams.

The study showed that the 1825 Super T beam option provides a cost efficient, safe to construct, durable and aesthetically appealing solution, which would fit well within the urban environment. This led to the design and construction of New Zealand’s first 1825 Super T beams. The Alliance team developed the mould for the 1825 Super T beams, and established a state of the art pre-casting yard capable of producing the hundreds of Super T beams required for the M2PP project.

**DESIGN ASPECTS OF 1825 SUPER T BEAMS**

**Geometric cross section**

The geometric cross section of the 1825 Super T beams was developed from the existing standard shape used in New Zealand for 1225 Super T beams. The beam’s shape consists of a 100 mm thick open top flange and a 260-325 mm thick bottom flange. For a 100 mm thick web section the shear stresses generated in the end regions exceeded the maximum shear stress limitations in the concrete code NZS 3101 (2006). The solution was to increase the web thickness to 150 mm in the end region and 120 mm in the central region, which gave greater concrete shear capacity and allowed more width to place larger bars. Figure 1 shows a typical pre-cast 1825 Super-T beam cross section,

**Figure 1. Geometry of 1825 mm deep Super T Beam at midspan (a) and near beam ends (b)**

The bottom flange contains the pre-tensioned strands which are arranged in rows at 50 mm spacing. Four strands are also located in the top flange to assist in minimising tensile stresses during transfer. The strands are straight for the full length of the beam. Internal diaphragms are located at intermediate locations.
**Internal Diaphragm**
The purpose of intermediate diaphragms is to prevent any web distortions of the beam, particularly during transportation and erection. Internal diaphragms are provided at one-third of the length of the beams and restraint couplers installed in the top flange at mid-point between the internal diaphragms and end block. This was to limit the unrestrained length of the top flange to less than 8 m and minimise the number of internal diaphragms to improve construction efficiency. This also helps in reducing the lifting weight of the beam.

![Diagram of internal diaphragm](image1)

**Figure 2.** (a) Typical Plan of Super T beam and (b) Restraint coupler details

**End Block**
The purpose of end blocks is to develop the prestressing, allow placing of beam lifting hardware and to have adequate room for cross diaphragm beam connections. Besides typical end block detailing as shown in Figure 3, urban design aspects of the bridge pier appearance also required the detailing of dapped-ends of beams.

The detailing of the dapped-end block required special consideration (Mattock and Can 1979) to prevent durability weaknesses which have been historically encountered with dapped-end joints, in particular in this situation where tensile forces due to pre stressing induced end-zone equilibrium reduced the effectiveness of the vertical hanger bars. In this regard, earlier test results have shown, that the detailing of inclined reinforcing bars improves the behaviour of the dapped-end joints under service loading.
Creep and Shrinkage of Concrete

Shrinkage is the sum of chemical (autogenous) and drying shrinkage. Autogenous shrinkage is an important phenomenon in young concrete. At low water/cement ratios, all the water is rapidly drawn into the hydration process and the demand for more water creates very fine capillaries. Drying shrinkage is the contraction of hardened concrete due to the loss of capillary water. This shrinkage causes an increase in tensile stresses, which may lead to cracking, internal warping, and external deflection, before the concrete is subjected to any kind of loading. Autogenous shrinkage is not dependent on section geometry or humidity, while drying shrinkage is. The shrinkage of concrete induces losses in the prestressing force.

Concrete creep is a deformation of structure under sustained load. Basically, long term pressure or stress on concrete can make it change shape. This deformation usually occurs in the direction the force is being applied and creep also causes loss of prestressing force in the section.

Creep and shrinkage effects were derived in accordance with AS 3600 (2009), as recommended by NZTA Bridge Manual 3rd Edition (2013), with suggested adjustments. Beams were assumed to be 30 days old at the time of installation. Table 2 summarises the creep coefficients and shrinkage strains applied.

<table>
<thead>
<tr>
<th>Concrete age</th>
<th>Creep Coefficient</th>
<th>Shrinkage Strain</th>
</tr>
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<tbody>
<tr>
<td>T = 1 day</td>
<td>0.12</td>
<td>3x10^{-5}</td>
</tr>
<tr>
<td>T = 30 days</td>
<td>1.15</td>
<td>2.8x10^{-4}</td>
</tr>
<tr>
<td>T = 10,000 days</td>
<td>2.81</td>
<td>4.9x10^{-4}</td>
</tr>
</tbody>
</table>

Stress-stain relationship of pre-stressing steel

High strength and low relaxation pre-stressing steel does not have a defined yield point as the conventional reinforcement, therefore it is important to adopt a stress-stain curve, where an accurate stress in strands corresponding to the strain can be estimated. Equation (1) below was adopted in the design.

\[ f_{ps} = \varepsilon_{ps} \left[ A + \left( \frac{B}{1 + (C \varepsilon_{ps})^D} \right)^{1/D} \right] < f_{pu} \]  

The procedure for calculating the power formula constants for prestressing steel is outlined in a reference paper (Davalapura and Tadros 1992).
Relaxation of Stands due to Heat Curing

Elevated temperature curing was adopted to accelerate beam production. However, elevated temperature curing causes early relaxation of the strands. In such cases AS 5100.5 (2004) clause 6.3.4.4 recommends that ultimate relaxation shall be deemed to have occurred by the end of the curing cycle. For the purpose of calculating stresses at transfer a lower bound relaxation value of 5% was assumed and for the calculation of prestress losses an upper bound value of 10% was assumed.

Adopted Concrete Properties

The end block detailing, in particular the detailing of the dapped-end, drove the study of appropriate flow-able concrete mixes, suitable for congested reinforcement areas and to achieve a high quality finish. A parametric study was conducted using two different concrete mixes. As part of the study, each mix design was assessed in regards to creep and shrinkage behaviour, prestress loses, shear and torsion design of beams.

Table 3: A comparison of concrete properties to two mix designs

<table>
<thead>
<tr>
<th>Concrete Parameters</th>
<th>Mix No. 1</th>
<th>Mix No. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameters</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength, $f'_c$ (MPa)</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td>56 Days Drying shrinkage strain, $\varepsilon_{csd}$ ($\mu$)</td>
<td>1200</td>
<td>1000</td>
</tr>
<tr>
<td>Maximum aggregate size (mm)</td>
<td>19</td>
<td>13</td>
</tr>
<tr>
<td>Autogenous shrinkage strain, $\varepsilon_{cse}$ ($\mu$)</td>
<td>50</td>
<td>65</td>
</tr>
<tr>
<td>Design expected shrinkage strain, $\varepsilon_{cs}$ ($\mu$)</td>
<td>650</td>
<td>553</td>
</tr>
<tr>
<td>Short term creep coefficient, 30 days, $\varphi_{cc}$</td>
<td>0.898</td>
<td>0.963</td>
</tr>
<tr>
<td>Long term creep coefficient, 10,000 days $\varphi_{cc}$</td>
<td>2.137</td>
<td>2.29</td>
</tr>
<tr>
<td>Losses as % of $P_j$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Short term creep losses</td>
<td>4.9%</td>
<td>5.2%</td>
</tr>
<tr>
<td>Short term shrinkage losses</td>
<td>3.9%</td>
<td>3.4%</td>
</tr>
<tr>
<td>Long term creep losses</td>
<td>1.7%</td>
<td>1.9%</td>
</tr>
<tr>
<td>Long term shrinkage losses</td>
<td>4.5%</td>
<td>3.6%</td>
</tr>
<tr>
<td>residual creep losses</td>
<td>-0.8%</td>
<td>-1.0%</td>
</tr>
<tr>
<td>Differential shrinkage</td>
<td>0.1%</td>
<td>0.1%</td>
</tr>
</tbody>
</table>
The study showed that shrinkage reduced but creep increased for higher strength concrete. The losses in prestressing force reduced by 1-3% for higher strength concrete. The shear strength calculated from NZS 3101 (2006) code equation 9.5 is reduced by aggregate size, further the code limits concrete strength when calculating shear strength to 50 MPa. Since clause 7.5.2, limits the maximum nominal shear stress, $v_{\text{max}}$, to 8MPa, there is no benefit available for increased concrete strength and also no detrimental effect from reduced aggregate size. Therefore, in order to provide some flexibility for the construction team, the 1825 Super T beams were designed for concrete Mix No 1 and checked for Mix No. 2.

**Torsion constants adopted**

There are a number of approaches being used both in Australia and New Zealand for modelling the torsional rigidity of Super 'T' beams in grillages.

NZ 3101 (2006) clause 7.6.1.3 states “If torsion in a member arises because the member twists to maintain compatibility, the effect of torsion on the member may be neglected provided the requirements of either (a) or (b) below are satisfied...” Therefore, this clause of NZS 3101 (2006) allows torsionless design of members where torsional strength is not required for equilibrium. Hambly (1998) states that torsionless design methods should not be used when beams have high torsional stiffness (as is the case with Super 'T' beams). Wyche (Jeo Wyche 2000) maintains that Super 'T' beams should be designed for torsions. Another approach is to assume a reduced torsional constant assuming the section cracks in torsion. Typically designers in New Zealand are using a value ranging from 0.1 $J_g$ (torsion constant) to 0.2 $J_g$. At ULS, Super Tee beam members have been treated as reinforced concrete members and it is generally accepted that the their flexural stiffness is about 50% of the gross stiffness. With a ratio of 10% of gross torsional stiffness to 50% of gross flexural stiffness for reinforced concrete members, 20% of gross torsional stiffness and full flexural stiffness has been assumed for Super T beams at the service limit state. The reduced torsional stiffness approach is considered reasonable and has been adopted for the beam designs.

**Survivability Checks**

1825 Super Tee beams have been designed to satisfy the serviceability limit state (SLS) requirements of NZS 3101 (2006) Clause 19.3.3 using the partial prestressing approach for the load combinations in Tables 3.1 of NZTA Bridge Manual (2013). The following limits in stress changes in the pre-stressing and non-prestressing reinforcement have been adopted:

- prestressed tendons
  - $\Delta f_s < 150$ MPa for HN live loads cases
  - $\Delta f_s < 200$ MPa for HO live loads cases
- for non-prestressing reinforcement
  - $\Delta f_s < 150$ MPa

The concrete tensile strength was ignored in the design. Prestressing strands have been debonded along their length as necessary to ensure the appropriate stress limits, as per NZS 3101 (2006), are complied with.

**Issues identified with NZS 3101 (2006)**

Clause 7.5.2 & 19.3.11.1 Maximum nominal shear stress, $V_{\text{max}}$:

These clauses limit the nominal shear stress of concrete to the smaller of 0.2$f_c$ or 8 MPa. Due to this arbitrary limits of 8MPa for maximum nominal shear stress there is no benefit for the
use of high strength concrete. Further clause 19.3.11.1 states that in calculating the shear stress resisted by concrete, $f'_c$ shall not be taken greater than 50 MPa.

These clauses limit the use of high strength concrete in New Zealand bridge industry. In comparison, in other international codes, there are no such limits are set out.

**AS 3600 (2009)**
Clause 8.2.6, equation 8.2.6 $V_{u,\text{max}} = 0.2f'_c b_v d_o + P_v$

**AS 56001.5 (2004)**
Clause 8.2.6, equation 8.2.6 $V_{u,\text{max}} = 0.2f'_c b_v d_o + P_v$ or $V_{u,\text{max}} = 0.2 (0.85f'_c) b_v d_o + P_v$ at transfer.

Clause 5.8.3.3, Nominal Shear Resistance
The nominal shear resistance, $V_n$, shall be determined as the lesser of:

$$V_n = V_c + V_d + V_p$$

$$V_n = 0.25 f'_c b_v d_o + P_v$$

As can be seen from the above, the Australian codes restrict the maximum shear stress to $0.2f'_c$ and in American code this is limited to $0.25f'_c$, but neither of the codes placed any arbitrary number on the maximum shear stress or require that the concrete strength $f'_c$ shall not be greater than 50MPa in calculating the shear stress resisted by concrete. These restriction limits in NZS 3101 (2006) resulted in more onerous bridge beams design as compared to other international codes.

In the New Zealand bridge industry the use of 50MPa concrete is common, and the constructors and designer are keen to use high strength concrete to optimize bridge beam design. It is not clear why this 50MPa strength limit is placed on the shear design of prestress beams in NZS 3101 (2006), where for flexural ductile element design the code allows the use of 70 MPa concrete. In this regard, the authors believe that an alignment of NZS 3101 (2006) to other international codes such as AS 3600 (2009), AS 5100 (2004) and AASHTO LRDF (2012) will result in more economical design of bridge beams.

**Clause 7.6.1.3 Torsion due to defamation compatibility:**

This clause of NZS 3101 (2006) allows torsionless design of members where torsional strength is not required for equilibrium. The torsionless design approach permitted in the code is likely to produce non-conservative results with respect to combined shear and torsion. Clarification on the appropriateness of torsionless design methods for torsionally stiff bridge beams is required in the NZS 3101 (2006) code.

Clause 7.6.4.1 Design moment for torsion:

Fenwick and Cook’s (2010) proposed amended equation (a) was used in designing the reinforcement for nominal torsion moment, as below:

\[
(a) \quad T_n = 0.44 A_{ctc} f_c \left( 1 + \frac{N}{0.33 A_t f_c} \right) \quad \text{or} \quad (a) \quad T_n = 0.10 A_{ctc} f_c \sqrt{f_c} \left( 1 + \frac{N}{0.33 A_t \sqrt{f_c}} \right)
\]

NZS 3101(2006) equation \quad Richard & Dean’s (2010) equation
Clause 8.3.9 Spacing between pre-tension steel:

The clause requires a minimum clear spacing of 3 times the diameter of strands. If we are to take $d_0$ as the nominal diameter of the strand of 15.2 mm, the clear distance would have to be 45.6 mm and the spacing of the strands would have to be 60.8 mm. This clause was introduced in 2006 and in the commentary of this clause, it is mentioned that these requirements are provided to prevent weakening planes for splitting bond failure developing in the cover concrete.

For the design of the 1825 Super T beams a reduced spacing of 50 mm centre to centre was adopted, on the basis of other international references, as below:

- Russell and Burns (1993) reported no difference in measured transfer lengths for 15.2 mm diameter strands at 51 and 57 mm spacing.
- Burdette et al. (1994) reported similar findings for 12.7 mm diameter strands at 44 and 51 mm spacing.

Naturally, there is a minimum spacing for a given strand diameter at which the splitting resistance of the concrete will be exceeded. However, it is important to note that splitting was not observed in the tests of 15.2 mm diameter strands at 51 mm spacing by Russell and Burns.

The wording of clause 8.3.9 of NZS 3101 (2006) is quite similar to clause 7.6.7.1 of ACI-318 (1995). However, in the 2002 version of ACI-318 (2002) clause 7.6.7.1 was modified and a closer spacing of 44 mm for 12.7 mm strand and 50 mm for 15.2 mm strand is allowed if the concrete compressive strength at transfer ($f_{ci}$) is 4000 psi (28 MPa) or more. AASHTO LRFD Bridge Design Specifications (2012), also allows similar spacing of stands.

The specified concrete compressive strength at transfer, $f_{ci}$, is 33 MPa, so the adopted 50 mm centre-to-centre spacing is in accordance with ACI-318 (2002) code and AASHTO LRFD Bridge Design Specifications (2012).

MANUFACTURING 1825 SUPER T

The Alliance team developed the mould for the 1825 Super T beams, and established a state of the art pre-casting yard capable of producing hundreds of Super T beams required for the M2PP project. This section presents the design and construction methodology for New Zealand’s first 1825 Super T beams. It also demonstrates how the Alliance team carried out the investigations and trials for the construction of complex end blocks, use of high slump concrete, hot-water curing and the establishment of a state of the art pre-casting yard. Some of the challenges are discussed in this section.

The planning

The biggest challenge was to find a place to build these very large beams. The first three sites fell through because of resource consenting complications. The final solution was to locate the yard alongside the main alignment in Otaihanga between a wastewater treatment plant and public refuse site.
The process
Production efficiency became a focal point and any opportunity to reduce the cycle time had to be exploited at the outset with efficient and effective setting out of the yard and processes. The average time for manufacturing a beam was 2.8 days, at peak production the plant produced a beam per day.

The first step was the assembly of the reinforcing cages in special jigs. The dapped-ends were very congested with reinforcing which affected concrete flow. Mock cages were built to simulate the worst cage and different bars sizes and concrete mixes to achieve a good finish with no concrete voids. The samples were cut in half with diamond wire saws to ensure the strands were completely encapsulated with well compacted concrete.

After the mould had been cleaned and oiled the cage was lifted by using a 40 m lifting beam. Once the cage was in position stressing strands were fed through the live end of the stressing beam and pulled through to the dead end and locked off with stressing barrels and wedges. After all of the strands had been installed they were individually tensioned using a small handheld jack. A check was done to ensure that there were no crossed over strands at this stage. As the load increased with each successive strand, the stressing beam started to deflect and the compression struts shortened. The previously stressed strands lost some tension as the system moved, so a second round of tensioning was required.
The stressing frame at the live end housed the hydraulic rams and the stressing beam (dead end only has a beam). The 2 m deep stressing beam was designed to transfer a force of 1,120 tonnes into the 60 strands via the jacks. The stressing strands were locked off against a block of steel plating 500 mm thick. The main jacks pushed the stressing beam along in increments of 100 mm. The locking rings were also driven forward to prevent the rams from releasing if a failure were to occur anywhere in the hydraulic system. The beam was advanced until the desired pressure was achieved. At this point the overall extension of the strand was recorded and compared against calculated values. Hi-definition camera’s allowed staff to measure deflections without standing directly next to the stressing operation.

The internal diaphragm steel and void forms were the last things to be installed prior to the concrete pour. The pour itself took an hour to complete. To ensure we achieved a F5 surface finish, 22 variable frequency electric form vibrators were installed. The positioning, orientation and calibration of these motors was done by the German supplier. Controlling the frequency of the units allowed the amount of energy imparted into the concrete to be controlled. It also controlled individual units and created a ‘wave’ of energy that followed the concrete pour.

Accelerated curing of the beams was achieved by circulating hot water around the skin of the mould via a network of tubes and pipes. With the cranes already consuming enough electricity to power a small city, a LPG fired boiler was used. A typical heating run in the middle of winter would consume about 190kg of LPG overnight.

An important part of the process was to know when the concrete reached its transfer strength. The thermal maturity of the concrete was well correlated to its strength. Sacrificial probes cast into various positions of the beams relayed temperatures to a data logger. Once the concrete was at strength, the void forms were removed and the beam de-stressed.

During the de-stressing, the ends of the beam had to be lifted to prevent them from crushing the mould as the beam hogs. It was a fine balancing act because too much lifting could damage the beam. Both the beam and mould were dragged towards the dead end as the strands on this end shortened. The mould was periodically dragged back to its original position.

The tops of the beams and the end diaphragms were water blasted to achieve a type B construction joint which generated lot of caustic concrete slurry that has to be collected and drained to a treatment facility. Any defects were noted and made good before the beam was transferred to the storage yard.

All concrete laden storm water had to be treated and was drained to a pH treatment facility that could fully neutralise up to 36,000L of highly caustic water per hour.
Manufacturing Challenges
The barrels and wedges probably had the biggest impact on production. The design called for a strand spacing of 50 mm. Most barrels and wedges were designed for 55 mm spacing so had to be sourced from overseas. An Italian supplier, requested samples and some stressing trials undertaken to determine their suitability. After 5 months of production some breaking of wires occurred while stressing. Worked stopped for a month to investigate the cause and identified that the barrels and wedges were faulty. The inside of the barrels were stretching and affecting the angle of bite on the wedges. The supplier undertook their own investigation and confirmed that the metal being used was softer than the required specification. It was changed to a German supplier and the problem went away.

Figure 10: A broken outer wire
Figure 11: A damaged barrel and

The ends of the cages posed the biggest threat to productivity because of the strand/reinforcing congestion and the presence of critical inserts. Add to this a positional accuracy requirement of plus or minus 2 mm for all coupler bars/inserts and there was a sleepless month of problem solving.

Bearing inserts are normally fixed to the mould using magnetic fasteners but very prone to being knocked out of position. Thick steel templates to lock the bearing inserts into position. These bearing templates were bolted to the stop ends. The stop ends were checked for correct orientated and the correct distance apart.

Special steel templates were used to accurately secure the transverse bars into position. The threaded couplers on the ends of the transverse coupler bars enabled the two opposing templates to be bolted together and created a very rigid frame. The steel templates also incorporated a cunning mechanism that enabled bars to be fed in from the sides of the cage making assembly a lot simpler.

Figure 12 Bearing insert and coupler bar templates sitting in the reinforcing cage end block jig.
Figure 13: A completed stop end showing the transvers couplers.
INSTALLATION ASPECTS OF 1825 SUPER T

Transportation
The Super-T beams have good stiffness properties for bending in the non-principal axis which gives the beam rigidity during lifting and transportation. The long span Super T beams were easily transported using a prime mover at one end of the beam and an independently steered jinker at the other end of the beam.

![Image of transportation of Super-T girders using a prime mover and steered jinker.](image)

Figure 14. Transportation of Super-T girders using a prime mover and steered jinker.

Lifting of 1825 Super T Beams
Lifting of long span Super T beams is one of the critical aspects of the bridge construction. The end blocks at both ends are designed and detailed to accommodate the placing of beam lifting hardware. Cast-in wire rope lifting hoops were adopted as these are the most economical way of providing a crane hook attachment in Super T beams. However, these require relatively large edge distances. Consideration was also given to the exposure of steel rope hoops after the beams were placed. Once the beams were finally in position, protruding hoops were cut off, and the appropriate corrosion protection coating applied to the cut ends to avoid rust staining. The flexibility of steel rope makes it the safest method for forming a cast-in hoop.

![Image of lifting of 1825 Super T beams using a mobile crane.](image)

Figure 15: Lifting of 1825 Super T beams using a mobile crane.

Constructability
Another benefit of the Super T beams is the versatility in erection techniques. The 1825 Super T beams were installed by a 400T mobile crane (biggest crane of New Zealand). The beams were delivered to the site, lifted and installed by the mobile crane span by span.
The most dominant feature of the Super T beams is the safe working platform that the beams create once erected. The outer flanges are erected edge to edge with the adjacent girder and sacrificial formwork is placed in the open section. Temporary hand rails are attached at the edges which reduced the risk of potential falls.

CONCLUSIONS

The conclusions from the design and construction of New Zealand’s first 1825 deep Super T beams are:

- Innovation happens when designers and constructors collaborate to put existing concepts together in a new way. By pushing to find innovative solutions in the design and construction for M2PP Bridges, achieved significant construction efficiencies and savings have been achieved.
- Collaboration between designers and constructors allowed the construction use of New Zealand’s first 1825 Super T beams and the longest pre-cast bridge beam of New Zealand for the M2PP project.
- The 1825 Super T beams provide an efficient solution to the Resource Consent conditions to achieve the required clear span.
- The use of restraint couplers reduced the number of internal diaphragms and kept the unrestrained length of beam flanges and webs to within 8 m.
- Designing the beams for a range of concrete properties allows flexibility in choosing different concrete mixes by the pre-casting team.
- 1825 Super T beams have high torsional stiffness, and so the reduced torsion stiffness approach is used in the design, as the torsionless design approach permitted in the code NZS 3101 (2006) is likely to produce non-conservative results with respect to combined shear and torsion.
- There are a number of issues noted with the current NZS 3101 (2006) and these have been highlighted to the current NZS 3101 code committee to be addressed in new amendments.
The Alliance team for M2PP developed the mould for 1825 Super T beams, and established a state of the art pre-casting yard capable of producing the hundreds of Super T beams required for the M2PP project.

Production efficiency of Super T beams was a focal point and any opportunity to reduce the cycle time had been exploited at the outset with efficient and effective setting out of the yard and processes. The average time for manufacturing a beam was 2.8 days, at peak production the plant produced a beam per day.

The Alliance team carried out investigations and trials for the construction of complex end blocks, use of high slump concrete, hot-water curing and the establishment of a state of the art pre-casting yard.

The use of a jig for fixing the reinforcement cages, special steel templates for diaphragm's and bearing inserts, enabled a number challenges to be overcome during the casting of Super T beams.

This paper illustrates the opportunity offered in an Alliance contract to develop innovative approaches and deliver an acceptable balance between structural design, constructability, performance, risk and cost.

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