1. INTRODUCTION

Sydney Trains engaged Arcadis (formerly Hyder Consulting) to undertake an options study and subsequent detail design for the superstructure replacement of the railway bridge located in Southern Sydney at Guess Avenue, Wolli Creek 7.684km, Illawarra line.

The original single-span bridge comprised two (2) side-by-side separate transom-top riveted steel half-through girder superstructures, supported on brick abutments. Each half-through girder superstructure supported two (2) tracks and had a span (centre to centre of bearings) of 13.1 metres.

The full four-track wide brick abutments (in anticipation of future quadruplication) were constructed around 1915, however, only one (1) superstructure was installed then, being for the two (2) tracks that were existing at that time. This bridge was constructed as part of a new road connection through the railway embankment.

When quadruplication of the Sydenham to Rockdale section of the Illawarra line occurred in 1923, the second superstructure was installed on the existing extended abutments.

The vertical clearance prior to superstructure replacement was approximately 4.13 metres (signposted as 4.0 metres). As such, the original bridge had a substandard vertical clearance, below the preferred standard height of 4.6 metres, as specified in AS 5100.1 Bridge design for clearances of bridges above local roads.

Figure 1 shows Guess Avenue underbridge shortly before superstructure replacement work.

Figure 1 – Guess Avenue Underbridge, Wolli Creek (June 2015)
According to the existing bridge drawings, each brick abutment rests on a concrete footing 914mm (3 feet) thick, which is supported on fifty-six (56) timber piles, whereby each pile is noted as being 16 inches (406mm) in diameter.

The objectives of the project were as follows:

- Bridge superstructure replacement options to be either ballast top or direct rail fixation, in order to reduce maintenance costs and improve road vertical clearance.
- Design railway loading of 300LA (plus dynamic load allowance) to apply.
- All options shall consider re-use of the existing substructures as support for the replacement superstructures. Any strengthening work of the substructures shall also be investigated due to additional loading condition of the new superstructure.
- Preparation of a summary report containing all options studied in the investigation including concept General Arrangement drawings and recommendations for the selection of the preferred design option.
- Provision of professional services for the detailed design and documentation of the preferred bridge renewal option.

2. PRELIMINARY DESIGN – REPLACEMENT SUPERSTRUCTURES

The estimated depth from rail level to underside of main girders (commonly referred as construction depth) for the individual superstructures is as shown in Figure 2. This dimension gives an indication of the replacement superstructure type/s required to provide an increase in the vertical clearance above Guess Avenue.

Based on the above, it was obvious that a ballast-top replacement superstructure comprising standard Sydney Trains pretensioned girders was not suitable for this site, given one of the criteria was to increase the vertical clearance to the road below. Even if transverse post-tensioning was used to create the structural benefits of plate-type deck behaviour, for the anticipated span length of 13m, the overall proposed construction depth would be 1820mm (that is, 600mm (track & ballast) + 20mm (waterproof membrane & rubber ballast mat) + 1200mm (girder depth)). This far exceeds the approximate existing dimension from rail level to underside of through girder shown in Figure 2.

This lead to consideration of the filler beam form of construction for the replacement superstructures, together with direct rail fixation, thereby eliminating ballast.

Filler beam bridge decks are classified as a shallow deck system, having relatively high slenderness (span to depth), however, they possess high stiffness. As such, they are particularly suited to projects where construction depth (that is, rail level to underside of superstructure) is to be minimised.
This form of construction uses steel I sections embedded within a reinforced concrete slab unit. The concrete is cast to the level of the underside of the bottom flanges (or to the top of the bottom flanges) and covers the tops of the sections.

Some European references indicate slenderness (span to depth) ratios for filler beam railway bridges of between 15 for short spans (around 5m) and 25 for long spans (around 15m). This corresponds to a structure depth of approximately 570mm, based on a span length of 13.0 metres. The deck units at Guess Avenue underbridge are 630mm deep.

As railway bridge superstructures are required to be replaced in a limited period of time (typically, during a track possession period of 48 hours), a precast or pre-fabricated superstructure system was required.

The chosen superstructure replacement option comprised precast filler beam deck units, 1200mm, 1340mm and 2400mm wide, and match-cast to ensure uniform contact between adjacent deck units. They were to be placed side-by-side and interlocked using longitudinal shear-key joints. Finally, the deck units were to be transversely post-tensioned to form an orthotropic slab deck. This work was to be carried out during a track possession, following erection of the individual deck units on the precast concrete abutment headstocks. The entire deck was also to act as propping for the existing brick abutments.

3. EXISTING ABUTMENTS – VERTICAL LOAD-CARRYING CAPACITY

As the existing brick abutments were to be re-used, their theoretical vertical load-carrying capacity was checked.

Hand calculations were initially carried out, however, it became evident that more detailed analysis work, using computer modelling, was required.

Figure 3 shows the assumed subsurface profile and foundation system, derived from the geotechnical investigation carried out by Golder Associates.
The vertical load-carrying capacity, estimated settlement and stability of the existing abutments to satisfactorily support the new superstructures were assessed, utilising geotechnical information contained in the geotechnical investigation report prepared by Golder Associates.

Also, a test pit was excavated at the front of the abutment to confirm the presence of timber piles and then to inspect their condition and diameter.

The total mass of the original transom-top steel half-through girder superstructure spans was estimated as 112 tonnes each. The mass of the replacement filler beam deck unit superstructure spans (including rails, inner guardrails, rail plates and cantilever walkways) is 498 tonnes, over four (4) times heavier. With such significant additional permanent loading to be imposed on the existing abutments, it was essential to carry out a detailed check of the vertical load-carrying capacity of these abutments.

There were a number of assumptions that had to be made, as follows:

- Pile length.
- Pile diameter (based on a small representative field sample only).
- Pile integrity (based on a small representative field sample only).
- Total number of piles (based on existing bridge drawings only).

The results of the Plaxis3D modelling carried out by Arcadis indicated that the pile axial load is relatively uniformly distributed over all fifty-six (56) timber piles, and the pile cap in bearing is expected to share 10% to 20% of the total loading.

It was concluded that the existing pile foundation at each abutment would be suitable for the proposed bridge upgrading loads and that the existing abutments were considered satisfactory to re-use, without strengthening or underpinning being required.

A stability check was also carried out for the temporary case when the original superstructure is removed, leaving the brick abutment to act as a gravity retaining wall. It was found that to ensure an adequate factor of safety against overturning that the top 1 metre of backfill behind the existing abutments be removed prior to the removal of the existing superstructures, in order to reduce the overturning effect of the retained fill on the temporarily free-standing abutments.

4. STRUCTURAL DESIGN – REPLACEMENT SUPERSTRUCTURES

4.1 Features

To the author’s knowledge, this project was the first time a segmental (in the transverse direction) match cast filler beam deck unit, with direct rail fixation, has been used in New South Wales, if not Australia.

Deck units either comprise two (2) or four (4) concrete encased steel sections, depending on their width. Transverse tie bars (VSL CT Stressbar, Macalloy Bar, or equivalent) ensure effective shear key behaviour is achieved and prevents differential vertical displacement between adjacent deck units (refer Figures 4 and 5).

A filler beam deck unit superstructure was found to have an overall construction depth of 865mm (that is, 235mm (60kg rail + Alt.1 fastening system + 15mm thick epoxy mortar pad) + 630mm deck unit thickness).

The proposed vertical clearance from the underside of the filler beam superstructure to Guess Avenue was found to be around 4550mm, and with a track lift of 50mm, the preferred 4.6 metre vertical clearance was achieved.

Full-penetration butt welds are required by Sydney Trains at the flange/web connections of the built-up encased steel I sections. As such, commercially available Welded Column sections are not appropriate, as these have fillet welds on each side of the web.

The match-cast deck unit side faces were then ‘cemented’ together with a thin (approximately 0.5mm to 1mm) layer of epoxy bonder. Apart from compensating for minor imperfections in the combined surfaces, the epoxy provides a waterproof seal in the joints.
Initial grillage modelling assumed longitudinal ‘hinges’ along the shear keys between adjacent filler beam deck units, in order to determine the maximum longitudinal bending moment and deflection. This structural analysis model was subsequently modified to simulate the effect of transverse post-tensioning, by way of removing the ‘hinges’.

Live load deflection is a relatively complex determination, using the effective second moment of area ($I_{eff}$) and the short term modular ratio. Analysis showed instantaneous theoretical deflection due to 300LA railway loading (plus DLA) on both tracks to be just below that determined from Clause 8.9 of AS 5100.2, which stipulates the deflection of a railway bridge under traffic shall be no greater than $1/640$ of the span length (20mm in this case).

In order to comply with Clause 8.9 of AS 5100.2 that no sag should occur under permanent loads, the deck units were pre-cambered by 7mm.

Precast reinforced concrete abutment headstocks were installed during the track possession and fixed to the cut-down surfaces of the existing abutment brickwork. Also, the installation of ballast-top precast reinforced concrete approach slabs were included in the work.

The approximate mass of the 2400mm wide filler beam deck units, 13.6m long, was 67.7 tonnes (with Delkor rail plates attached), while the approximate mass of the smaller deck units was 34.7 tonnes.

The bearings comprised elastomeric strips, whereby the bearing loads are more uniformly distributed with this type of superstructure, compared with the concentrated loads from pot bearings of composite steel box deck units for example, as elastomeric strip bearings uniformly distribute superstructure loads to the substructure.
In relation to corrosion protection of the concrete embedded steel sections, two (2) alternatives are recommended: (a) Only the bottom flange (soffit, upper surface, root radii and edges) of the steel section requires protective paint coating, as shown in Figure 6 (from booklet titled Bridges with rolled sections, produced by AcelorMittal), or (b) Hot dip galvanising, followed by grit blasting of entire steelwork (to promote concrete to steel adhesion).

![Corrosion Protection System (Painting)](image)

**Figure 6 – Corrosion Protection System (Painting)**

### 4.2 Design Principles

AS 5100 Bridge design does not cover filler beam design theory. As such, for the design of the filler beam superstructure, the following codes were used:


There are a number of geometric criteria in relation to the filler beam form of construction, as outlined in Appendix 1 of this paper.

As previously noted, the superstructure was transversely post-tensioned using high-tensile tie bars (Macalloy Bar, 40mm diameter), to assist the structural interaction between adjacent deck units. There are fifteen (15) transverse tie bars, spaced at 850mm.

ACES bridge analysis software was used to determine the theoretical design actions along each line of longitudinal ‘girders’, whereby each ‘girder’ comprised an individual steel section and associated width of concrete encasement. Appendix 2 of this paper shows the determination of the theoretical bending moment capacity of an individual filler beam section, whereby the plastic moment capacity is used.

Initially, continuous hinges were modelled between adjacent deck units (shown as rectangles in Figure 7) to simulate the transverse load distribution mechanism to the longitudinal girders, whereby transverse load distribution between adjacent units is effected by transmission of load through the continuous shear keys by shear action only. As such, for this initial modelling, the superstructure behaved as an articulated plate, which by definition has zero transverse bending stiffness (EI) across the longitudinal shear key.

However, with the introduction of transverse post-tensioning across the superstructure, the initial articulated plate model was transformed into orthotropic plate structural behaviour, effected by the removal of the member end releases for transverse bending. This brought about a superior load distribution system, whereby the entire deck width is more effectively engaged to contribute to the load-carrying capacity.

The primary purpose of the post-tensioned transverse tie bars was to ensure the shear key connected deck units remain in compression, thereby preventing separation. As such, the post-tensioning arrangement (that is, bar spacing and bar force) was determined to be sufficient to generate a contact pressure between deck units to overcome the applied load effects (transverse bending moments).
Deflection typically governs the design (whereby deflection due to live load shall be ≤ span/640, in accordance with AS 5100. A span-to-depth ratio of 20.6 was achieved, which satisfied the deflection limit.

For deflection determination, the transformed section is required for I_{uncracked} and I_{cracked}. Deflection calculations use I_{effective} = 0.5 (I_{uncracked} + I_{cracked}).

It is worth noting that for reinforced concrete sections I_{effective} ≈ 0.55 I_{uncracked} but for filler beam sections I_{effective} ≈ 0.9 I_{uncracked}. This is due to steel being the dominant material in the cross-section.

The entire shear force between adjacent deck units was designed to be carried by the continuous 30mm protruding shear keys.

No shear studs/connectors were required, as composite action is theoretically achieved through steel/concrete bond across the relatively large interface surface between these two materials.

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**Figure 7 – ACES Structural Analysis Grillage Model (articulated plate model)**

As can be seen in Figure 8, the new superstructure has a reduced construction depth of 420mm compared with the original steel half-through girder superstructure.

**Figure 8 – Comparison of Superstructure Depths**
5. SUPERSTRUCTURE TRIAL ASSEMBLY

As railway bridges are required to be installed during a limited track possession/closure period (typically 48 hours), it is essential that any unforeseen installation issues are identified and eliminated prior to the actual on-site work.

To this end, a trial assembly of the superstructures, supported on its headstocks, was undertaken in a Sydney Trains yard at Lidcombe in Western Sydney several weeks before the actual installation at the bridge site, as shown in Figure 9.

The deck units interlocked precisely as per plan and all transverse tie rod holes perfectly lined up.

6. SUPERSTRUCTURE INSTALLATION

During two (2) consecutive weekend track possessions of approximately 48 hours each, one at the end of June 2015 and the other at the beginning of July 2015, the separate superstructures were installed.

Prior to the first track possession, a 750 tonne crawler crane was assembled adjacent to the Sydney abutment, beside the railway tracks.

Horizontal saw cutting of the abutment ledges to accommodate the proposed precast concrete headstocks was carried out on the Friday afternoon of each track possession. Temporary steel stitching plates were installed across the cut lines to prevent any differential movement/separation due to the action of rail traffic in the period leading up to the actual possession.

Following track occupation, slewing of the overhead wires and cutting of the rails, the existing steel superstructure was removed by crane and cut up into pieces near the site. The upper sections of the previously saw cut abutment ledges were removed and the precast reinforced concrete headstocks installed on a levelling bed of mortar (Figure 9). Vertical dowel bars were then inserted to provide shear connection between the new headstocks and trimmed down existing brick abutments.

Figures 10, 11, 12, 13 and 14 show various stages of the construction work.
Following installation of elastomeric bearing strips along the ledges of the new headstocks, the filler beam deck units were installed (Figure 12).
Following installation of all filler beam deck units, the entire deck was transversely stressed and the rails installed.
7. CONCLUSIONS

The upgrading of Guess Avenue underbridge with filler beam deck units superstructures represented an appropriate solution for this particular site.

It satisfied Sydney Trains’ project objectives, primarily being to increase the vertical clearance to the road below, re-use the existing substructures and to construct a bridge upgrading form of construction that represents reduced maintenance.

8. ACKNOWLEDGEMENT

The author would like to thank Sydney Trains for providing access to the site during the superstructure installation work and also supplying the time-lapse video of the construction work.
APPENDIX 1 – FILLER BEAM GEOMETRIC CRITERIA


6.3 Filler beam decks

6.3.1 Scope

(1) Clauses 6.3.1 to 6.3.5 are applicable to decks defined in 1.5.2.14. A typical cross-section of a filler beam deck with non-participating permanent formwork is shown in Figure 6.8. No application rules are given for fully encased beams.

NOTE: The National Annex may give a reference to rules for transverse filler beams

(2) Steel beams may be rolled sections, or welded sections with a uniform cross-section. For welded sections, both the width of the flanges and the depth of the web should be within the ranges that are available for rolled H- or I- sections.

(3) Spans may be simply supported or continuous. Supports may be square or skew.

![Figure 6.8: Typical cross-section of a filler beam deck](image)

Key:
- 1 non participating formwork

(4) Filler-beam decks should comply with the following:
- the steel beams are not curved in plan;
- the skew angle $\theta$ should not be greater than $30^\circ$ (the value $\theta = 0$ corresponding to a non-skew deck);
- the nominal depth $h$ of the steel beams complies with: $210 \, \text{mm} \leq h \leq 1100 \, \text{mm}$;
- the spacing $s_w$ of webs of the steel beams should not exceed the lesser of $h/3 = 600 \, \text{mm}$ and $750 \, \text{mm}$, where $h$ is the nominal depth of the steel beams in mm;
- the concrete cover $c_{st}$ above the steel beams satisfies the conditions:
  - $c_{st} \geq 70 \, \text{mm}$,
  - $c_{st} \leq 150 \, \text{mm}$,
  - $c_{st} \leq h/3$,
  - $c_{st} \leq x_{pi} - t_f$
  where $x_{pi}$ is the distance between the plastic neutral axis for sagging bending and the extreme fibre of the concrete in compression, and $t_f$ is the thickness of the steel flange;
- the concrete cover to the side of an encased steel flange is not less than 80 mm;
- the clear distance $s_f$ between the upper flanges of the steel beams is not less than 150 mm, so as to allow pouring and compaction of concrete;
- the soffit of the lower flange of the steel beams is not encased;
- a bottom layer of transverse reinforcement passes through the webs of the steel beams, and is anchored beyond the end steel beams, and at each end of each bar, so as to develop its yield strength in accordance with 8.4 of EN 1992-1-1: 2004; ribbed bars in accordance with EN 1992-1-1: 2004, 3.2.2 and Annex C are used; their diameter is not less than 16 mm and their spacing is not more than 300 mm;
- normal-density concrete is used;
- the surface of the steel beams should be descaled. The soffit, the upper surfaces and the edges of the lower flange of the steel beams should be protected against corrosion;
- for road and railway bridges the holes in the webs of the steel section should be drilled.
APPENDIX 2 – FILLER BEAM FLEXURAL DESIGN


The ultimate moment is the maximum moment, which a cross-section can resist before failure. This moment is obtained, when all the materials of the cross-section become plastic.

Calculation of $M_{Rd}$

Calculation of the position of the plastic neutral axis $x_G$:

For the equilibrium of the cross-section the force resulting from the tension stresses in the beam must be equal to the sum of the forces resulting from the compression stresses in the steel and concrete.

\[
F_{St} = F_{Bc} + F_{Sc}
\]

\[
F_{St} = \frac{f_y}{\gamma_a} \left[ bt + t_a(x_G - t) \right]
\]

\[
F_{Sc} = \frac{f_y}{\gamma_a} \left[ bt + t_a(h - t - x_G) \right]
\]

\[
F_{Bc} = \frac{\lambda f_{ck}}{1.5} \left[ B(H - x_G) - bt - t_a(h - t - x_G) \right]
\]
\[
\frac{f_y}{\gamma_a} (bt - t_a) + x_G \frac{f_y}{\gamma_a} t_a = \frac{\lambda f_{ck}}{1.5} [B(H-bt - t_a(h-t))] + x_G \frac{\lambda f_{ck}}{1.5} [t_a(B) + \frac{f_y}{\gamma_a} (bt + t_a(h-t))] - x_G \frac{f_y}{\gamma_a} t_a
\]

\[
x_G = \frac{\frac{\lambda f_{ck}}{1.5} [B(H-bt - t_a(h-t))] + \frac{f_y}{\gamma_a} t_a h}{\frac{\lambda f_{ck}}{1.5} (B-t_a) + 2 \frac{f_y}{\gamma_a} t_a}
\]

The resulting ultimate moment is the sum of the moments of these forces related to \(X_G\):

\[
M_{Rd} = F_{Sc} x_{FSc} + F_{Bc} x_{FBc} + F_{St} x_{FSSt}
\]

Calculation of \(x_{FSSt}\): distance between the resulting \(F_{St}\) to \(x_G\)

This is the position of the centre of gravity of the tension zone of the steel.

\[
x_{FSSt} = \frac{t_a (x_G-t)^2}{2} + b t \left( x_G - \frac{t}{2} \right)
\]

Calculation of \(x_{FSc}\): distance between the resulting \(F_{Sc}\) to \(x_G\)

This is the position of the centre of gravity of the compression zone of the steel.

\[
x_{FSc} = \frac{t_a (h-x_G-t)^2}{2} + b t \left( h - x_G - \frac{t}{2} \right)
\]

Calculation of \(x_{FBc}\): distance between the resulting \(F_{Bc}\) to \(x_G\)

This is the position of the centre of gravity of the compression zone of the concrete.

\[
x_{FBc} = \frac{B \left( H - h \right) \left( \frac{h}{2} - x_G + \frac{H}{2} \right) + t (B-b) \left( h - x_G - \frac{t}{2} \right) + \left( B - t_a \right) \left( h - x_G - t \right)^2}{B \left( H - x_G \right) - b t - t_a \left( h - x_G - t \right)}
\]