Load Rating and Condition Assessment of the Grand Avenue Overbridge

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ABSTRACT
Grand Avenue overbridge at Rosehill is an existing Sydney Trains asset constructed in 1980. The bridge design was based on a T44 truck loading to the NAASRA code (1976).

The bridge superstructure is a skewed, three span continuous deck, consisting of rectangular prestressed concrete beams on the end spans and standard Type 3 NAASRA pre-tensioned concrete girders at the central span. The construction of the bridge is unique in that the end spans are 10 metres long, significantly shorter than the 27 metre long middle span, causing a reversal of load action at the abutments under the bridge self-weight.

The deterioration of the expansion joints and concern about the pier slenderness prompted the request for the condition assessment, load rating and fatigue assessment. The assessment undertaken by Aurecon also explored any deficiency in the structure to carry heavy vehicles on this B-Double approved route.

The detailing of bridge components such as the movement joints, span lengths, elastomeric bearings and continuous girder connectivity for the full span had significant implications on the assessment and repair solutions.

This paper explores the key facets of the condition assessment, load rating and subsequent repair recommendations, which were unique to this structure due to unusual bridge detailing, original construction methodology and site specific constraints

INTRODUCTION

Project Background

Grand Avenue overbridge is located in Camellia, adjacent to Rosehill Gardens racecourse and local heavy industries. Grand Avenue is the main access road in and out of the industrial area and, as an approved B-double route, it is subjected to heavy vehicles on an ongoing basis.
Of the Carlingford and Sandown rail lines which pass underneath, only the Carlingford line is currently in service. The primary issues which Aurecon was engaged to investigate were failure of the expansion joints and the potential for the pier slenderness to contribute to joint failure due to excessive longitudinal movements.

Bridge Description

The three span skewed bridge has on overall deck length of 47.1m; each end span being 10m long and the central span of 27.1m. An overall deck width of 10.7m comprises two 3m wide lanes and a pedestrian walkway on the northern edge.

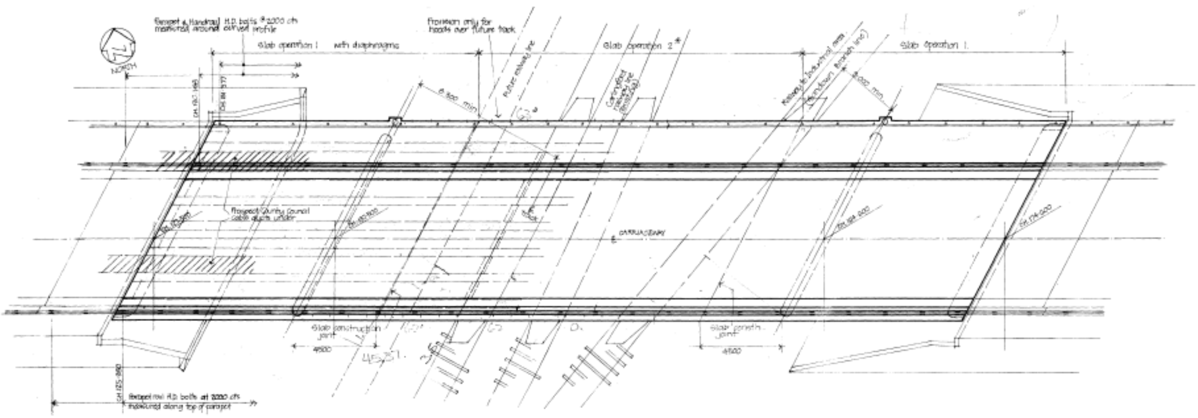

Figure 1 Plan

The deck is formed of seven pre-tensioned beams within each span. Each end span consists of a rectangular section girder of variable depth, increasing towards the pier. The central span is formed of standard Type 3 NAASRA pre-tensioned girder profiles which increase in depth toward the centre of span (Figure 2). Due to the skew effect, each girder profile varies across the deck. The deck is made continuous at the piers through a connection at the pier diaphragm. Elastomeric bearings support the deck at each abutment and centrally along the pier centreline.

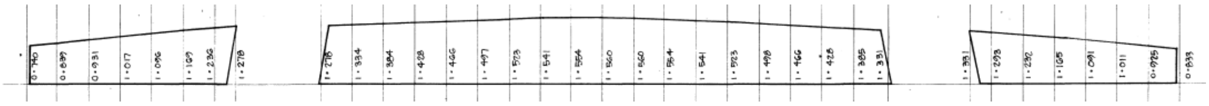

Figure 2 Indicative girder profile

Each 7m high pier is formed of a 300mm thick blade wall, with a thickened circular column terminating each end (Figure 3).

Figure 3 Pier cross section

Abutments are formed of a headstock supported on raked piles and surrounded by a reinforced batter slope. The deck joints at each abutment are formed of a steel angle nosing anchored to the deck slab and a rubber compression seal.

Noticeable Design Features

Approach slabs behind each abutment rest on the embankment fill, allowing for expected settlement. However, the slab connection is detailed with no movement joint at the deck surface, despite the slab being free to rotate at the dowelled joint on the abutment. This detailing is subject to degradation as the asphalt will crack over time.

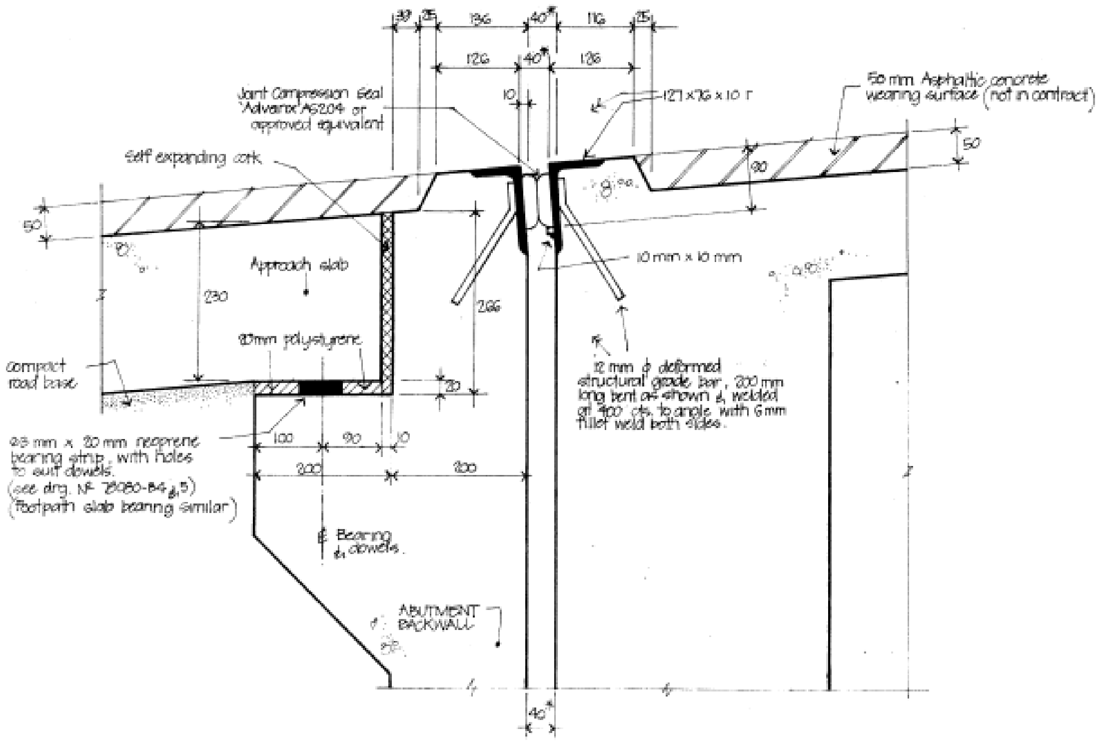

Figure 4 Deck joint section

Each end span of the continuous deck is 10m long, with a span ratio 0.37 of the 27m long middle span. To achieve an optimal balance in load distribution, the “typical” ratio of end to middle span falls between 0.7 to 0.85, but no less than 0.4. Thus the 0.37 ratio for this bridge falls outside this range.

This contributes to a reversal of load actions at the abutments under the bridge self-weight in the deck configuration. This will be further described in the structural analysis.

Girders are individually pre-tensioned and have been made continuous over the three spans through stitching of the reinforcement bars at the diaphragms above the piers, prior to placement of the deck slab. High shear and bending effects are concentrated at this location. The welded detail shown in Figure 5 is therefore an area subject to fatigue effects.

The construction method is facilitated by the inclined end faces on the girders, allowing the middle span to be placed onto the one bearing first before placement of the outer spans.

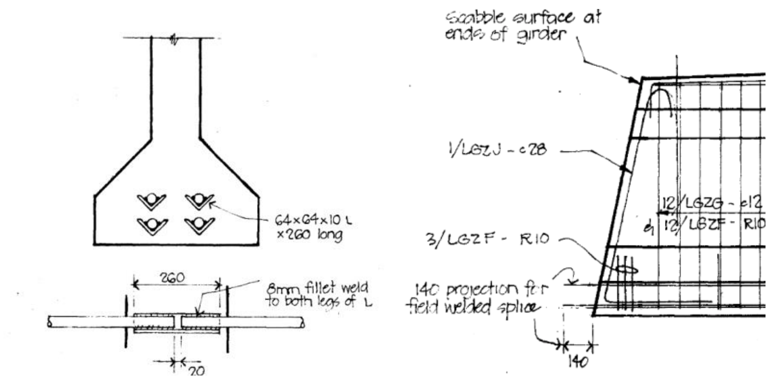

Figure 5 Girder reinforcement welding above piers at diaphragm

The bridge superstructure is fixed to the blade piers through two sets of three dowels (Figure 6). The deck is therefore fully constrained at the piers and longitudinal movements are provided for at the abutments only. The flexibility of the blade piers gives adequate tolerance for the middle span movements.

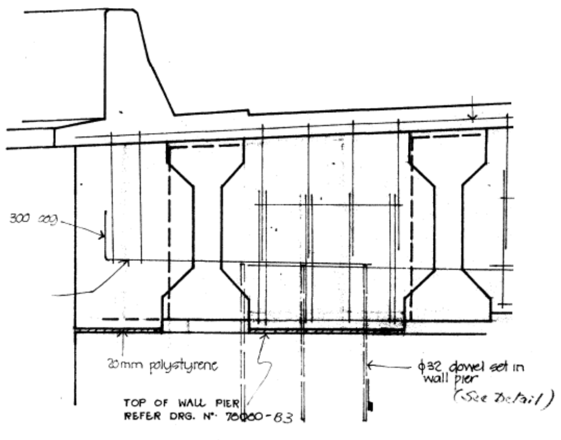

Figure 6 Dowel connection to pier and diaphragm restricting horizontal movement

ASSESSMENT

Condition Assessment

A condition assessment confirmed that the main defect was deterioration of the joint angles and rubber seal with sections of steel missing; in addition, the asphaltic concrete was degrading in the local area and notably across the approach slab where no provision for movement has been made. These defects are due to the repeated high impact loads from a high proportion of heavy vehicles.

Minor bulging was observed on a few pier bearings. All abutment bearings were in good condition.

Defects were observed on the deck soffit, which appeared to be cracks on first sight. With a detailed inspection using an elevated platform, these defects were found to be due to the permanent formwork cracking during the original concrete pour and did not affect the structural integrity of the bridge. Overall the bridge was in good condition with no defects noted on the piers.

Figure 7 Deck joint failure **Figure 8 Deck soffit construction defect**

Analysis

The bridge deck was initially modelled as a skewed grillage. A unique characteristic of the deck behaviour was the distribution of load due to the staged construction method coupled with the unusual range span ratio.

Due to the 0.37 span ratio, there is uplift at the abutments and additional shear and negative bending under the self-weight of the bridge deck and live loads at the piers. This would not have occurred during the placement of the girders at construction stage, but after the three spans were made continuous.

A second stage analysis was therefore undertaken to determine the reactions allowing for the construction staging as this has a significant impact on the bending and shear reactions within the girders.

The initial placement of girders was on middle span, allowing the girders to be simply supported under their own weight. The end girders were then aligned with the scabbled end faces of the middle span.

The end span deck and pier diaphragm concrete was poured on top of the simply supported girders that rest on the piers and abutments. Thus all girders were subjected to sagging moments at this stage. Continuity of the span was then achieved once the concrete cured on the end spans. The effects from the central deck additional load created hogging effects to the continuous girders at the piers.

To capture the change in load distribution which results from being a simply supported to continuous span, the final reactions of the imposed dead load were calculated in two separate stages: the self-weight of the girders and outer deck concrete as simply supported and once the deck cured, all additional superimposed dead loads (including central deck pour) were applied to the continuous structure.

This resulted in lower reactions at the abutment bearings and higher loads being applied to the piers, under the full self-weight of the structure and vehicle loading. This correlated well with the visible signs of overstressed bearings with the slight bulging noted at the piers.

This staged analysis is indicatively shown in Figure 9 and Figure 10.

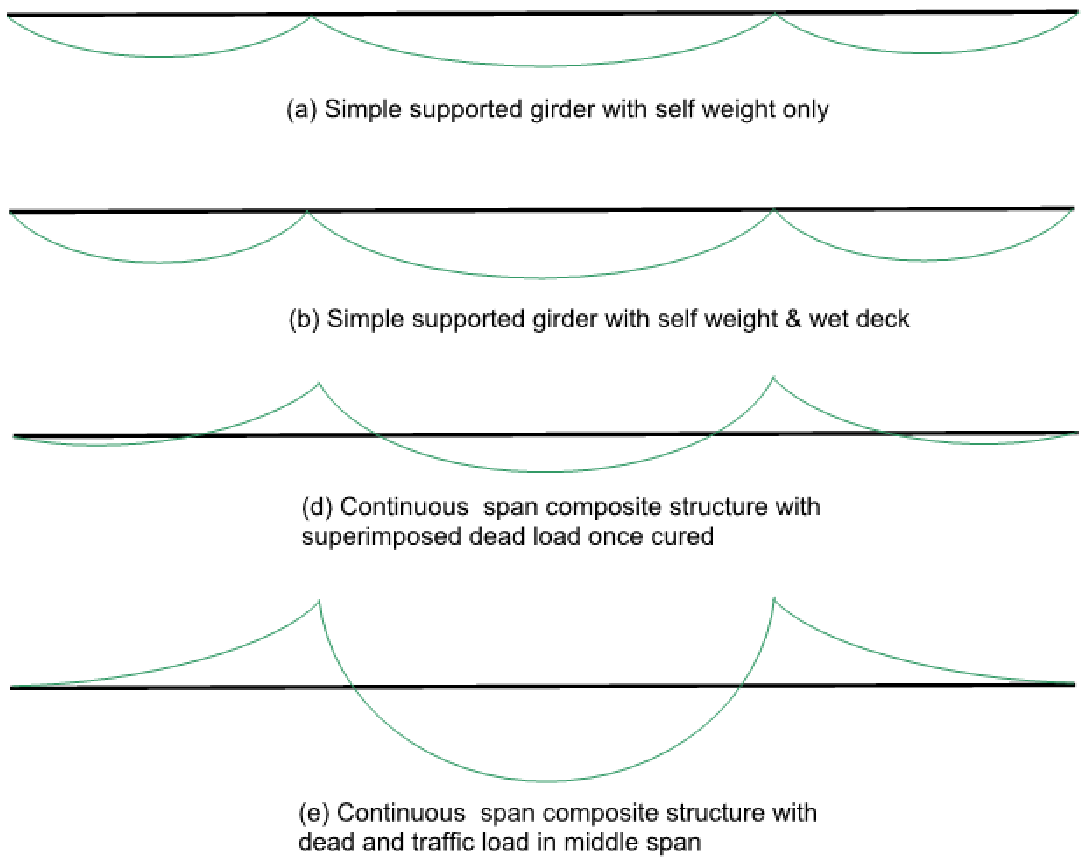

Figure 9 Indicative bending moment diagram at different stages

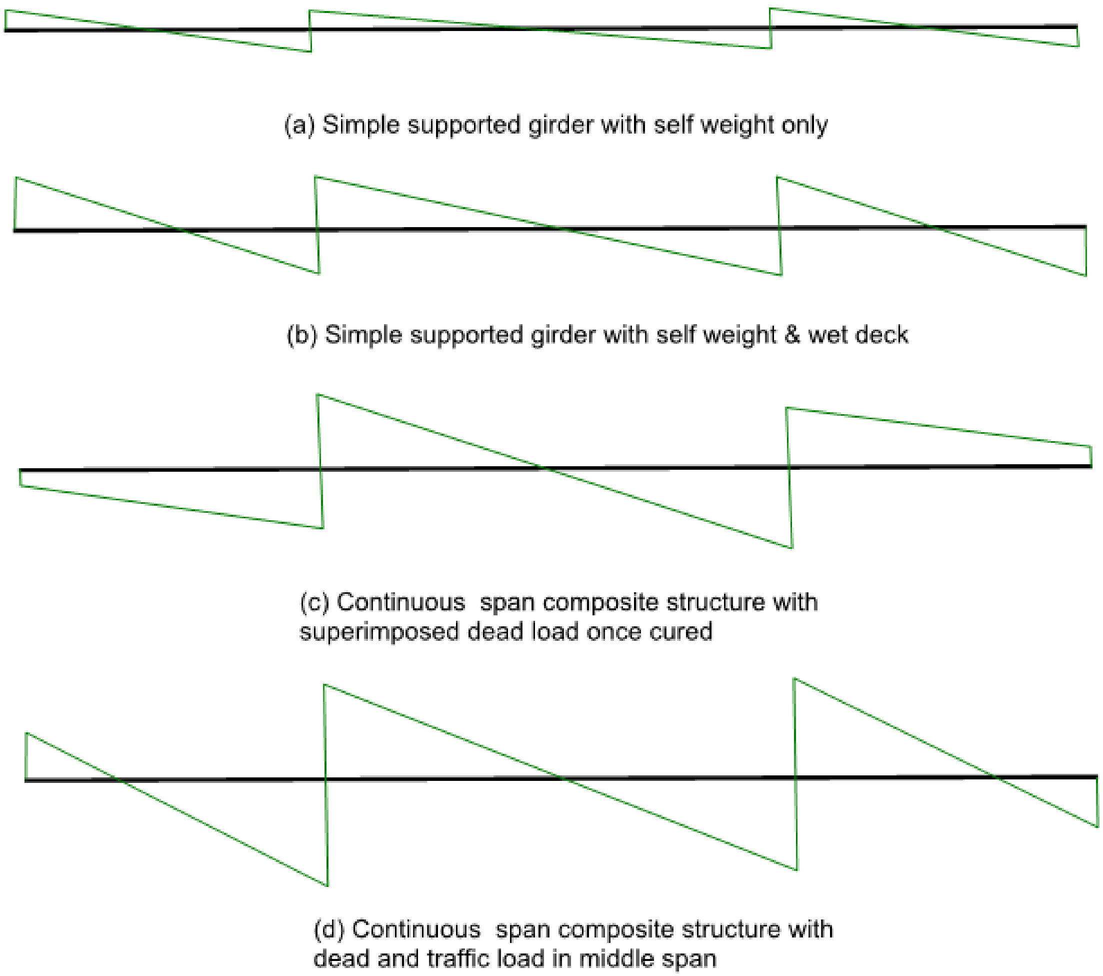

Figure 10 Indicative shear force diagram at different stages

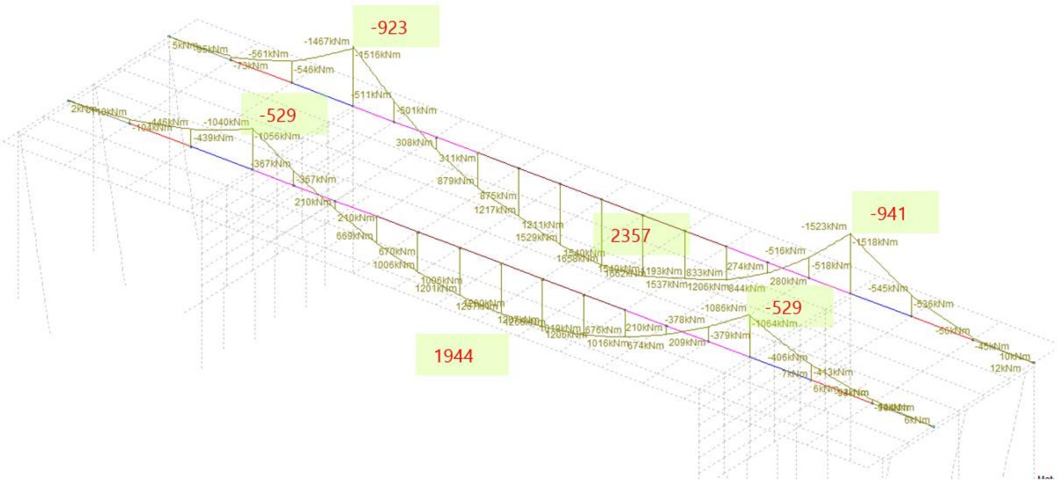

Figure 11 Bending moments under dead load as a continuous span (adjusted bending in red)

A simplified 3D model of the bridge determined the expected bridge movements under braking loads and thermal effects. Shrinkage and creep effects were calculated separately and assumed to be stabilised after over 30 years since construction.

The movements of the deck are relatively small (16mm at abutments) and there was no evidence of excessive movements causing cracking at the abutment wall or in the piers.

REPAIRS

The remedial design proposed a standard RMS small movement joint retrofitted to the approach slabs and removal of existing angle and compression slab for the Granor XJS joint, allowing for ease of construction on one lane at a time whilst maintaining access to the industrial estates at all times.

CONCLUSION

The deck joint defects were observed on site and correlated well with the expected degradation, due to repeated high load impact and the lack of a joint detail at approach slabs. Overstressing of the pier bearings is an expected effect of disproportionate span lengths and was determined through the staged analysis of this structure for the particular construction method adopted for the deck.
Span ratios and continuity of spans can adversely affect bearings and supporting structures. Today, critical design thinking and adherence to standard practice and current guidelines can usually avoid the design flaws exemplified by this bridge.

REFERENCES

- Parramatta City Council Grand Avenue Bridge Project Approach Roadworks drawings (dated March 1980)

ACKNOWLEDGEMENTS
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AUTHOR BIOGRAPHIES

Marcia Prelog is an Associate within Aurecon who has extensive experience in the inspection and assessment of bridge structures in Australia and overseas. Her 15 years in the Bridges discipline encompasses design in steel, concrete, composites and timber.

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