Load Testing of a 1920's Cast-in-Place Reinforced Concrete Tee Beam Bridge

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ABSTRACT

The Arden Street Bridge over the Moonee Ponds Creek is a seven-span cast-in-place reinforced concrete tee beam bridge constructed in 1923. Originally designed for a 16-ton roller, the bridge was strengthened in 2004 to accommodate semi-trailers and B-doubles operating at Higher Mass Limits.

pitt&sherry was initially engaged to undertake desktop load rating of the bridge to see if it could be used during the transport of various Heavy Load Platform configurations, including platforms carrying 180t and 130t transformers, as part of the planned West Melbourne Terminal Station upgrade.

Following the desktop exercise physical load testing of the bridge was carried out. The test regime included performance load testing using two semi-trailers loaded up to 54.5t each (Stage 1) followed by proof load testing of a representative tee beam using a hydraulic jack and loading frame (Stage 2).

1 INTRODUCTION

1.1 Bridge Description

The Arden Street Bridge is a seven-span cast-in-place reinforced concrete tee beam bridge that carries road traffic over the Moonee Ponds Creek in North Melbourne. The bridge was opened in 1923, taking the Reinforced Concrete & Monier Pipe Co. eight months to construct at a cost of £4,825. The construction authority and current asset owner is the City of Melbourne.

The total length and width of the structure are approximately 52.1m and 11.3m respectively. Span lengths are 7.3m with the exception of a 7.6m centre span. The traffic width between barriers is 7.6m and footpaths are provided on both sides of the deck.

The original design load was a 16-ton roller with 30% impact allowance. The superstructure consists of a 190mm thick reinforced concrete slab cast monolithic with 356mm wide by 762mm deep reinforced concrete continuous beams spaced at 1.9m centres. The beams are directly supported at abutments and piers by 356mm square reinforced concrete columns built upon 356mm square precast reinforced concrete piles driven into stiff clay.

Load rating of the bridge circa 2003 found that the structure was unable to accommodate semi-trailers and B-doubles operating at Higher Mass Limits (HML) due to insufficient beam capacity in flexure and both vertical and longitudinal shear in the regions close to the columns. A strengthening design was subsequently prepared by **pitt&sherry** and implemented by Council in 2004.

Strengthening involved installation of folded steel plates to the soffit of the deck and the side of the beams. To ensure structural continuity the folded plates were epoxy bonded to the concrete substrate and fixed to the deck and beams with chemical anchors. The combination of the plate and the anchors provided increased capacity for both flexure and longitudinal shear over the supports.

Increasing the shear capacity of the beams in the vicinity of the columns was achieved by application of layers of carbon fibre. As it was not possible to fully wrap the carbon fibre around the tee beam section, the folded steel plates, used for beam flexural strengthening, were used to provide anchorage of the carbon fibre at the deck/beam interface.

1.2 Heavy Load Review

In 2011 **pitt&sherry** carried out a heavy load review on behalf of SP AusNet to determine if the Arden Street Bridge could be used for the transportation of 180 tonne and 130 tonne transformers on

various platform/trailer configurations. The transformer movements would be necessary for the planned upgrade of the West Melbourne Terminal Station.

The heavy load review included a comparison of the design action effects induced by HML vehicles and the proposed Heavy Load Platform (HLP) transformer movements. HML loading was found to be more critical for the superstructure however the loading on columns and piles was more severe under HLP load cases. The capacities of the substructure elements were then determined for comparison with the HLP design actions.

The strength of the columns was calculated in accordance with the relevant provisions of AS5100 (2004) and found to be satisfactory for the proposed HLP configurations. The geotechnical capacity of the piles was estimated using a range of values for the undrained shear strength of stiff clay, based on qualitative soil profile information shown on the available design drawings. The pile capacity was found to be adequate only if the soil strength parameters were at the high end of the range.

A site geotechnical investigation was recommended to obtain quantitative data on the stiff clay present at the site. The investigation was carried out and included two boreholes with soil logging, Standard Penetration Tests and laboratory analysis. The results were favorable and provided confidence that the theoretical load carrying capacity of the structure was adequate for the proposed HLP loading.

SP AusNet and Council were advised of the outcome of the heavy load review, including restrictions on the speed of travel and lateral positioning for HLP movements over the bridge, should Council give approval. Potential risks due to latent conditions were explained to the asset owner.

1.3 As-Built Details

A visual inspection of the bridge was undertaken by **pitt&sherry** bridge engineers during the heavy load review. The inspection identified two missing anchors from one strengthening angle in an end span. Replacement of the missing anchors was recommended, and it was subsequently found that the embedment depth of the missing anchors was significantly less than had been specified for the 2004 strengthening works.

The observation raised concern over the number of anchors that may not have been installed with 130mm embedment depth as specified on the strengthening design drawings. A survey of anchor lengths was undertaken using ultrasonic testing. Almost 90% of the 2,625 anchors installed as part of past strengthening works were tested of which approximately 150 vertical anchors and 25 horizontal anchors had less than half of the specified embedment depth.

The survey identified three areas of the bridge where almost half of the vertical anchors securing the strengthening angles to the deck soffit had embedment depths well below the specified value, with some anchors having as little as 10mm embedment. The findings brought the effectiveness of the past strengthening works into question.

During the review of As-Built details it became apparent that the anchors were installed through slotted holes in the strengthening angles. Such holes were not specified on the strengthening design documentation and this added further concern due to the potential for slip at the deck/beam interface.

1.4 Rectification Works

In order to evaluate the impact of the As-Built anchor embedment depths **pitt&sherry** reassessed the past strengthening design, focusing on the vertical and longitudinal shear capacities. The vertical shear capacity was assessed using ACI 440.2R-08 Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures (American Concrete Institute, 2008). This publication, not available at the time of strengthening design, provides guidance on the efficiency of various FRP wrapping schemes. The strengthening design was found to be adequate for the HML design loading considering a 3-sided U-wrap scheme, and therefore the strengthening angles were not necessary for anchorage of the FRP.

The longitudinal shear force acting at the deck/beam interface was assessed and the minimum number of vertical anchors required to achieve sufficient capacity was calculated. It was found that at least 60% of the vertical anchors, with adequate embedment depth, were required on each strengthening angle.

In order to address deficiencies for longitudinal shear **pitt&sherry** recommended that a certain number of vertical anchors be reinstalled with the correct embedment depth, and all slotted holes be filled with a high-strength non-shrink grout and then covered with a steel plate washer. Council accepted the recommendations and engaged a contractor to implement the rectification works.

2 LOAD TESTING

During discussions with the asset owner concerning the effectiveness of As-Built strengthening details and other potential risks due to latent conditions Council requested a proposal from **pitt&sherry** to undertake physical load testing of the bridge. A test methodology was subsequently developed for performance load testing of the bridge (Stage 1) and proof load testing of one representative beam (Stage 2).

2.1 Objectives

The aim of Stage 1 was to subject the bridge superstructure and substructure to serviceability limit state load effects comparable in magnitude to the effects of the proposed HLP movements. Another objective was to validate the computer model of the bridge using Stage 1 test results prior to confirming test loads to be used during Stage 2.

The objective of the Stage 2 static proof load testing was to progressively load one representative tee beam to a level approaching its design bending capacity at midspan (Stage 2 _ Test 1) and design shear capacity near an internal support (Stage 2 _ Test 2), without inducing non-linear behavior in the structure.

2.2 Methodology

The performance load testing involved progressively increasing the live load travelling over the bridge from one 45.5t semi-trailer to two 54.5t semi-trailers. Various combinations of vehicle lateral positioning and speed (10km/h and 30km/h) were used. Modification of the dynamic load allowance based on testing was not an objective.

The results of Stage 1 testing were compared with analysis results obtained from a complete frame model of the bridge developed using the program Spacegass. The results of a linear static analysis compared well with the beam deflections measured during Stage 1 testing. The model was then used to determine target proof loads of 425kN and 480kN for bending and shear respectively.

In order to mitigate the risk of causing major damage to the structure during proof load testing a methodology was devised involving application of a concentrated load to the bridge deck by reacting off a loading frame positioned on the bridge. The frame was placed over the two westernmost spans and supported directly above the three southernmost columns at the abutment and piers. Counterweights were loaded onto the frame.

The loading frame included diaphragms located above the midspan position of the end span and at 1.4m offset from the westernmost pier centerline (also located above the end span). These diaphragms were used to apply a reaction force from a 50t hydraulic jack to the bridge deck directly over Beam 2 (from south). This beam was selected for the proof load test as there were no utility services connected to it (unlike other beams) and there was relatively good access in the event that it was damaged and needed repair.

2.3 Instrumentation

Sensors used during physical load testing of the bridge included strain gauges, linear potentiometers and slip measuring load cells (Figure 5). Two strain gauges were applied to each of the five columns

at the westernmost pier. Linear potentiometers were set up under each beam in the end span at the midspan position. Eight slip measuring load cells were installed on Beam 2, with four either side of the westernmost pier.

The data was captured using a data logger scanning all of the sensors simultaneously. For Stage 1 testing using moving vehicles the sample rate was 25Hz and during Stage 2 testing a sample rate of 1Hz was adopted. Deflection results were displayed on a laptop computer and monitored in real time during the proof load testing.

2.4 Results

2.4.1 Performance Load Testing (Stage 1)

Performance load testing was undertaken on 9 April 2017 following completion of rectification works on the strengthening angles that were installed on the bridge previously. Two test vehicles loaded to 45.5t and then 54.5t were used during the load testing (Figure 5).

During and at the completion of the testing, no noticeable signs of structural distress were sighted or heard. Upon completion of the test, the field data collected were processed and reviewed. The field results showed no noticeable slip between the strengthening angles and anchor bolts.

A maximum deflection of 1.48mm was recorded for the central beam (Beam 3) under the passage of two 54.5t semi-trailers (Figure 1). The magnitude of the deflection and distribution of live load compared well with the Spacegass frame model.

2.4.2 Proof Load Testing (Stage 2)

Proof load testing was undertaken on 7 May 2017. The loading frame was placed into position and counterweight blocks of known mass were then placed uniformly onto the loading frame. The hydraulic jack was placed under the loading frame to react against the counterweights at nominated locations directly over Beam 2 (Figure 5).

At the nominated locations, the load in the hydraulic jack was gradually increased until the target proof loads for either bending moment or shear design action had been induced in Beam 2. For bending the applied load from the jack reached a maximum of 44t and for shear a maximum of 48t was reached.

For the bending and shear tests the load in the jack was twice steadily increased up to a point below the target level and then reduced to confirm linear behavior before finally achieving loads close to the targets. During and at the completion of the testing, no noticeable signs of structural distress were sighted or heard.

A deflection of 2.38mm was recorded for Beam 2 at midspan during application of the maximum induced bending moment and the data shows a linear relationship between applied load and measured deflections (Figure 2).

A maximum concrete strain of approximately 100μm was measured in Column 2 during application of the peak shear test load and the data shows a linear relationship between applied load and measured strain (Figure 3).

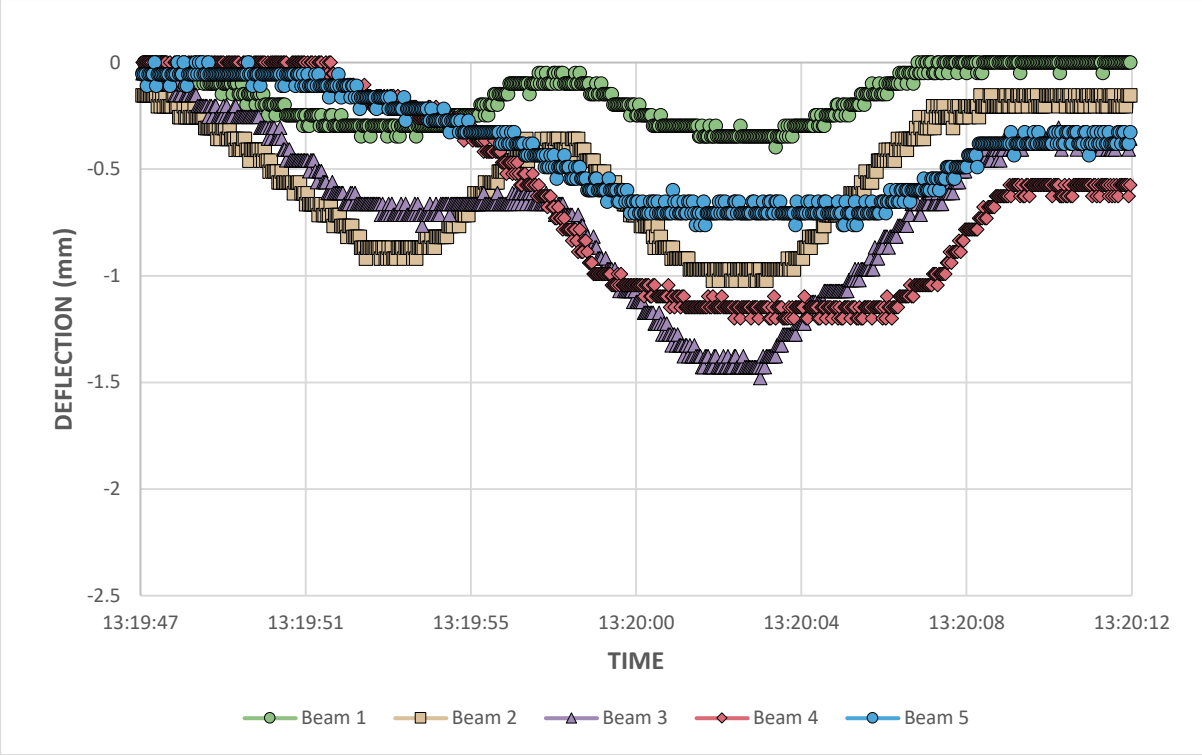

Figure 1 - Beam deflection due to passage of two 54.5t trucks

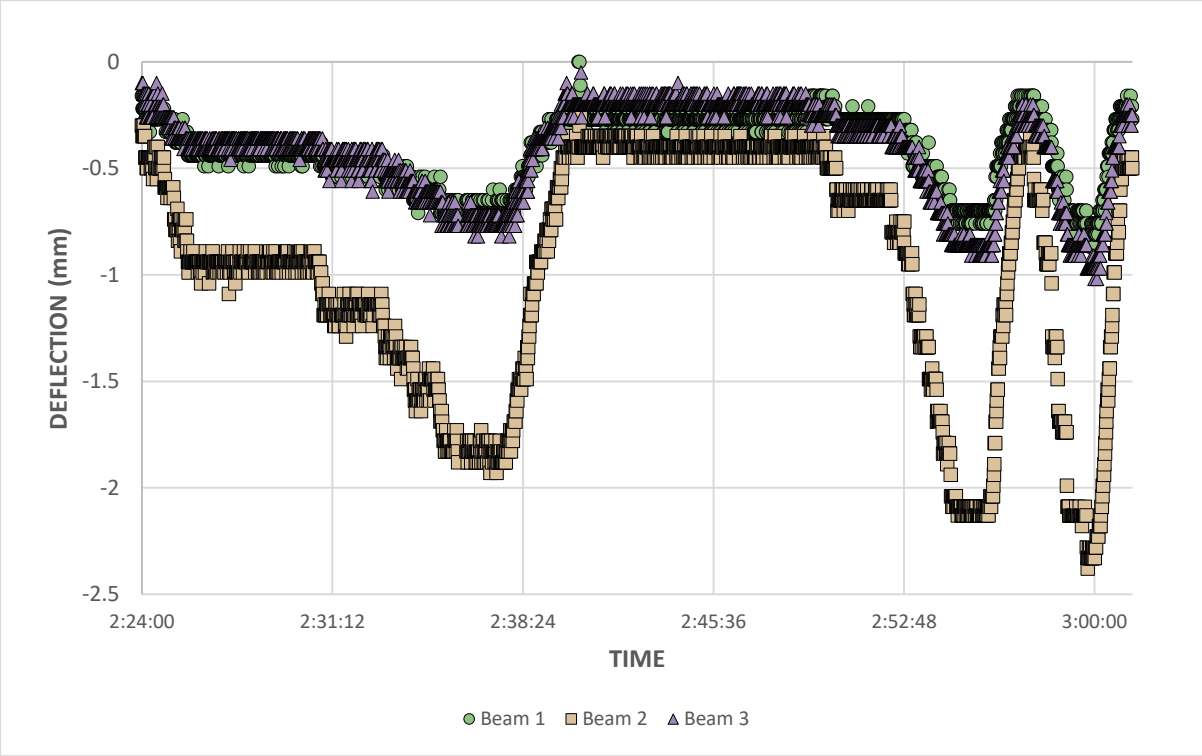

Figure 2 - Beam deflection due to concentrated load on Beam 2 at midspan

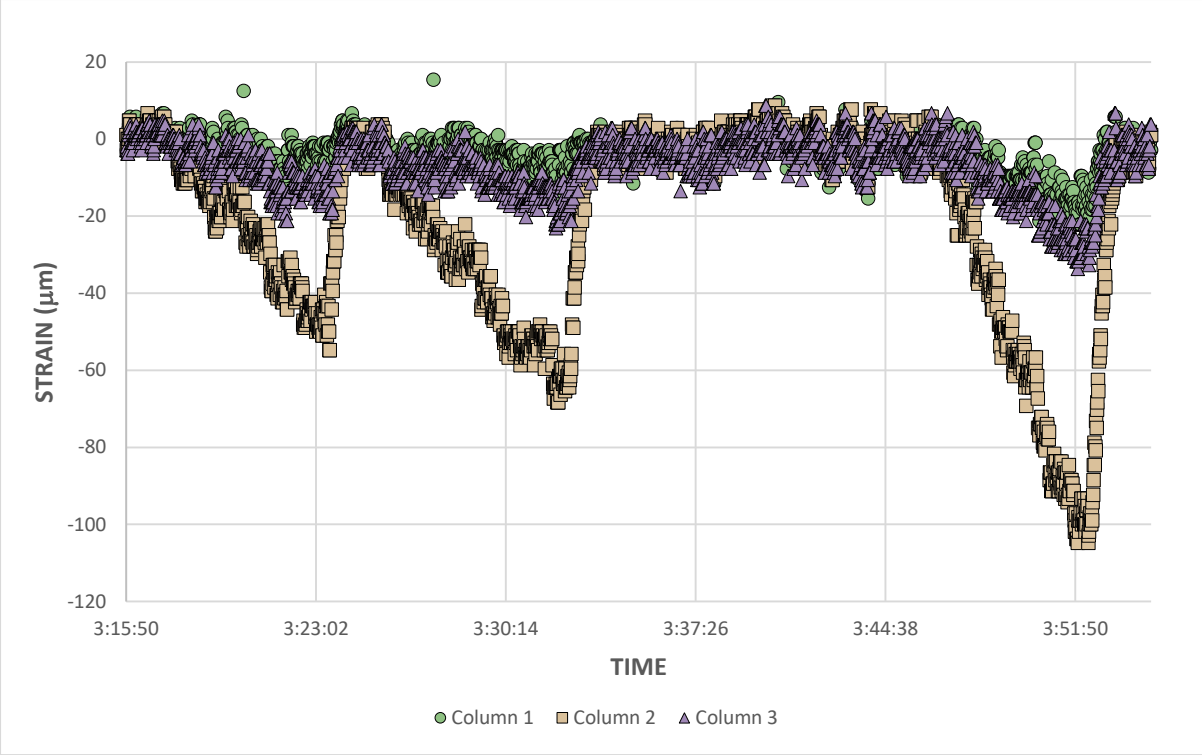

Figure 3 – Column strain due to concentrated load on Beam 2 near pier

Strain measurements in the slip measuring load cells did not indicate any significant relative movement between the anchors and strengthening angles (Figure 4). A strain of 40µm corresponds to a movement of approximately 0.0016mm (gauge length of 40mm). The range of the strain measured was very low and considered to be noise.

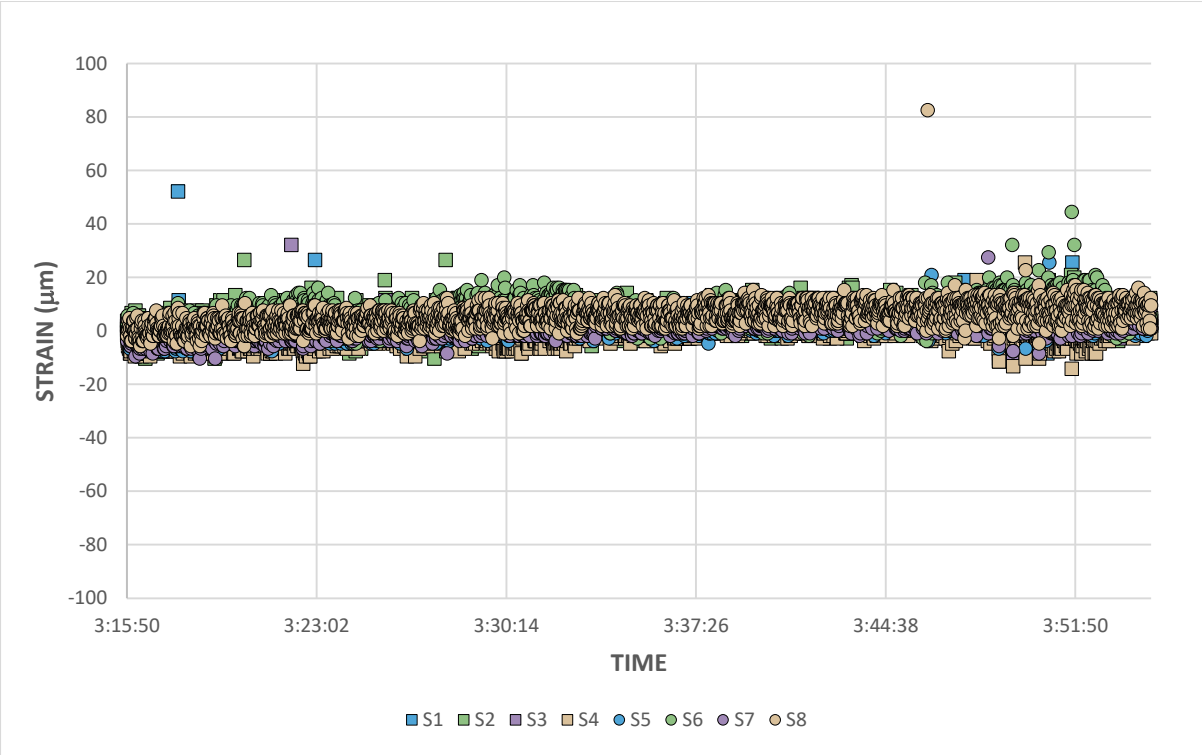

Figure 4 – Slip gauge strain due to concentrated load on Beam 2 near pier

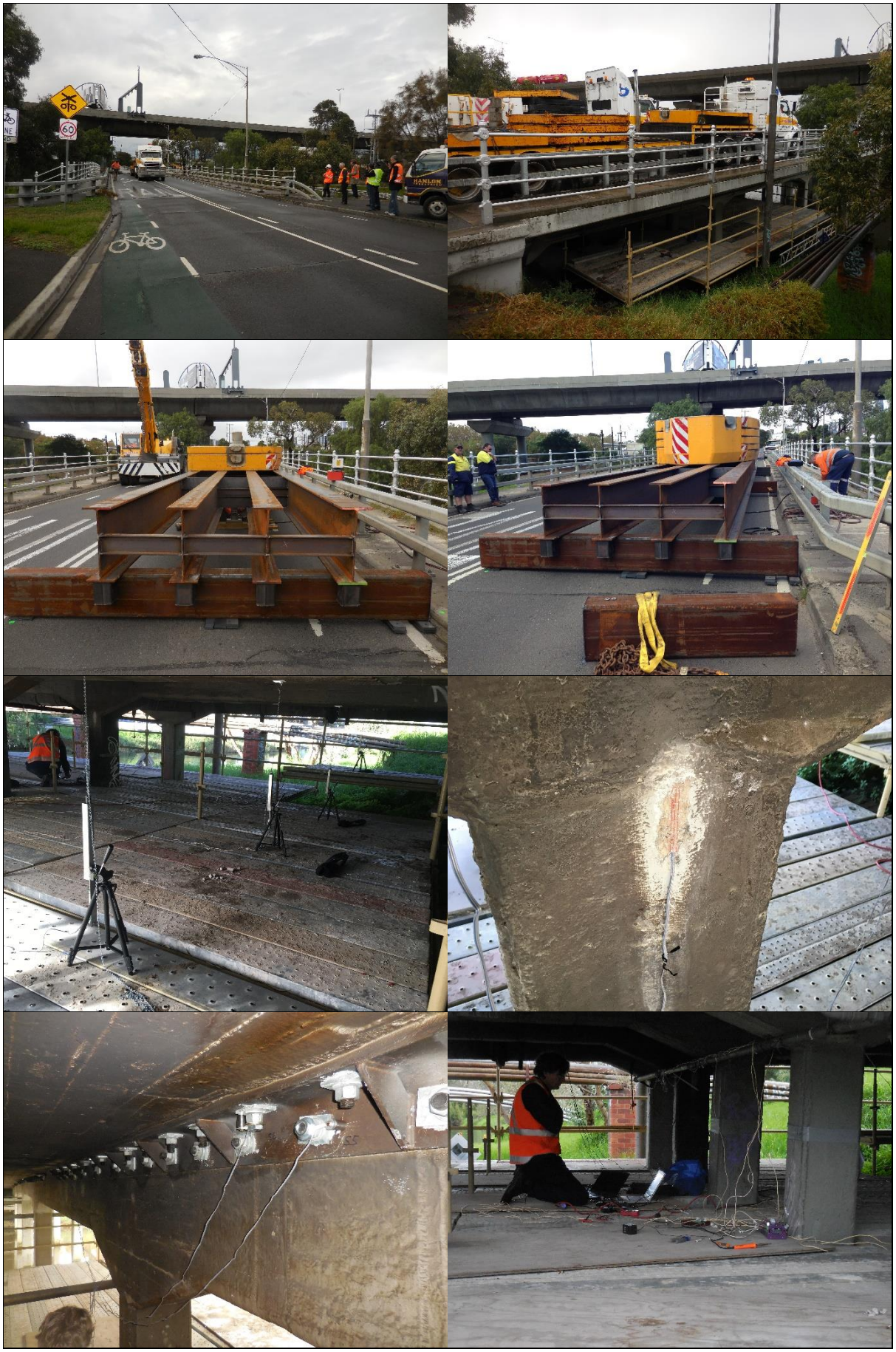

Figure 5 - Arden Street Bridge Load Testing

3 CONCLUSION

The Arden Street Bridge project highlights the importance of understanding critical As-Built structural details in order to evaluate the load carrying capacity of a structure. It also shows that, during construction, seemingly minor departures from design drawing details can have a significant adverse impact in terms of achieving the design intent.

Without complete As-Built records, as is often the case with older structures, there is always a risk that actual bridge details differ from those shown on original design drawings. In many situations it is not practical to confirm all details throughout a structure, such as reinforcement arrangement, pile lengths and condition of buried elements.

Performance and proof load testing can be undertaken to demonstrate the in-service performance and behavior of the structure. For the Arden Street Bridge performance load testing was carried out to ensure the bridge superstructure and substructure would perform satisfactorily under Heavy Load Platform movements critical to the upgrade of the West Melbourne Terminal Station.

Physical load testing of bridges requires detailed planning and coordination. It is essential that the objectives, methodology, roles, responsibilities and risks are clearly communicated and understood by all parties involved.

4 ACKNOWLEDGEMENTS

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5 AUTHOR BIOGRAPHY

Chris Morton is a Principal Bridge Engineer with Pitt & Sherry with 15 years' experience covering a broad range of bridge engineering activities including inspections, load rating, rehabilitation and strengthening design, concept and detailed design, feasibility studies, writing reports and specifications, cost estimates and hold point inspections.

Key achievements include undertaking detailed strengthening design for two heritage listed bridges.

Chris is currently responsible for managing a range of small to medium scale bridge projects for various asset owners including local government, road and rail authorities.