Faculty of Heath, Engineering and Sciences

School of Civil Engineering and Surveying

Investigating design and construction issues for precast concrete bridge over Bookookoorara Creek

A dissertation submitted by:

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in fulfilment of the requirements of

ENG4111 and ENG4112 Research project

towards the degree of

Bachelor of Engineering (Honours) (Civil Engineering)

Submitted October 2016

Abstract

NSW local government is faced with an aging timber bridge stock and resultant increase in maintenance burden. When considering replacement of there are a number of precast concrete proprietary systems which are typically investigated including the Holcim HumeDeck, Rocla M-Lock, Doolan Deck and plank bridge. RMS is developing a new type of modular bridge called Country Bridge Solutions which consists of precast headstocks, sill beams and wing walls in addition to precast prestressed deck modules. Constructability and safety in design are key considerations when investigating these options in the development stage of a bridge replacement. Time, quality, safety and cost are all important criterions in such an evaluation, as optimisation of these variables will inevitably lead to an overall improvement in project outcomes.

This dissertation firstly documents the construction of the bridge over Bookookoorara Creek in Tenterfield Shire Council as the pilot bridge under the Country Bridge Solutions system. The construction identified twenty areas in which the design or methodology could be altered to create better outcomes with regard to constructability or safety in design. Concepts were then developed for each of these issues, with selected concepts evaluated using weighted time, quality, cost and safety criteria to determine a recommended concept that may be progressed to detailed design by others.

Overall, this project has contributed to the engineering body of knowledge by documenting the construction of the pilot bridge for the benefit of future construction teams. The identified areas and concepts are presented to assist in the development of the Country Bridge Solutions system which is ultimately aimed at providing an efficient and effective bridge replacement option on low volume roads.

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Certification

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I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Acknowledgements

I would firstly like to thank my project supervisors, Dr Weena Lokuge (USQ) and Mr Peter Young (RMS) for their support through this project. The time that they have taken and support they have given during course of this project has been invaluable.

I would also like to that the Country Bridge Solutions project team, both management (Greg Evans, David Andrews, Peter Young, Peter Mahar (Bridge Knowledge), Paul Ensby, Stephen Bell (Tenterfield Shire Council) and RMS Bridge Engineering), construction (David Jones, Stephen Pereira, Jon Chevalley, Anthony O'Shannessy, Neil Griffin, Wayne Hamel, Hamilton Brennan, Taigan Heath and contractors) and support (Sharlene Martin and Belinda Bowman) for their time, expertise and input during the bridge construction process. It goes without saying that without a successful bridge construction this project would not exist. I would also express gratitude to RMS in general for allowing me to use the trial bridge for the purposes of this project.

I would also like to thank my family and friends for the support they have shown over this protracted period of study. My wife Magentah, who has understood the difficulties and time associated with this task, my parents for reminding me that there is a light at the end of this study tunnel and my siblings and friends for the constant support mixed with light-hearted banter.

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1 Introduction

The following report presents the findings of an undergraduate research project titled *"Investigating design and construction issues for a precast concrete bridge over Bookookoorara Creek"*. The subject bridge for this report was the pilot construction under the NSW Roads and Maritime Services (RMS) *Country Bridge Solutions* (CBS) program. The purpose of the report is to document the construction of the pilot bridge and identify areas of refinement from a constructability perspective before proposing and evaluating concepts to make progress towards improving the CBS system to improve the ease and safety of future bridge construction projects.

The purpose of this introductory chapter is to introduce the context of the project and define the project objectives.

1.1 Timber bridges in NSW

Local Government is responsible for the management of about 27,000 timber bridges within Australia (Crews et al 2004) with a combined value of $1.5 billion, of which most are at least 80 years old (Balendra et al, 2009). A report published by the Australian Local Government Association (ALGA, 2013) states that 65% of local government controlled bridges are classified as being in a poor to very poor state, while a report published by the Institute of Public Works Engineers (IPWEA, 2012) provides a figure of 30% being in poor condition and a further 49% being in fair condition. Regardless of the figure being relied upon, it is clear that there is a substantial amount of work needed to be done on these bridges in order for the road network to remain serviceable.

1.2 Who are the Roads and Maritime Services?

The Roads and Maritime Services (RMS) is a NSW government authority established on 1 November 2011 by a merger of the former Roads and Traffic Authority of NSW (RTA) and the former Maritime Authority of NSW (NSW Maritime) (RMS, 2012). RMS is an agency under the NSW Transport Cluster, and is primarily responsible for enabling safe and efficient journeys by; managing the road network and optimising travel times, providing capacity and maintenance solutions for all road and maritime users, educating and licensing drivers and vessel operators and registering and inspecting vehicles and vessels and improving road and maritime safety (RMS, 2015). The vision of RMS as an organisation is to be a leader in the management and delivery of safe, efficient and high quality services and infrastructure to NSW with a focus on the customer. RMS places the customer at the centre of everything they do, and has a focus on collaboration, solutions, integrity and safety by considering the effectiveness, efficiency, impact and reputation of everything they do (RMS, 2015).

As manager of the NSW state road network, RMS is responsible for:

- 18,036km of State roads, including 4,317km of the National Road Network and 147km of privately funded toll roads
- 2,970km of regional and local roads
- 5,287 bridges and major culverts
- 22 tunnels
- 3,945 traffic signal sites
- 12,000 other traffic facilities, systems and corridor assets

In 2014/15, RMS delivered a $5.5 billion program of works which included

- Early works to Westconnex and Northconnex
- Ongoing work to duplicate the Pacific Highway, including completion of the Sapphire to Woolgoolga project
- Ongoing work to upgrade the M1 Princess Motorway
- Ongoing safety work on the Great Western Highway
- Ongoing delivery of the $210 million Bridges for the Bush program
- $1.5 billion of general maintenance
- Investing $1.1 billion in network improvements and pinch point upgrades

These projects were delivered by a workforce of over 6000 Full Time Equivalent staff (as at 30 June 2015).

1.3 What is Country Bridge Solutions?

When replacing a bridge, local government often implements a modular concrete solution as construction of a precast bridge outsources a significant quantity of the specialist bridge construction skills, skills that councils have historically struggled to attract. RMS is developing a new type of modular, precast, prestressed concrete bridge known as Country Bridge Solutions (CBS) to assist local councils in replacing their aging bridge infrastructure. The bridge will be certified to SM1600 loading, have a 100 year design life, be fully submersible, the components will be able to be transported on a standard semi-trailer with axle loadings not exceeding T44 load state and, perhaps most importantly, the design will be available to industry free of charge and with no Intellectual Property claim. This final point is expected to increase competition in supply of the precast elements, thereby reducing the cost of construction while encouraging further innovation and iteration of the design.

1.4 What is Constructability and Safety in Design?

Constructability is essentially optimisation of design in order to facilitate ease of construction. In a similar vein, Safety in Design is the consideration of construction and operation safety during design in order to minimise safety risk. It is widely acknowledged in the literature introduced later in this dissertation that early consideration of constructability and safety in design will lead to a safer and more constructable project when measured against key criteria of time, quality, cost and safety.

1.5 Where is the pilot construction occurring?

The pilot bridge will be built to replace the existing crossing of Bookookoorara Creek on Mount Lindesay Road, 34 km north east of Tenterfield in NSW. Mount Lindesay Road is an important community link between Tenterfield and Woodenbong in NSW and the Darling Downs in QLD.

The existing single lane timber beam bridge (as shown in Figure 1-1 looking from Stanthorpe towards Tenterfield) was constructed in the early half of the 1900's and, although it has served the community well, exhibits degradation including dry rot and termite damage. The new two lane concrete bridge will be built to the left of frame, after which the old bridge shall be demolished.

Figure 1-1 - Bookookoorara Creek bridge (existing)

1.6 Project objectives

The broad aims of this research project can be divided into two stages. The first stage of the project is to construct the new bridge over Bookookoorara Creek as the pilot construction under the CBS program in order to document construction progress and identify areas that the CBS design may be improved from a constructability perspective. The second stage is to formulate concept designs or methodology changes to assist in addressing the identified issues, develop an analysis matrix and recommend a single concept for progression for each issue.

While this project is an academic exercise, it is envisaged that some of the issues raised in this dissertation may be considered further by RMS and the outcomes potentially adopted by RMS for inclusion in a revised design and drawing set. The primary purpose of this project, and the anticipated end contribution to the relevant body of knowledge, is to assist in further refinement of the CBS system by providing information and suggestion to the relevant parties regarding construction issues for the project.

These general aims can be broken down into specific project objectives, being:

- To investigate and discuss key constructability aspects and issues with a focus on concrete bridge construction
- To investigate existing precast concrete bridge systems available to the general market
- To construct the new bridge over Bookookoorara Creek and maintain a construction diary noting key activities and progress
- To identify design and construction issues experienced during the bridge construction
- To develop concept options (design or methodology) that may resolve the identified issues
- To develop a matrix and (time permitting) analyse each of the concepts on the basis of constructability and safety in design prior to recommending one concept for each identified issue

1.7 Conclusion

This chapter has broadly provided context of the current state of local government timber bridge stock in NSW and introduced RMS, CBS and the general concepts of constructability. This general information will be explored further in the following chapters. The aim of this report is essentially to present the findings of the project and demonstrate fulfilment of the project objectives, with information presented as follows. Chapter One has present background information to the topic and introduced the project objectives and drivers. Chapter Two includes a literature review to further explore the project context and justify the direction of the project, followed by Chapter Three which defines the project methodology. Chapter Four presents the bridge construction methodology, is followed by Chapter Five which introduces and discusses the constructability and safety issues experienced during of the bridge. Chapter Six then introduces concepts that may be suited to resolve or assist in resolving the identified issue, before Chapter Seven evaluates some of the options to identify a recommended. The report will conclude with summation of findings and recommendation for future research in Chapter Eight.

2 Literature review

2.1 Introduction

The purpose of this section is to present, analyse and discuss the available literature relating to modular bridges and constructability. Presenting the available information in this way will allow justify the existence of this dissertation by identifying the knowledge gap this project is intended to assist in filling.

The review will first provide a brief overview of bridge construction and types in Australia, with a focus on the simple timber beam bridges that are typically found on council roads today. The current stock and condition of council timber bridges will then be discussed, as well as the typical modular concrete bridge types that are used to replace failed, aged or deteriorated bridges. Finally, the concepts, elements and drivers of constructability and safety in design will be introduced and explored.

2.2 History of timber bridge construction in NSW

In 1770, European Captain James Cook charted the east coast of Australia and claimed it for King George II of England under the name of New South Wales. Some 18 years later, the First Fleet arrived and established a settlement at Sydney under command of Captain Arthur Phillip (Australian Government, 2015). The colony was initially confined to the Sydney Basin and slowly expanded along the northern and southern coastal areas of the state. Initial exploration was by ship, with expansion to the inland areas of the state being limited by shear distance from Sydney and attributes of the waterways.

Soon after settlement, a timber log bridge was constructed over the Tank Stream near the present site of Bridge Street (RMS 2016 from DMR, 1950). The colony of Sydney began expanding westward with a timber bridge being constructed over Duck Creek at Granville by 1797, and a further 10 timber bridges being constructed on the Parramatta Road by 1805 (RTA & Cardno, 2006). These early bridges were simple timber log construction made with local timber; however primitive construction techniques and adverse condition typically resulted in a relatively short lifespan (RMS, 2016 from DMR, 1976). The bridges are thought to have consisted of large longitudinal girders topped by smaller transverse logs to create a deck. Side logs were occasionally added to form a kerb. The colony gradually expanded further west, with a crossing over the Blue Mountains being in place by 1815 (RTA & Cardno, 2006).

By 1800, the colony had formed the basis of a government, works department and civil service which, under the command of Governor Lachlan Macquarie, began providing the civil works and infrastructure. By 1858, the colony had a basic road network (Glencross Grant, 2011) even though it was without scientifically designed bridges until 1832 (RMS 2016 from DMR, 1950). Despite these advancements in the Sydney Basin, settlement in the rest of NSW was primarily confined to coastal areas until the Gold Rush of the 1850's. The coastal settlements were well served by ships, with the few river crossings using punts or ferries as any permanent bridges would need to be of sufficient height to avoid impact on the river navigability and trade (Berger et al, 2015), but settlement of areas west of the Great Dividing Range was sparse. During the Gold Rush, settlers headed west towards the Riverina and north towards the New England regions of the state, with major river crossings typically consisting of fords or punts.

At the beginning of the Gold Rush, building of infrastructure was a function on the Colonial Architect, however, due to increasing population size and distribution, the capacity of this department to provide the required works was exceeded and the Public Works Department was established in 1859 (RTA & Cardno, 2006, Glencross-Grant, 2009 and Glencross-Grant, 2012). In 1861, the state government decreed that local materials (including timber) should be used in preference to wrought iron, likely due to the high cost of import from England (RTA & Cardno, 2006), hence the 'timber bridge boom' was born.

Timber bridges were constructed in two different ways depending on the traffic and flood requirements.

Large timber truss bridges of up to 27 m span were constructed in five main designs between 1850 and 1936, being Old Public Works (1860-1886), McDonald (1886-1894), Allan (1894-1929), De Burgh (1889-1905) and Dare (1904-1936) (Glencross-Grant 2011 and Fraser 2009). These bridges typically existed either on larger roads which are not under council control, or cover such large spans (up to 27 metres) that they unsuitable for the modular construction method that is the focus of this dissertation. As such, large timber truss bridges will not be discussed further.

Bridges on smaller roads were typically 10 m span timber beam bridges. These bridges were cheap, quick, easy to construct and utilised local materials and, as a result, countless thousands were built, collectively forming the most common type of road bridge for the period. So prolific was that construction of timber beam bridges that, by the beginning of the 20tn century, some 87% of the bridges in NSW were of timber beam construction (RTA, 2000).

In their heritage study published in 2000, RTA classifies the bridges in this era into two design phases, being pre-1894 (traditional design) and post 1894.

2.3 Current timber bridge stock

Local Government is responsible for the management of about 27,000 timber bridges within Australia (Crews et al 2004) with a combined value of $1.5 billion, of which most are at least 80 years old (Balendra et al, 2009). A report published by the Australian Local Government Association (ALGA, 2013) states that 65% of local government controlled bridges are classified as being in a poor to very poor state, while a report published by the Institute of Public Works Engineers (IPWEA, 2012) provides a figure of 30% being in poor condition and a further 49% being in fair condition. Regardless of the figure being relied upon, it is clear that there is a substantial amount of work needed to be done on these bridges in order for the road network to remain serviceable.

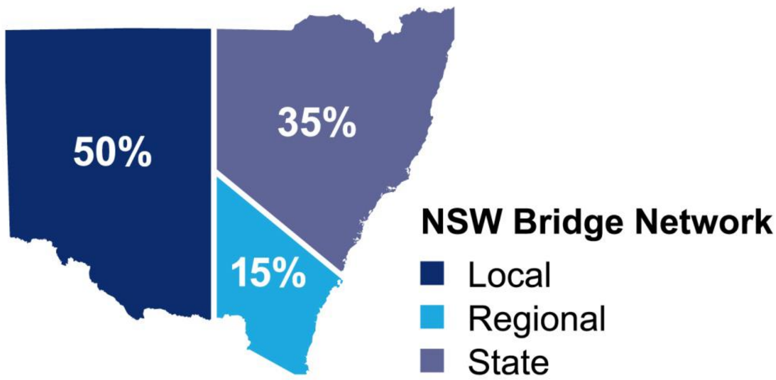

Figure 2-1 - Distribution of NSW Bridges (RMS, 2015)

By means of example, in 2013, Kyogle Council, with a population of 10,000 people and annual rate revenue of less than $5 million, had 420 bridges under its direct control, of which approximately half are in good or very good condition. This small council has had four bridges collapse since 2004, but only has a $900,000 bridge replacement program which allows six single lane bridge replacements per year, well below the quantity required to improve the overall bridge condition in an acceptable timeframe (*The Sydney Morning Herald,* 2013). In 2014, RMS estimated that completing all the required timber bridge replacements on regional roads in NSW would cost approximately $460 million.

2.4 Maintenance requirements

Balendra et al (2009) and McDougall (2006) recognise that timber degrades when exposed to the environment, and therefore has a high maintenance requirement. Timber is susceptible to damage from fungal rot, borers, termites, fire and impact damage, so much so that a general heuristic is to allow for a major rebuild of a timber bridge every 20 years (S. Pereira, pers. Comm. 2016). Regular inspection of timber bridges is required in order to maintain a structure condition inventory, but, as discussed by the Local Government Engineers Association of NSW in their 2013 submission to the Local Government Review Panel, access to skilled staff such as those required to inspect and maintain such an inventory is an issue, especially for more remote councils. Moore et al highlights this issue in their 2009 publication, stating that 64% of council have no knowledge of the load capacity of their bridges, 17% have staff with qualified bridge councils and only 4% plan to load test their bridges within one year of the publication date.

This skills shortage, combined with an infrastructure backlog and uncertain bridge capacities, leaves councils in a difficult situation of needing skilled maintenance work with neither the funding nor the required level of technical skills to deliver such works. Bridge replacements are often required, and a number of options exist to complete such works.

2.5 Bridge replacement options

Murray (n.d.) describes the first dilemma for a municipal engineer is not finding bridges that need replacing; rather it is selecting which bridge to replace first. Repair of bridges is generally the preferred option as it is quicker, easier and, most importantly, cheaper than a full bridge replacement. Nonetheless, bridge replacement is a common council activity hence a number of proprietary products are available to assist.

The majority of deteriorated timber bridges under local government control are located on local roads with AADT of less than 1000. Modular precast concrete bridges are a common choice as, by completing the major structural elements off site, there is less site work required so construction is quicker (and therefore lower cost), quality is easier to control and less specialised skills are required in the site staff (Degenhart, 2013). The four most common modular bridge products used in such applications are the Holcim HumeDeck, Rocla M-Lock, Doolan deck (Murray, n.d.) and deck planks from various manufacturers (Structural Concrete Industries, n.d. and Civilbuild, 2016). Each of these options will be discussed in more detail in following sections.

2.5.1 Holcim HumeDeck

Holcim (2015) describes the HumeDeck as a precast modular bridge system capable of spanning between 6 and 12 metres which is commonly used in regional areas for council timber bridge replacements. The units incorporate a combined deck and girder arrangement (as shown in Figure 2-2) and can be installed on the existing bridge substructure or as a new bridge construction. The units have a design life of 100 years in accordance with *AS5100-2004 Bridge Design*.

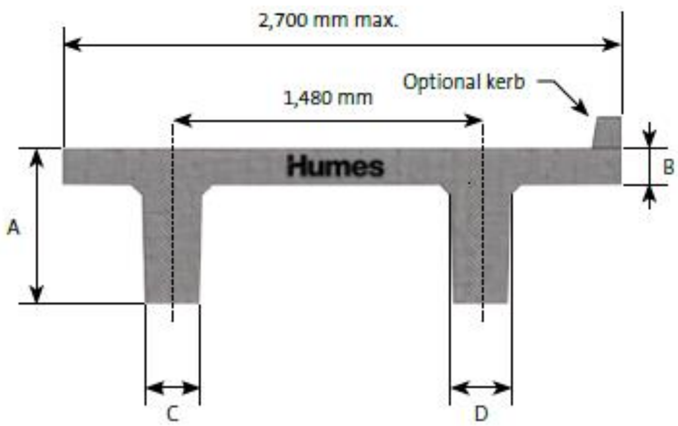

Figure 2-2 - Humedeck typical cross section (Holcim, 2016)

The substructure consists of 550 x 550 mm rectangular reinforced concrete piles or prestressed octagonal piles from 400 to 550 mm diameter (driven or potted depending on geotechnical conditions) topped by precast headstocks and abutments (Holcim, 2015). Hold down bolts secure the deck units, while elastomeric bearing pads provide allowance for movement. Once placed, the units are typically butt jointed with a 10mm gap for sealant although there is the option for an in-situ stitch joint to provide a more rigid connection. The HumeDeck is only available for use and purchase from Holcim. Despite the modular arrangement, Degenhart (2003) states that each application of the system is custom design. This statement is contrary to the information available directly from the manufacturer. Degenhart also provides that the mass of a 12 m x 2.7 m and 12 m x 2.4 m deck unit is 29 and 30 tonnes respectively, which may present logistical difficulties if other bridges en-route to the site are load limited below this weight.

2.5.2 Rocla M-Lock

Rocla (n.d.) describes the M-Lock ® as a precast bridge system capable of bridging small to medium spans of 7 to 15 metres with skews of up to 30°. The deck units consist of 1200mm wide inverted U-sections with transverse end diaphragms as shown in Figure 2-3. The typical application is roads with AADT less than 1000, but transverse stressing can be used for roads with traffic greater than 1000 AADT. The units are certified by Rocla to the T44/HLP320 or SM1600 loading case, dependent on client requirements.

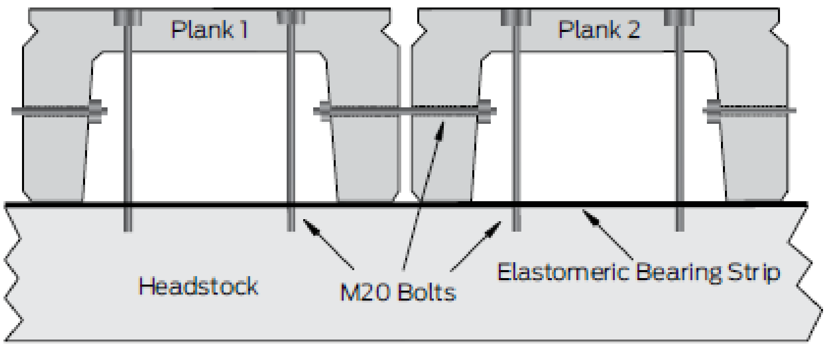

Figure 2-3 - M-Lock typical cross section (Rocla, n.d.)

Similar to the HumeDeck, the substructure consists of driven or potted reinforced concrete piles with concrete headstocks. The units are secured with hold down bolts but, unlike the HumeDeck units, sit atop bearing strips rather than pads. Once placed, the units are but jointed together and sealant applied between them. Scott et al (n.d.) state that 'through innovative design and good construction techniques, construction of the M-Lock bridges achieved minimal construction cost combined with minimal site works'. The M-Lock is only available for purchase and use from Rocla.

2.5.3 Doolan Deck

The Doolan Deck is a composite timber girder and reinforced concrete deck unit developed by DMR engineer Terrence Doolan in the 1990's. The concrete deck protects the timber girders from the weather, thereby removing the main driver of timber deterioration. The girders have a plate and coach screw arrangement which is cast into the concrete slab, with the arrangement achieving sufficient connection to allow composite action (Austroads, 2009). The deck units could be placed on either a timber or concrete headstock and were butt jointed with a bead of sealant applied between adjacent units.

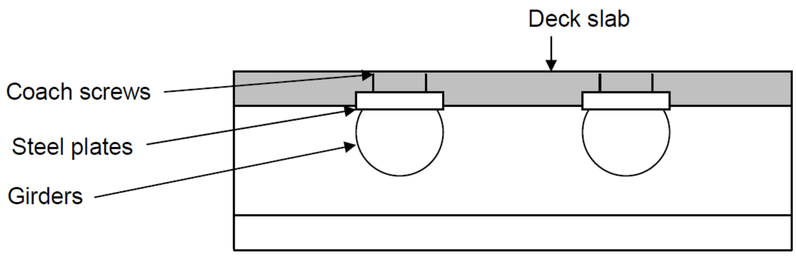

Figure 2-4 - Doolan deck cross section (Austroads, 2009)

2.5.4 Plank bridges

Austroads (2009) describe prestressed concrete plank as the standard bridge type for spans of up to 22 metres. Solid planks vary from 240 to 380 mm thick and are capable of spanning up to 10 metres, whereas voided plank vary from 422 to 700 mm thick and are capable of spanning up to 22m (Structural Concrete Industries, n.d.). The planks are placed side by side on a concrete headstock and have a topping slab cast over the top.

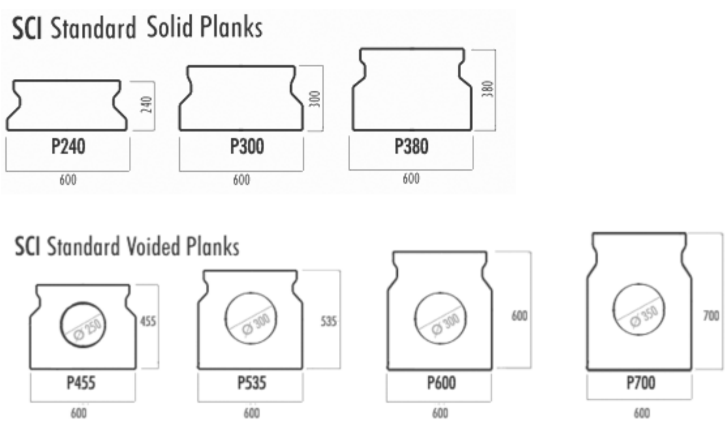

Figure 2-5 - Standard plank cross sections (Structural Concrete Industries, n.d.)

2.5.5 RMS Country Bridge Solutions

Country Bridge Solutions (hereafter CBS) is a NSW State Government program aimed at developing an innovative and cost effective solution to enable regional council to replace bridges on their roads (RMS, 2016). The modular bridge system consists of precast, prestressed double-tee deck units placed over elastomeric bearings on precast headstocks. Headstocks are supported on either piles or cast in-situ foundations. Deck units are joined by a cast in-situ stitch pour longitudinal to the traffic direction, with a simple sealant joint between spans. Typical cross sections of the bridge and deck units are presented in Figure 2-6 and Figure 2-7.

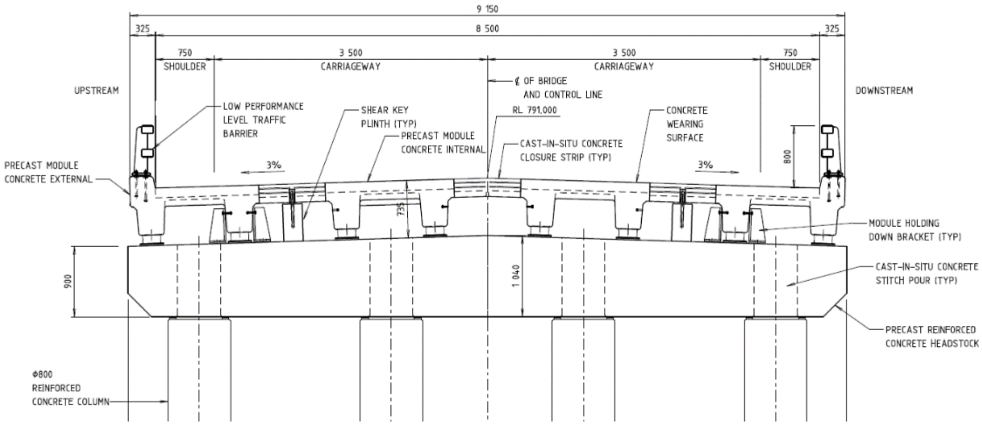

Figure 2-6 - CBS cross section (RMS, 2016)

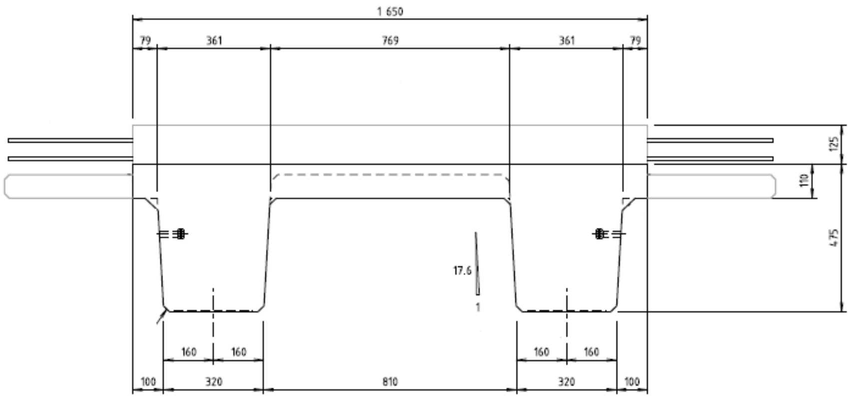

Figure 2-7 - CBS deck module cross section (RMS, 2016)

This project will be the first time the system has been constructed, hence there is no literature available on the construction of the bridge system. The preparation of this report is intended to assist with closing this knowledge gap.

2.6 Constructability

The Construction Industry Research and Information Association (CIRIA n.d.) through Zhong et al (2015) and Cheetham et al (2012) describes constructability as 'the extent to which the design … facilitates the ease of construction, subject to the overall requirements of the completed building', while Kannan et al (2012) describes constructability as 'the optimum use of construction knowledge and experience in planning, design, procurements and field operations to achieve [the] overall project objective'. While slightly different, both of these definitions encompass the concept of optimisation of design as a method to aid construction (Mbamali et al, 2005). It is important to note that in the available literature, constructability and buildability are used interchangeably.

Kannan et al provides a useful diagram demonstrating the relationship between design and construction which is reproduced as Figure 2-8. Griffith & Sidwell (1995) concur with this viewpoint, stating 'it is essential to consider constructability at an early stage in the total construction process, because the ability to influence project cost, and so value for money, from the client viewpoint, diminish as the project progresses in time'. This concept is further postulated by Jergeas et al (2001), who states that 'while constructability does not necessarily add to or improve the function or operating reliability of a project, the inclusion of construction knowledge and experience into the planning and design or a project can result in reduced installed cost and improved safety during construction' and that 'all benefit or constructability can solely be achieved by the integration of the construction knowledge and experience into each phase of the project'.

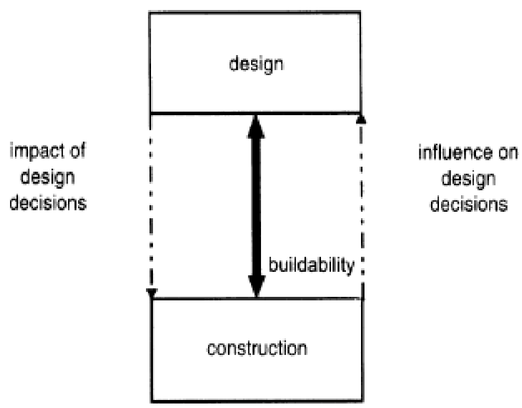

Figure 2-8 - Constructability relationship (Kannan et al 2012)

2.6.1 Elements of constructability

Zhong et al (2015) and Lam & Wong (2011) provide elements of constructability in their papers which, while using different phrasing, are thematically similar. These elements are presented below in Table 2-1.

Table 2-1 - Elements of constructability

Zhong et al (2015)	Lam & Wong (2011)
Construction duration	Allowing economic use of contractor's resources
Construction safety	Allowing design to achieve safe construction sequence on site
Construction flexibility	Enabling contractors to develop and adopt alternative construction details
	Enabling contractors to overcome restrictive site conditions
	Enabling freedom of choice between prefabricated and onsite works
	Enabling simplification of construction details in the case of non-repetitive elements
	Minimising the impact due to adverse weather by enabling a more flexible construction program
Construction quality	Enabling standardisation and repetition
	Enabling design requirements to be easily visualised and coordinated by site staff

In their earlier paper, Lam & Wong (2008) condense the concept of constructability to be a summation of the considerations of construction time, cost, quality and safety, values consistent with, but less verbose than, the above table. Monghasemi et al (2014) also uses the optimisation of time, quality and cost as a method of raising the efficiency of construction practices, a quality that concurs with the earlier definition of constructability as presented by Kannan, while Lam et al (2012) conclude that by enhancing efficiency and safety of designs, quality, value and buildability will improve. Mbamali et al (2005) agrees with this sentiment, and states that bringing together the technical experience of the builder and design experience of the engineer at an early stage is necessary for integrating ease of construction into design.

Further to this, El-Rayes and Kandil (2005) state that the minimisation of construction time and cost, combined with the maximisation of quality, will present to most optimum solution, although, as discussed by Monghasemi et al (2014), these values are often competing whilst rarely complementing. For example, Zhang and Feng (2010) discuss that using lower cost resources (desirable) generally increases construction time (undesirable), while using higher cost resources (undesirable) generally reduces construction time (desirable) and, as a combination of the two, reduction of construction time or cost (desirable) generally reduces construction quality (undesirable). As such, it is important to find a balance between these competing elements.

Monghasemi et al (2014), El-Rayes and Kandil (2005) and Lam and Wong (2008) all present qualitative tools for measuring and optimising performance of specific projects, however they are considered to be beyond the scope of this review due to their complexity and project specific nature.

Further detail on the importance and value of time, quality, cost and safety in construction projects will be explored further in the following pages. It will be shown that these elements cannot be considered in isolation; rather that they are interconnected, commonly being referred to as the “project management triangle” (Eze Castle, 2010).

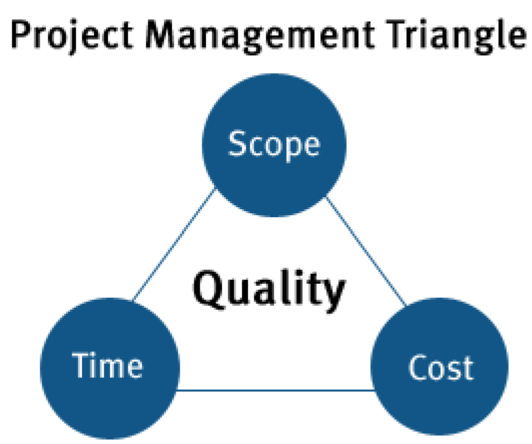

Figure 2-9 - Project Management Triangle (Eze Castle, 2010)

2.6.2 Construction time

Despite continual technological advances, construction is a labour intensive industry with labour productivity being a key performance measure (Jarkas, 2015). Bowen et al (2012) agrees with this viewpoint, and concludes that timely completion of construction projects is a major indicator of project success from the view of the client. The client in Bowen et al's analysis is described as the asset manager or owner however, given the municipal setting of the bridge construction in question in this project, it is pertinent to consider the client as the end user i.e. the ratepayer or general public. While not directly considered in the aforementioned publication, the public's initial assessment of a projects progress or success is largely conducted on the basis of time management and construction duration as they are typically not privy to the finer details of the project's financial or quality circumstance and, as such, efficient construction practices play a major role in public satisfaction or otherwise with their various levels of government.

Labour productivity and output is a complex area, however Jarkas (2015) states that constructability is amongst the most important factors in determining and reducing construction time. In their 1991 study of constructability in the automation of pipe laying operations, Fisher and O'Connor (1991) found that productivity improved (activity duration decreased) by 24% when constructability was considered in the design process. While this study was not specifically related to bridge construction, this is consistent with the conclusions of a 1997 survey by the CIRIA which found that 80% of the 66 industry respondents agree that reduced construction time is a measurable benefit of constructability (Atkinson et al, 1997 through Lam & Wong, 2009). Francis et al (1999) also presents six individual projects in their paper and discusses the benefits obtained by each project as a result of considering constructability, with five of these projects realising significant time savings.

Holla et al (2016) describe the benefits of using precast concrete products with reference to time, not only due to components being delivered to site at a set stage in the construction which minimises handling and equipment usage time, but also due to the repetition of installation. Dineshkumar & Kathirvel (2015) briefly discuss another significant time advantage that can be realised by using precast, namely the removal of the need to wait for on-site curing of concrete as elements would be not be delivered to site until the specified concrete strength has been reached. Nonetheless, Shazar et al

(2015) make note that any efficiencies gained by the use of precast components are project specific hence results are not guaranteed.

The influence of constructability is not confined to ease of construction, it also encompasses additional time required due to variations and rework. Variations are defined as the difference between the planned task and the actual task (Russell et al, 2014), whereas rework is the need to improve or make good a defect (Gorse et al, 2012) which, in the context of this review, could be avoided by improved design practices.

Oladapo (2007) states that some 68% of time overruns are due to design variations during the construction phase. Indeed, of the 30 projects included in Oladapo's study, all projects experienced time delays ranging from 11.1% to 800% of the contract period with the most common cause of the delays identified as design errors resolvable with greater consideration of constructability by the design team. This conclusion is echoed by Ndihokubwayo & Haupt (2008) who, after completion of their industry survey, suggest "a need to refocus the design stage with regards to the occurrence of variation orders" and Ismail et al (2012) who, after their survey, find that "errors or omissions in design" is the second most leading cause of variations, beaten only by "Change of plans or scope by employer [principal]", with the most important effect of variations being a delay in project schedule. Further to this, Russell et al (2014) note that construction projects contain a high degree of task interdependency hence delay in one project area will inevitably result in delay in other projects areas. The outcomes of design variations are typically an addition to the quantity of work performed, or a requirement to perform rework to correct prior construction activities.

With regards to rework, Forcada et al (2014) studied a major highway construction project as a collection of eight sub-project and found that 5/8 projects experienced as a result of "inappropriate design". Russell et al (2014) estimated that rework added an average of 1.81 hours of work per week to the typical construction project, whilst Simpeh et al (2015) states that the mean total rework cost is 5.12%, however the probability of exceedance of this value is high at 76%. Yang (2014) and Hwang (2007) concur with this sentiment, independently finding that rework is one of the single greatest causes of changes to construction time

It is therefore concluded that consideration of constructability and methodology throughout the project design and development phase has significant potential to result in more efficient construction time.

2.6.3 Construction quality

Quality is "the totality of features or characteristics of a product or service that bears on its ability to satisfy stated or implied needs" (American Society for Quality, 2016). The studies available in relating quality and constructability are typically qualitative as it is not practical to construction multiple identical projects for the purpose of a comparative study, hence the conclusions discussed herein are primarily the result of industry surveys.

Gransberg et al (2004) describe how the design team can influence quality not only by ensuring that their design can be built in accordance with industry best practice, but also by designing to appropriate technical specifications, with Tan (2000) acknowledging through Low (2001) that buildability [constructability] typically gives rise to better construction quality and reduced rework. Low then goes on to analyse past projects and concludes that a positive relationship exists between buildability and structural quality, with structural elements assessed including precast and in-situ reinforced concrete such as would be required for the previously introduced concrete bridge options.

Consideration of constructability in the design phase to avoid over-complication of site works typically results in efficient site operations and results in less rework (Lam et al, 2005), the benefits of which has been discussed in the preceding section of this review. Despite this wide acknowledgement, Trigunarsyah (2007) conducted an Indonesian industry study which concluded that "the majority of designers were more interested in preparing their design than interfacing with construction personnel", a finding that is concerning when read in the context of available literature. Further to this, Trigunarsyah also found that the most designers self-assessed their project quality performance as above average, a finding that is both statistically impossible and counter-intuitive given the hesitation towards consultation. It should however be noted that this is a single study into the conditions of a single country, hence it would be unwise to draw broad conclusions about other countries, particularly Australia, from this study alone.

The use of precast elements in construction projects is a sound constructability outcome as it effectively remove the construction tasks associated with some of the most complex components of the job, some of which are unable to be undertaken on the majority of construction sites (e.g. pre-stressing), however improved quality also typically results due to repetition and controlled environs (Holla et al, 2016 and Kim et al, 2014). Regardless the environment, Kendall et al (2003) note that the quality of precast output is largely dependent on the individual construction team hence adequate surveillance is a necessity.

2.6.4 Construction cost

Construction cost is essentially a summation of the cost of time (labour and plant) and the cost of materials. Section 2.6.2 discussed how good constructability can reduce construction time, a change which will have positive flow on effects to construction cost. This viewpoint is reinforced by the previously referred 1997 CIRIA study which found that the overwhelming majority of respondents agreed that major cost benefits for clients, designers and contractors will result from good constructability. This concept has also been quantified by the Business Roundtable (1982) through Lam et al (2005) who identify the benefits of good constructability as being in the range of “10-20 times the cost of achieving them”.

Minimising site works by the incorporation of precast elements into construction projects is an example of sound constructability. Tam (2005) and Chan (2001) discuss this concept and conclude the incorporation of precast may have positive impacts on overall project cost. This impact presumably due to the removal of the need for site facilities including traffic control, travel costs and ease of working, however additional costs will be incurred as a result of transporting each precast element to site and installation cranage.

Holla et al (2016) state that a major cost benefit or precast components is repetition of construction which inevitably creates time efficiencies, thereby lowering construction cost. Almansour & Zounis (2010) concur with this conclusion, stating that “the use of standard precast-prestressed girder sections is a popular and cost effective solution for the construction and replacement of short and medium span bridges” not only with regard to upfront cost but reduced maintenance cost due to higher quality (Chen et al, 2010). It should however be noted that the degree of cost efficiency realised is largely influenced by the quantity of elements included in each production run, so the use of a significant number of different non-repeated precast elements in unlikely to result in notable cost savings.

Project cost estimates are prepared on the basis of the final design and anticipated construction durations. Incorporating constructability into the design can not only result in reduced duration and therefore cost, but cost savings can also come through reducing variations to design (and typically the contract) during construction (Jergeas & Put, 2001).

Oladapo (2007) describes the impact of variations on construction cost, and concludes that 79% of cost overruns for the projects (30) in the study is the result of variations and/or associated rework. Moreover, Hwang et al (2007) studies 359 projects and concluded the rework accounted for an additional 5% of total construction costs with design error or omission being one of the leading causes.

When considering the various elements of constructability, it is readily apparent that the elements are all linked and interdependent. A poor constructability outcome that negatively influences quality will have flow on affects to time and cost through rework and increased maintenance requirements. Likewise, a highly constructable project will typically result in decreased working time producing a decreased cost, however care must be taken to ensure that decreased quality does not result. Conversely, increased time (not specifically related to constructability) will likely result in an increased cost but may also be accompanied by increased quality. It is therefore imperative that these elements and their impacts are considered in their totality during design as it is not uncommon for a trade-off between the various areas; a decision that must be made in the best interests on the project (Lam et al, 2005).

2.7 Safety in design

Safe Work Australia (2012) describes safety in design, or safe design, as "the integration of control measures early in the design risk process to eliminate or, if this is not reasonably practicable, minimise risks to health and safety throughout the life of the structure being designed". Put simply, safety in design is the consideration of safety and safe work practices during both the construction and operation (inspection and maintenance) phases of the asset lifecycle. It is noted that safety in design is also referred to as prevention through design, mainly in UK literature.

2.7.1 Legislation

Under the Commonwealth Work Health and Safety Act (2011), the person conducting a business or undertaking (PCBU) has the primary responsibility to ensure, so far as reasonable practicable, the health and safety of worker while they are at work in the business or undertaking, with all Australian states and territories also having their own legislation. When considered in the context of construction projects, the project proponent is the PCBU, however various levels of management have responsibility for implementing appropriate policies and controls to ensure that safety in competently considered and maintained while at work.

2.7.2 Impact

Weinstein et al (2005) postulates that "assessments of the impact on safety in design reveals considerable promise for the concept in reducing construction site injuries", with an estimated 60% of construction accidents thought to be eliminated, reduced or avoided with more thought taken during the design phase, a sentiment concurred with by Morrow et al (2014) who states that 'designers can play an important role in reducing risks to those involved in construction activities'. The available literature suggests that in the range of 40-50% of construction workplace fatalities the design was linked to the incident (Behm, 2005 & Driscoll, 2005), however it is noted that it this is a broad conclusion and could relate to any area related to the design e.g. plant selection, methodology.

The ability to manipulate project direction (including scope and construction methodology and therefore safety) is greatest when in the early stages of project development, a reality represented by Figure 2-10. Project development is often an iterative process involving range of stakeholder with competing priorities, many of which will change over the life of a project (Lingard et al, 2013 & Olander, 2007). Furthermore, Fadier et al (2003) states that the engineering and safety choices made early in development can set boundary conditions reflecting tolerance for project risk, with Lingard et al (2015) noting that when hazards are identified and control measures implemented early in the project, the controls are likely to be of a higher order (elimination or substitution) than those which would be implemented responsively in the construction phase (engineering, administrative or PPE).

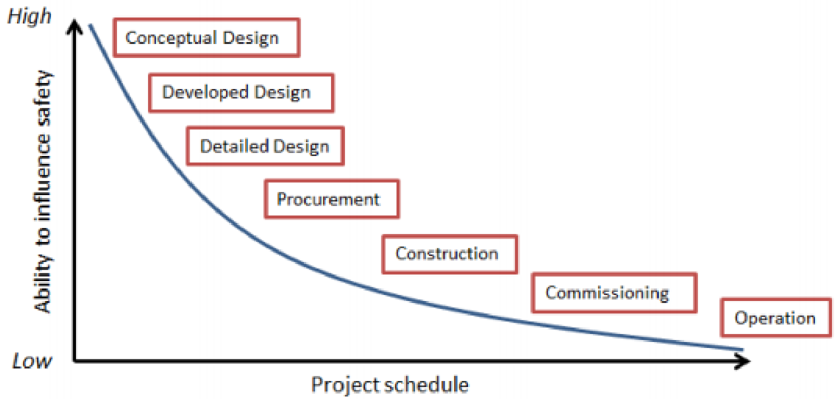

Figure 2-10 - The time/safety influence curve (Hochwimmer & de Krester, 2015)

It is well established in the available literature that design and the resultant method of construction are contributors to workplace accidents and injuries, however Manu et al (2014) argue that knowing the degree of harm is an essential component in determining the overall risk (impact and likelihood) on the construction site. Manu et al (2009) agree with this sentiment, having independently developed an evaluation system of the different Construction Project Features (CPF's) some years earlier which essentially multiplies the impact of the factor by the likelihood of its risk being realised. The output of such a process is an objective evaluation of the greatest project risk, with an interesting note that, regardless of the magnitude of a hazard, if there is no exposure to the hazard then no risk results. It is noted that this approach is generally consistent with the risk matrix evaluation approach typically carried out on construction sites, however, as discussed earlier, performing this activity through project development is likely to pay safety dividends during construction.

Further to Manu et al's approach, it is essential to develop a project safety risk register as early as possible and actively update it throughout project development (Hochwimmer & de Krester, 2015) including rationale behind trade-offs between safety and other competing elements when required (Lingard et al, 2013). Maintenance of such a register is important not only to show consideration of safety as a sound design element, but also to demonstrate pro-active compliance with legal requirements. At completion of the project, the lessons learnt, both positive and negative, should be distributed throughout the company and potentially industry to add to the collective knowledge pool.

2.8 Conclusion and research direction

The literature review contained herein has discussed the history of timber bridge construction in the state of NSW and the resultant modern day maintenance burden. When faced with replacement of these aging bridges, the responsible authority typically considers a number of common and commercially available precast bridge systems including the Holcim Humedeck, Rocla M-Lock, Doolan Deck or plank bridge, however a new system called Country Bridge Solutions (CBS) is currently being developed by RMS as another option to assist local government with bridge replacements on low volume roads. The available literature for the current systems is generally limited to manufacturer advertising material however, in the case of CBS, there is no literature available as the system has never been constructed.

The review then introduced the concept of constructability and the key aspects of time, quality and cost. The consensus amongst the available literature is that early consideration of these aspects will increase the potential of a favourable project outcome, being a project that is as quick and easy to build as possible whilst still resulting in a high quality product. Safety in design was then introduced and briefly discussed, with the literature again showing the early consideration of this concept will pay dividends during construction of the project.

Drawing on these conclusions, this project will involve construction of the pilot bridge under the RMS Country Bridge solutions. This system has not been constructed prior to this project, so there is an obvious gap that this project will make progress towards filling. The construction will be documented and presented in a methodical manner to assist the design and site team to visualise and understand the process to assist in considering constructability and safety in design for future constructions of the system. Further to this, areas of design or methodology improvement will be presented and concepts devised to assist in resolution of these issues. Time permitting, the concepts will be analysed using the time, quality, cost and safety criterions identified herein with a view to recommending the most suitable concept. Full details of the methodology can be found in Chapter Three.

3 Project methodology

This project involves a combination of site work in order to construct the pilot bridge and identify site issues, and office based work that will focus on formulating design and methodology concepts that make progress towards resolving the site issues. This chapter will expand on the project objectives identified in Chapter One to clarify the tasks and techniques required for each activity and discuss the method and criteria that will be used for evaluating the suitability of the treatment concepts. Overall, the aim of this chapter is to define the project methodology

3.1 Project objectives

The principal aim of this project is to identify general or specific areas of the CBS design that can be refined from a constructability perspective. Each of the project objectives identified in Chapter One will be further expanded and described to define the process followed to realise the desired outcomes.

The project works were divided into three distinct but interrelated phases. The first phase was to conduct a comprehensive literature review to present the academic and practical context of the project. The second phase was to construct the bridge over Bookookoorara Creek and record key project phases, activities and issues encountered, with the third phase involving analysis of the construction records to present concepts to assist in resolving or mitigating such issues. Time permitting, the concepts will then be analysed in order to determine a preferred option which may be progressed to detailed design by others.

3.2 Phase 1 - Review of available information and literature

The first phase in undertaking this project was to source, study and review literature available for the topic in order to identify some of the commonly available modular bridge types and key elements, elements of constructability and safety in design. Information sources for this review included print and online material available from the USQ Library, Google Scholar, RMS' technical library and discussions with experienced bridge design and construction practitioners. Peer reviewed journals, conference papers and technical reports formed the basis of the review into constructability and safety in design, however limited "hard academic" information was available on the existing precast modular bridge systems available to market and their performance.

The CBS system has never been constructed before; hence there is no publicly available information on the construction process. The direction of the research in the literature review was based on my personal interpretation of the specific objectives that the CBS system is intended to achieve, based on personal knowledge and industry experience. Through completion of the literature review, common themes became apparent and were explored in more detail.

3.3 Phase Two - Construct the pilot CBS project

The second phase in this project was construction of the pilot CBS Bridge over Bookookoorara Creek. The author of this report was the Project Delivery Manager (PDM) for the pilot construction and was therefore responsible for delivery of this construction project, however the implementation of management practices is beyond the scope of this dissertation.

The trial bridge was constructed over Bookookoorara Creek on Mount Lindesay Road in Tenterfield Shire Council LGA. A daily construction diary was kept to record daily activities, progress and issues raised in order to track the construction works and assist in the production of an "as built" program (program is included as Appendix B: Construction program). The author of this report attended site regularly both in capacity as a RMS employee and USQ student in order to lead (RMS) and document (USQ) both general and critical elements of the construction process. Photographs of key components and activities were taken and maintained with selected photos used in Chapter Four to explain the construction process.

3.4 Phase Three – Analyse records and develop concept options

3.4.1 Analyse construction records

Upon conclusion of Phase Two, the records kept were analysed by the student to catalogue the issues noted in the construction diary and identify any common themes (if any) which exist between the issues. Each issue was assigned a number consistent with Table 3-1 in order to provide clarity and track the issue record through the later stages of this project, then discussed further to provide context and a brief background to explain that rationale behind documenting the issue or improvement area.

The issues raised were recorded and catalogued, and named using an Element – Number system. This system is based is intuitive as shown in Figure 3-1 and Figure 3-2, and is intended to provide easy identification of the location of the issues to make the register easy to use.

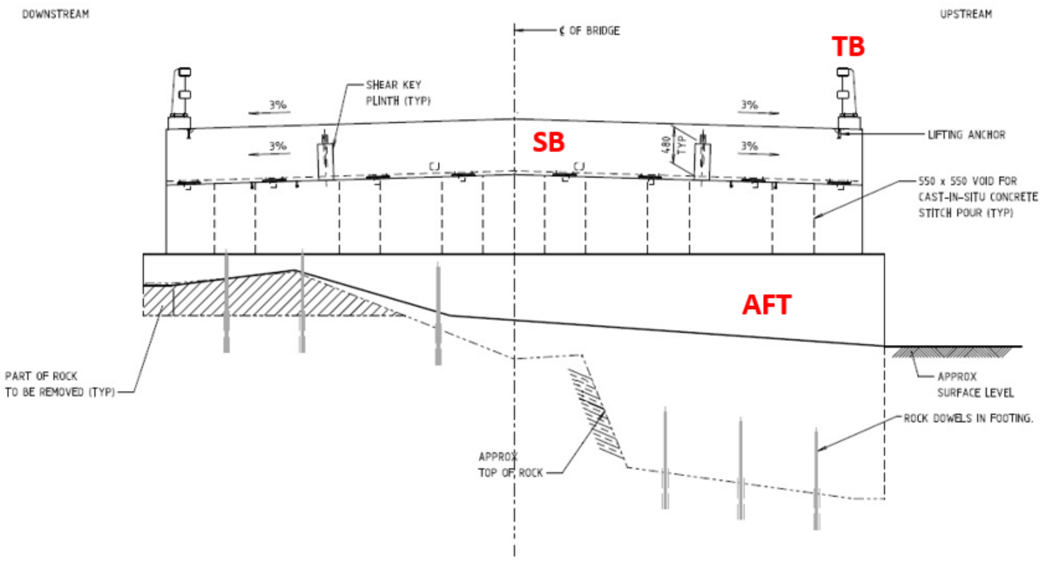

Figure 3-1 - Element coding system for Abutments

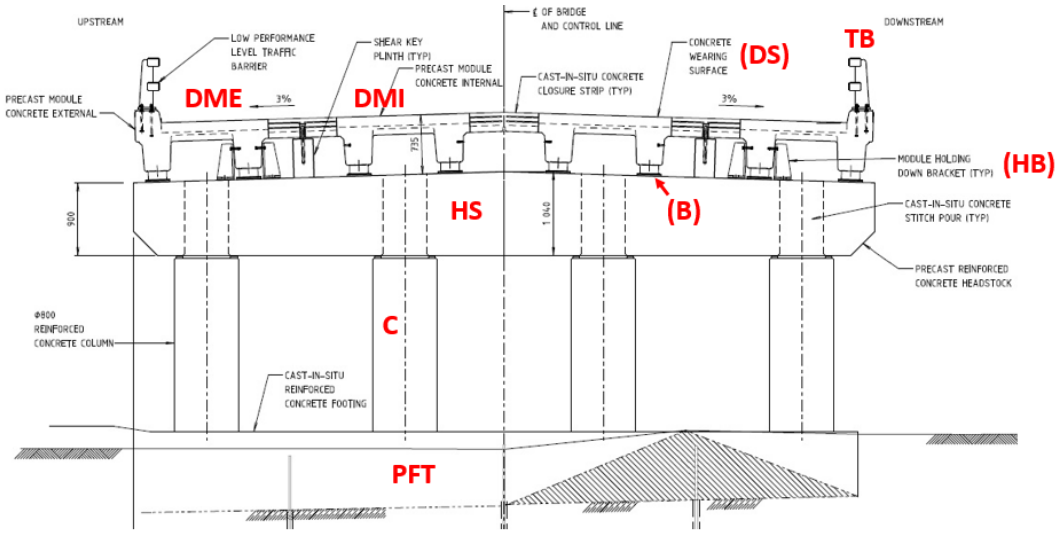

Figure 3-2 - Element coding system for Piers

Table 3-1 - Element coding system

Substructure		Superstructure	
AFT	Abutment Footing	DMI	Deck Module Internal (precast)
PFT	Pier Footing	DME	Deck Module External (precast)
C	Columns	DS	Deck Stitch
HS	Head Stock (precast)	TB	Traffic Barrier
SB	Sill Beam (precast)	J	Joints (expansion)
W	Wing wall (precast)		
B	Bearings		
HB	Hold down Bracket		

3.4.2 Produce concept designs for resolution of the construction issues

Once catalogued and discussed, the project proceeded to the concept design phase (including variations to methodology) in order to identify potential ways to address the issues. This stage of the project was centred on producing ideas and concepts rather than formal structural design, however it goes without saying that structural viability was be considered, even if not formally. A minimum of two concepts are presented for each issue.

3.5 Phase Four – Concept Evaluation (time permitting)

3.5.1 Concept evaluation

Upon completion of the concept designs, some of the concept options were objectively evaluated on time, quality, cost and safety criteria to identify the option that is assessed to be the most viable when considering each individual issue. The evaluation matrix is shown in Table 3-2 and discussed further in the next paragraph. As the purpose of this project is to improve a current system, the matrix is designed to analyse the change that each option will make to the current construction process.

Table 3-2 - Concept evaluation matrix

Weighting		Option 1	Score	Option 2	Score	Option 3	Score
Time	- - %						
Quality	- - %						
Cost	- - %						
Safety	- - %						
Sum							

To use this matrix, each issue was analysed to determine the key driver or underlying motivator or the issue (time, quality, cost or safety) and weightings assigned commensurately. The purpose of the weighting is ensure that the most important component of each issue is given due consideration and the relative importance of each criteria is maintained (for example, for a safety issue, safety would have the highest weighting thereby having greatest influence on the evaluation outcome).

Each option is given a score of -5 to 5 as shown in Figure 3-3, with -5 representing a significantly worse performance than the current arrangement, 0 representing no change, and 5 representing a significantly better performance that the current arrangement. The score was then multiplied by the weighting factor and the scores from each component added together the form a total score for each option. The highest scoring option was considered the most viable and is recommended for future investigation by others.

Figure 3-3 - Evaluation scoring system

4 Construction methodology and activities

4.1 Introduction

This chapter introduces and discusses the methodology employed for the site construction of the subject bridge. The approach pavement works were completed by other and are not discussed in this report. The off-site construction works (e.g. precasting of concrete elements, construction of traffic barrier) completed by specialist contractors are also not discussed due to matters of commercial confidentiality. Figure 4-1 and Figure 4-2 are extracts from the design drawing set which have been included to diagrammatically explain the names of the different bridge elements referred to in this chapter and supplement the naming convention discussed earlier in earlier in this report.

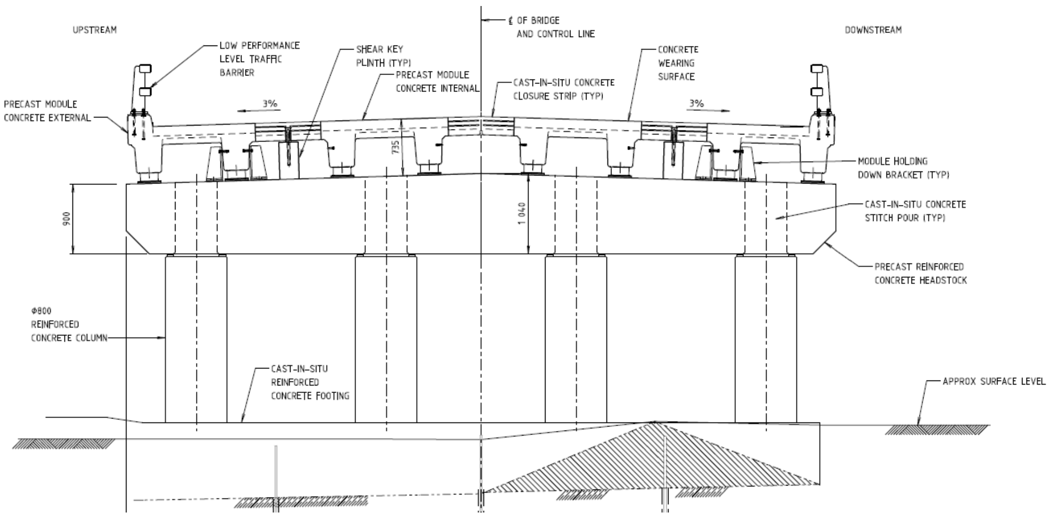

Figure 4-1 - Bridge layout Pier view

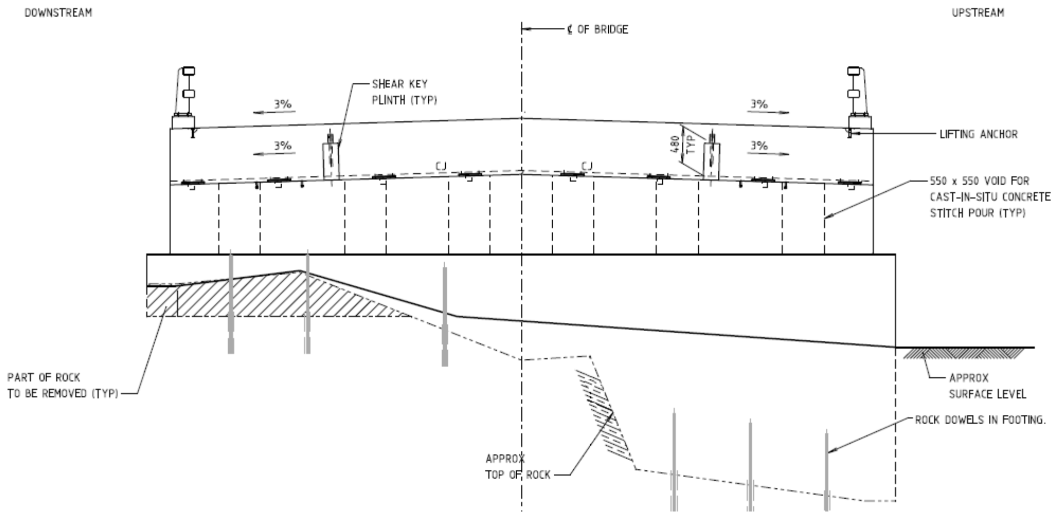

Figure 4-2 - Bridge layout Abutment view

4.2 Bridge construction activities

4.2.1 Site establishment

The first step in construction of the bridge over Bookookoorara Creek was to establish to site. The site boundary was first identified and marked with fluorescent bunting in order to prevent works from occurring outside of the approved footprint. A location for the site compound was identified on the eastern side of the road about 20 m north of the existing. A sediment fence was installed on the downhill side of the compound site before an excavator was used to level the ground in preparation for installation of the site facilities.

A site shed/lunchroom, ablution block and storage shed was delivered on a flatbed truck and placed onto the timber blocks to ensure that the bottom surfaces were not in contact with the ground and susceptible to water damage or corrosion. The site shed contained a microwave, washing up facilities, a drinking water supply, AED (defibrillator) and first aid kit and was nominated as the emergency assembly point and signposted as shown in Figure 4-3.

Figure 4-3 - First aid signage on site shed

In order to improve site safety and reduce the potential for worker injury, the site was divided into zones depending on plant activity based on the RMS Workers on Foot system. The compound site was designated as a safe zone, being an area where there will be no moving plant and therefore no risk to workers from mobile plant. The remainder of the site was designated as a restricted zone, being an area where plant and workers on foot will interact. Signs were erected to show the different zoning – a photo of such signage used during construction is shown below as Figure 4-4. A copy of the Workers on Foot plan is included as Appendix C: Workers on Foot plan.

Figure 4-4 - Worker on Foot safety signage

A vehicle movement plan was devised for the site showing the direction of all site vehicle movements and the designated reverse parking area. The preparation of this plan focused on minimising the area in which vehicles would be moving, and maximising the ratio of forward to reverse vehicle movements. A key feature of the plan was a single direction turnaround area to allow vehicles to enter the site, turn around and re-enter the live traffic lane without any reversing movement. A copy of the Vehicle Movement Plan is included as Appendix D: Vehicle Movement Plan.

Two material storage areas were established; one within five metres of the new bridge on the northern side of Abutment B and one in the centre of the vehicle turn around area. The material storage in the turnaround area was grade separated from the Bookookoorara Creek by a natural grassed berm that prevented any material from being washed into the creek in the event of heavy rain. The storage area closer to the bridge was located immediately uphill from the creek, so a sediment fence was erected on the downstream side.

The final step in setting up the site was the installation of environmental controls. Sediment fences were installed along the edge of the creek to prevent loose sediment entering the waterway. A copy of the Erosion and Sediment Control plan is provided in Appendix E: Erosion and Sediment Control Plan.

4.2.2 Survey setout

With the site facilities now in place, a surveyor was engaged to set out the locations of the Abutment and Pier footings. The surveyor used a local reference point in the form of a nail set into the kerb of the existing bridge to establish the location of the footings and marked their locations with pegs at known offsets.

4.2.3 Clear vegetation from the alignment

The first activity in the construction of the bridge was vegetation clearing. Six mature eucalypt trees were needed to be removed from the alignment, being four on the southern (Tenterfield) end and two on the northern (Stanthorpe) end. The logs were retained intact to provide fauna habitat and the heads chipped and used to form a sediment control berm around the site perimeter. The trees were cut off one metre (approx.) above ground height to allow the excavator to lever the stumps out. Once removed, the stumps were also placed intact near the boundary of the site to provide fauna habitat. Removal of these trees was consistent with the environmental approval conditions for the project. Figure 4-5 shows the felled trees at the northern side of the bridge, with one tree about the be lifted by the excavator to allow the head to be cut off.

Figure 4-5 - Removal of vegetation from Stanthorpe side

4.2.4 Excavation of footings

This bridge substructure consists of two abutments and two piers, all with reinforced concrete spread footings. Reference to the New England Geological Map (Geological Survey of NSW, 1973) indicates that the site is covered by the Stanthorpe Granite rock unit in the northern area of the New England Batholith. Excavation of the footing areas uncovered undulating very hard granite bedrock interspersed with granite boulders up to 2m diameter and decomposed granite gravel as shown in Figure 4-6. Geotechnical investigations in the planning stage of this project consisted of test pits dug at the footing locations and, while these tests showed rock at variable depth, the variability was greater than expected. This resulted in later activities taking longer than planned as all reinforcing steel needed to be bent to fit on site.

Figure 4-6 - Excavation of Pier One

The footings were excavated with a 14t excavator using a bucket and rock pick. Boulders up to two metres diameter were removed to ensure that the footing was founded on bedrock. All piers were excavated to or below the design level, after which a mass concrete blinding layer was poured to provide a safe and consistent working platform in the bottom of each excavation. In pier one, bedrock was uncovered at a depth of approximately 50mm below the finished surface of the footing and was split and removed by a contractor using a DARDA splitting cylinder with the end result shown in Figure 4-7. Once excavated, bunting was installed around the footing perimeter to alert workers to the presence of open excavation hazards.

Figure 4-7 - Rock splitting in Pier One

4.2.5 Pour mass concrete blinding

The bridge design drawings show the footings as being tied, formed and poured directly on top of the natural foundation material. Excavation of the footings uncovered a combination of rock and gravel material as discussed on page 33 and, although the rock provides suitable sound material to walk and form the footing on, the gravel is too soft and is susceptible to movement during construction. To address this issue, a mass concrete blinding was poured to cover the gravel areas and provide a consistent surface to construct the footing as shown in the following pages in Figure 4-8 on page 36.

4.2.6 Install dowels to rock

The spread footings are tied to the bedrock with 2700 mm long N36 galvanised steel dowels, with minimum embedment 2000 mm into drilled 100 mm diameter holes. The dowels are grouted into position and, once concrete is poured, provide shear connection into the bedrock to prevent sliding of the footing. The holes were drilled using a diamond coated core drill by a specialist contractor however, due to the depth of the holes, they were drilled 50mm diameter and inserted with a N24 mm galvanised steel dowel. This change is discussed further in Section 5.5. The embedment length was marked on the dowels and a length of scrap timber was tied perpendicular at the marked location. When inserted into the drilled hole, the timber rested on top of the rock, holding the dowel in position to ensure that minimum embedment was achieved. The outcome of this step is shown below in Figure 4-8.

Figure 4-8 - Concrete blinding and dowels at Abutment B

4.2.7 Bend and tie steel cage

The next step in construction of the footings was to tie the steel reinforcing cage. The dimensions of the cage vary depending of the footing, but generally consisted of N16 and N20 tied at 200 mm centres longitudinally and transversely in vertical and horizontal rows.

The 200 m spacing of the steel resulted in a safety issue as the gaps were large enough for a workers boot to slip through which may cause personal injury as discussed in Section 5.1. Timber planks and sheets of plywood were used to create safe walking paths across the cage during this activity.

The variable depth and dimension of the bedrock discussed on page 33 resulted in the steel being unable to be bent before arriving on site. This complication meant that the steel for the bottom layer of the cage was ordered in straight lengths and bent to fit using the portable bar bender shown in Figure 4-9. The steel used in the top skin of the footings was delivered to site pre-bent as the finished length and width of the completed footing were consistent with the design.

Figure 4-9 - Rebar bender

Starter bars project from the finished surface of the footings to provide steel to either lap the column cages to or to project through the blockouts in the precast sill beam or head stock. Care must be taken to ensure correct alignment of the starter bars, particularly for the connection to the column cage as any error in this stage may result in magnification of the error over the length of the column.

The result of this process was a custom bent cage that, although not dimensionally consistent with the drawings in the bottom of the footing, provided a generally equivalent steel ratio and was consistent with the spirit of the design. The cage was hand tied with black 1.6mm tie wire. Compressed concrete spacers (aspro's) were used to ensure that the 40mm design cover was achieved at all locations. Figure 4-10 shows the cage for Pier Two under construction.

Figure 4-10 - Steel cage for Pier Two

4.2.8 Install formwork and falsework

The next step was to install the formwork and falsework around the steel cage. The purpose of the formwork is to contain the concrete until it achieves initial set, while the purpose of the falsework is to brace and support the formwork. 17mm formply was used as formwork with 45mm x 90mm (2" x 4") timber used as falsework. In areas of soil, the falsework was held into position by steel star pickets driven to refusal. In areas of rock, the falsework was bolted to the rock using screw bolts to prevent movement of the formwork. A 15 mm chamfer strip was installed at 10 mm below the design finish level of the top of the footing so that there would be a level to screed to during concreting. The completed formwork and falsework for Pier Two is shown in Figure 4-11.

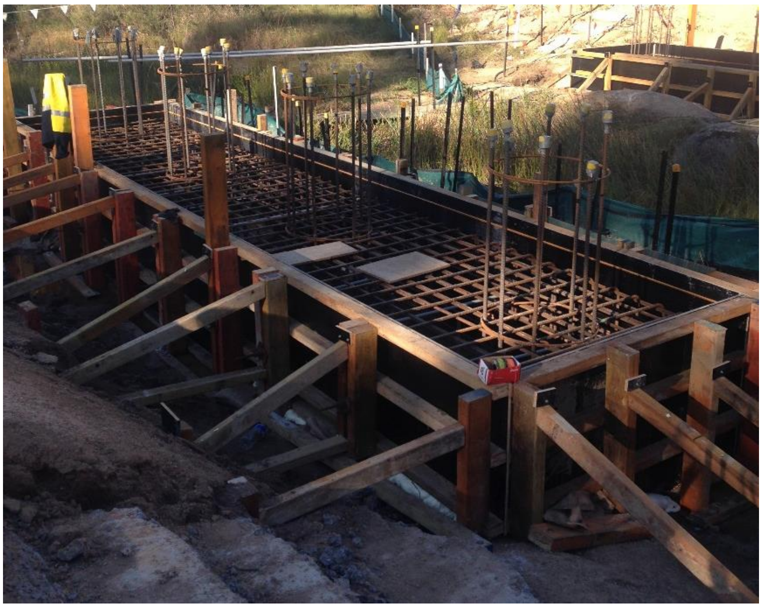

Figure 4-11 - Steel cage, formwork and falsework for Pier Two

The footings were not all constructed at the same time which meant that, with adequate care taken during stripping of the forms, formwork was able to be re-used in other locations. Figure 4-12 shows a section of formwork that was used for Pier One being installed intact as part of the formwork for Abutment B.

Figure 4-12 - Installing recycled formwork to Abutment B

4.2.9 Pour and finish concrete

The next step was to place and finish the concrete. The pier footings were located relatively close to the creek and were subjected to slow but continuous ingress of water so dewatering was required prior to placing concrete. All concrete in this bridge was 40MPa 28 day compressive strength, 80mm slump with 10mm aggregate compliant with RMS B80 Specification. The concrete was supplied by a local concrete plant located approximately 30 minutes from the construction site and delivered in 5m^3 and 7m^3 agitator trucks before being installed by a concrete pump. The truck was mixed for three minutes on arrival prior to samples being taken to test the slump in accordance with method 3.1 of *AS1012:2014 Methods of Testing Concrete*. Sample were taken to test compressive strength at a rate of one cylinder set (three cylinders) per 25 m^3 in accordance with method 9 of *AS1012:2014 Methods of Testing Concrete*.

Once in the pump, the concrete was delivered to the footing by an elevated delivery pipe and placed into position in layers approximately 400 mm deep and vibrated adequately as shown in Figure 4-13. Subsequent layers were placed before the lower layer had reached initial set in order to ensure that no cold joints were formed, with the vibrator extending into the top 150mm (approx.) of the lower layer to allow proper bonding between layers. Care was taken to ensure that the concrete was not dropped from a height greater than 500 mm to reduce potential for segregation.

Figure 4-13 - Pouring concrete for Pier One footing

After the concrete had been poured and vibrated it was time for finishing. The concrete was screeded into position with a trowel or floating screed as shown below in Figure 4-14. After approximately 30-45 minutes the concrete had bled and the water had been re-adsorbed into the concrete. Once this had happened, the concrete was smooth finished with a steel trowel and float. Areas around protruding reinforcement were left with a rough finish to promote adhesion of later concrete pours and form an adequate construction joint. The final process in this stage is curing of the concrete.

Figure 4-14 - Finishing concrete at Abutment B

Due to the remote location and unavailability of large quantities of water at this site, a commercial curing agent (Fosroc Concure A99) was sprayed onto the finished surface at a rate of 5m^2/litre to seal the concrete and mimic wet curing. If the concrete is not cured adequately then the end product may have lower durability or strength than what is indicated by the compressive strength test results.

4.2.10 Strip formwork

Once the concrete has set (allow about 48 hours) the next step was to remove the formwork. The falsework was removed first, followed by the formwork which was removed in whole panels for reuse in other footings. The formwork for this bridge was constructed of formply which is a form of structural formwork with a waterproof veneer designed to not adhere to concrete. If steel or plain plywood formwork was used then a formwork oil compound would need to have been applied in Section 4.2.8 to prevent the concrete from sticking to formwork.

4.2.11 Install column cage

Each pier has four 800mm diameter reinforced concrete columns which vary in height between 2,079 mm to 2,144 mm location dependant. This step is similar to Section 4.2.7 except that the cages were constructed off-site and delivered to site intact. The cages were able to be constructed off-site because, unlike the footings, all dimensions of the column were known and fixed.

The footings have N20 reinforcement projected from them in order to provide steel to lap the column cages to. The finished cages were too heavy to allow safe manual handling, so they were lifted and moved into position using an excavator and sling as shown in Figure 4-15. The starter bars from the footing were aligned with the vertical bars in the cage, lapped 910 mm and tied with black tie wire in the same manner as Section 4.2.7. The cages were checked upon arrival to site to confirm that they were in conformance with the design plans however, as a safeguard, additional hoops and straight bar were provided so that the cages could be extended if their length was too short.

Figure 4-15 - Installation of column reinforcement cage

The design cover for the column was 40mm however, unlike the footings, it would be impractical to tie aspros to the cage as they would likely shift during installation of the formwork. Plastic spacer wheel were used instead because, unlike aspros, they have a round shape that is more fitting with the finished shape of the column which minimises risk of movement of the spacer and damage to the thin formwork.

4.2.12 Install column formwork and falsework

The next activity was to install the column formwork. When considering cylindrical formwork there are typically three options: steel formwork such as pile casing, PVC formwork or cardboard formwork. Cardboard formwork with a smooth plastic liner was chosen for this project and supplied by EzyTube in 819mm external diameter. The formwork was also installed using the excavator and the join with the footing sealed with silicon to prevent concrete leakage. The formwork was supplied in a slightly longer length that the finished column (2200 mm) and, once installed, was cut down to be 50mm taller than the finished concrete level and the cut edge mended with tape. This meant that there would be a known target level to which the concrete would be poured and minimise the risk of the column being too long or too short.

The formwork is relatively flexible and, unless restrained, may move during concreting which would result in a column being either out of tolerance or out of position. Column support frames (falsework) consisting of plywood with a 820 mm hole cut out were placed over the top of the formwork and supported by vertical, horizontal and angled timbers. The completed formwork and falsework arrangement is shown below in Figure 4-16.

Figure 4-16 - Column formwork and falsework

4.2.13 Pour columns

After installation of formwork and falsework, the next step in construction of the columns was the pouring of concrete. The concrete supply and testing operations and considerations were consistent with Section 4.2.9. The delivery pipe from the concrete pump was placed down the centre of the reinforcement cage and concrete was poured and vibrated in a single continuous operation as shown in Figure 4-17. The top surface of the column will be a construction joint with the stitch pour in the head stock blockout and was only rough finished to promote adhesion and form the construction joint. The Fosroc curing agent was applied after bleed water had re-adsorbed at the same 5 m^2/litre rate as used for the footing concrete.

Figure 4-17 - Pouring concrete for Pier columns

4.2.14 Strip column forms

Once the concrete has set (allow about 48 hours), the formwork was removed. The falsework was removed first and retained for reuse. The EzyTube formwork was removed by running a sharp knife down the side of the form and peeling the tube away. The formwork is single use only hence damage was not a concern.

4.2.15 Bag the concrete

After the formwork is removed and the finished concrete surface is exposed, the next step is to bag the concrete. The concrete surface will likely have a number of small holes the represent air bubbles trapped between the concrete and the formwork. While these holes are small do not have any negative structural implication, they are unsightly and, unless filled, could give a false impression of poor construction quality. To fill in these holes, the concrete surface was dampened and a sand and cement mix applied to the surface in a similar manner to rendering over bricks as shown in Figure 4-18. This process is called bagging.

Figure 4-18 - Bagging Pier columns

4.2.16 Install packers to columns and abutments

As discussed in Section 4.2.8, the footing formwork had a chamfer piece installed at a set level to ensure that the finished concrete level was slightly lower than the design requirements. Regardless of this arrangement, there will always be areas of level inconsistency particularly in the centre of the footing.

This step involved checking the height of the footing using a self-levelling laser level and staff. Compressed plastic packers were then installed to design height immediately around the reinforcement protruding from the footing as shown in Figure 4-19 so that the precast head stock or precast sill beam is installed at the correct level. The packers are available in 1, 2, 3, 5 or 10 mm thicknesses each with a different colour code. A combination of sizes were employed at each location to minimise the level tolerance to ±1 mm.

The chamfer strip was installed 10 mm below design level in anticipation of this step as increasing the height of the packers to set the precast element at design level is a simple exercise, whereas lowering a footing that has been constructed too high is practically impossible.

Figure 4-19 - Packers and chalk line marking at Abutment B footing

4.2.17 Install creek crossing for crane

The site was now ready for the precast head stocks to be installed to the columns and the precast sill beams to be installed to the abutments. Installation of these elements required the use of a 100 tonne Liebherr LTR1100 crawler crane. The mass of each head stock was approximately 17.5 tonnes and the mass of each sill beam was approximately 22.4 tonnes which, due to crane capacity, required multiple crane pads to be set up.

The crane had sufficient capacity to install each sill beam from behind its' respective abutment but it needed to be able to access both abutments for this to happen. The crane was to arrive on site at Abutment B, however it was unable to cross the creek on the existing structure due to the uncertain load capacity of the bridge. This resulted in the requirement to install a creek crossing to allow the crane to access Abutment A and have a working platform for installing the pier head stocks.

The 5.5m wide creek crossing was installed over Bookookoorara Creek three days before the arrival of the crane. Geofabric was laid over the area where the track was to be installed and a 300 mm pipe placed to maintain the flow of water and fish passage. Clean blast rock over 300-500 mm dimension was placed around and on top of the pipe and overlain with 50-50 mm rock to create a smooth and safe working surface. The base layer of geofabric was oversized by 1.5 metres on the upstream and downstream sides and, after placement of rock, was raise up tight and secured with star pickets to prevent the smaller rock being washed into the creek in the event of overtopping. A photo of the crossing under construction is provided below in Figure 4-20.

Figure 4-20 - Installation of creek crossing

4.2.18 Install head stocks/sill beams

The site was now almost ready for the arrival and installation of the precast head stocks and precast sill beams. Prior to delivery of the precast elements, the construction team used a chalk line to mark centreline and perimeter guides to the location of the sill beam on each abutment and a centreline guide of the outside piers. Figure 4-19 on page 46 shows the perimeter and longitudinal centreline of the sill beam marked on Abutment A. These markings would later assist in aligning the sill beam or head stock during installation by the crane.

The precast elements were delivered to site on standard semi-trailers with no oversize restrictions as is an important characteristic of the CBS system. Hardwood timber packers were installed below each element and hard plastic edge protection installed over the tie down chains to prevent damage to the concrete. Each element was delivered on an individual truck and installed in the following order: Abutment A sill beam, Pier One head stock, Pier Two head stock, Abutment B sill beam. The order was chosen based on continuous progression from one side the bridge to the other as this minimised the number of individual crane setups, but could be reversed with no adverse issues.

Figure 4-21 - Delivery of sill beam

Once arrived on site, each truck reversed down to the crane and was unloaded. The precast elements all had two 10 tonne lifting anchors cast into the concrete to which standard lifting knuckles were attached. The design drawings show a 30˚ sling angle for lifting, however this was considered impractical due to the required long jib length so a spreader bar was used instead. A spreader bar is typically used where headroom is restricted by structures such as buildings or power lines, features which did not occur on this site, but minimising the length of the jib reduces the length of the moment arm and therefore affords greater crane capacity.

The precast elements were then lifted clear of the truck to allow the truck to leave the site. Once suspended and given authority to enter the area by the crane operator, the construction team then used a chalk line to mark the transverse and longitudinal centrelines on the suspended elements as shown in Figure 4-22. These lines were marked by standing either side of the sill beam, at no stage should any person be located underneath any suspended element due to the risk of crush injury including death should the load fall.

Figure 4-22 - Marking chalk lines on precast headstock

After the elements are accurately marked they were ready to be installed to the Abutment or Pier columns. The crane stayed stationary while loaded and the operator swung each element to above its final location and began to lower it over the projected reinforcement as shown in Figure 4-23. The dogman used a tag line to adjust the position of the element during lowering.

Figure 4-23 - Lowering a precast head stock

The construction crew communicated with the dogman who communicated with the crane operator to slowly lower and reposition each element. The chalk lines marked on centreline of the cast in-situ element were aligned with the chalk lines marked on centreline of the precast element as shown in Figure 4-24 to ensure that all items are in their correct relative positions.

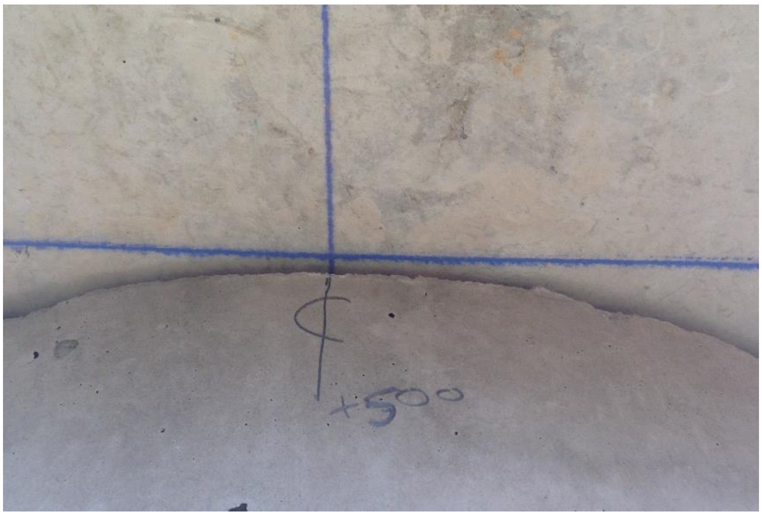

Figure 4-24 - Alignment of chalk lines on Pier Two column

The precast element was now lowered until the element is resting on the packers and all load is taken by the columns or abutment. Minor adjustments to the projected reinforcement from the columns or abutment were required during the step however little time was lost due to these adjustments. The surveyor then checked the level of the placed element before the crane has unhooked from the load so that the element could be raised and additional packers installed if needed. All of the elements were in their correct location in all planes (tolerance ±3 mm) so no adjustments were needed.

The crane then unhooked from the element and the process was repeated until all of the remaining precast head stocks or sill beams were installed. Installation of each element took between 30-45 minutes, with the entire step being completed in less than four hours.

4.2.19 Pour wing wall blinding

The design shows the precast wing walls as being fully supported on the abutment and natural ground at Abutment A and partially supported on the abutment at Abutment B with an overhang of 2,880 mm. This step is identical to that process discussed in Section 36 and involved pouring a mass concrete blinding layer over the natural ground on which the wing wall was to be supported in order to create a safe and consistent working area.

4.2.20 Fill head stock/sill beam voids

The precast head stocks and sill beams have 550 x 550 mm square voids which are lowered over the reinforcement projected from the cast in-situ substructure. These voids are now to be filled with 40 MPa concrete to stitch the elements together.

As shown in Figure 4-24 on page 50, there was a small gap between the top of the column and the underside of the head stock. This gap was filled with dry pack grout to prevent concrete leakage. There was also a gap of approximately 30mm between the abutment and the sill beam, however this gap extended along the full length and depth of the abutment and was impractical to fill with dry pack grout so foam joint backing rod was pushed tight around the base of the voids instead.

There was no additional steel required to be installed into the voids so, now that they were sealed, concrete was poured into all voids in one continuous operation. The concrete pump pumped the concrete into the voids where is was vibrated, screeded and finish from a platform ladder or scaffold as shown in Figure 4-25 and sprayed with a curing compound in the same manner as discussed in Section 4.2.9.

This step was completed on the same day that the blinding layer for the wing wall was poured.

Figure 4-25 - Pouring concrete for column to head stock stitch

4.2.21 Extend retaining wall

As discussed in Section 4.2.19, the wing wall at Abutment B is shown to be overhanging the end of the retaining wall by 2,880 mm. This overhanging portion would have been susceptible to damage during the pavement construction as later discussed in Section 5.3 so it was decided to extend the retaining wall to be the same length as the wing wall. This wall is in addition to the design so was not considered to be a permanent structural element, rather it was a change made to support constructability of the structure and was to have properties equal to or greater than that which would otherwise be provided by compacted general fill. Formwork and falsework was installed in the same manner as discussed in Section 4.2.8 and the retaining wall was poured with unreinforced mass concrete, vibrated and smooth finished.

4.2.22 Install packers to head stocks and sill beams

The next step was to install packers to the head stock to temporarily support the deck modules after installation while waiting for the bearings to be installed. The bearings have a nominal height of 127 mm from the surface of the head stock or sill beam which meant that the packers installed at this step would need to be significantly higher than those installed in section 4.2.16.

Timber packers made from cut pieces of 45 x 90 mm pine were installed and topped with compressed plastic packers to design height at the location of the webs for each double-tee deck unit. At the sill beam, each deck flange was supported on its’ own packer, whereas for the head stocks each deck flange shared a packer with its’ adjacent unit. The deck packers were installed to within ±1mm of the finished level, but this level is not required to be exact as the height of each deck and/or packer can be individually adjusted using jacks later in the construction if required. It was more important to ensure that the packers were in the correct location to land the decks in the lateral and transverse planes as adjustment in these directions would not be possible after the crane used for installing the deck units leaves the site.

4.2.23 Install deck units

The next step in the construction of this bridge was to install the precast deck units. Prior to the decks arriving, the construction team marked the centre line of each bearing onto the head stock or sill beam using a chalk line to assist in aligning the deck units during installation. Hold down brackets consisting of cut length of galvanised 310UC118 were installed to the headstock and sill beam to assist in guiding the decks into the correct position.

With these actions completed, the bridge was now ready for the deck units to be installed using the same 100 tonne Liebherr LTR1100 crawler crane as was used for installing the head stocks and sill beams. The mass of the deck units was approximately 15.7 tonnes for the internal modules and 16.6 tonnes for the external modules including handrail. These weights meant that the crane was to setup on the creek crossing and lift all the units from the same location.

The precast elements were delivered to site on standard semi-trailers with no oversize restrictions as is an important characteristic of the CBS system. Hardwood timber packers were installed below each unit and hard plastic edge protection installed over the tie down chains to prevent damage to the concrete. The manufacturer of the units cast tie down holes in the external flanges of each unit to secure the units during transportation.

Each element was delivered on an individual truck and installed in the order shown in Table 4-1. The order was chosen based on continuous progression from one side the bridge to the other starting with span one, but it could be reversed to start at span three if required with no adverse impacts.

Table 4-1 - Delivery and installation order for precast deck modules

Truck 1	External module
Trucks 2 and 3	Internal module
Trucks 4 and 5	External module
Trucks 6 and 7	Internal module
Trucks 8 and 9	External module
Trucks 10 and 11	Internal module
Truck 12	External module

Once arrived on site, each truck reversed down to the crane and was unloaded. The precast elements all had four five tonne lifting anchors cast into the concrete to which standard lifting knuckles were attached as shown below in Figure 4-26.

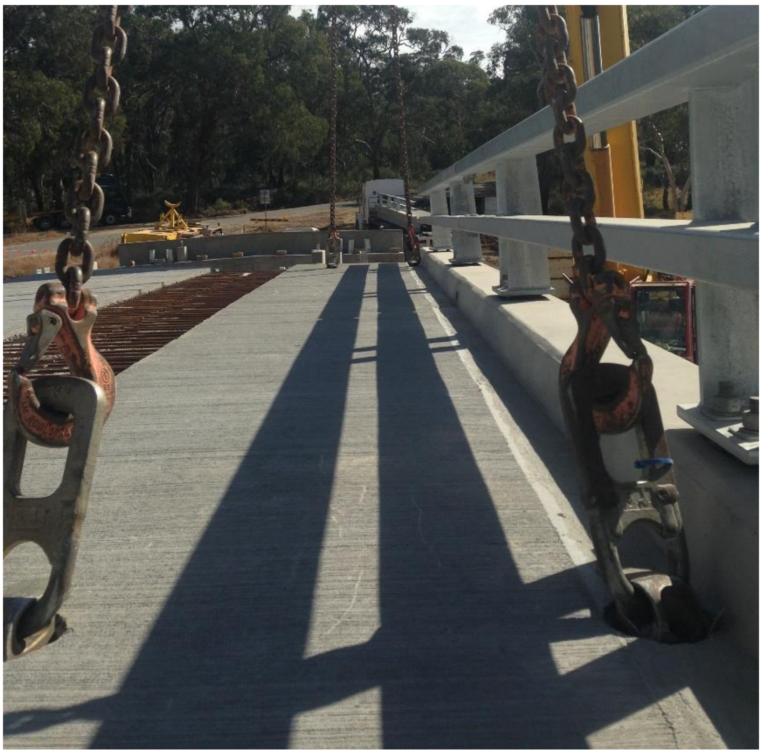

Figure 4-26 - Lifting arrangement for precast deck module

The precast elements were then lifted clear of the truck to allow the truck to leave the site. Once suspended and given authority to enter the area by the crane operator, the construction team then used a chalk line to mark the centre line of each of the deck webs. These lines were marked by standing either side of the each deck unit as shown in Figure 4-27, at no stage should any person be located underneath any suspended element due to the risk of crush injury including death should the load fall.

Figure 4-27 - Marking chalk line on precast deck module

After the elements were accurately marked they were ready to be moved into the required span. The crane stayed stationary while loaded and the operator swung each element to above its final location and began to lower it into position. The dogman used a tag line to adjust the position of the element during lowering of each unit, taking particular care to ensure that the webs of the deck fitted neatly and evenly between the hold down brackets as shown in Figure 4-28.

Figure 4-28 - Lowering an internal deck module

The first module installed was an external unit to span one. There is a 30 mm joint between the sill beam curtain wall and the edge of the deck unit, so a piece of 45 x 90 mm timber was cut to have the bottom 600 mm (approx.) as 30 mm thick and used to set the joint distance as shown in Figure 4-29.

Figure 4-29 - Setting joint gap between deck module and Abutment

The deck module was then lowered into its' final position and, after aligning the chalk lines, load was released and the deck was fully supported on the packers. Fox wedges were then installed as shown in Figure 4-30 to provide additional support.

Figure 4-30 - Internal deck module installed

The crane then unhooked from the deck and the process was repeated until all of the remaining precast deck units were installed. Installation of each deck took less than 30 minutes, with the entire step being completed within five hours. The crane remained onsite to install the precast wing walls the next day.

The hold down brackets were then installed to provide temporary restraint of the deck units until the deck stitch was poured. The hold down brackets were removed a few times during later bridge construction which resulted in snapping of some of the restraint bolts and is discussed further in Section 5.20.

The external deck units were delivered to site with the traffic barrier already installed which created the basis of an edge protection system. The barrier was unable to function as legal edge protection in its own right as discussed in Section 5.14, however it did provide some protection which provided immediate safety benefits for the construction crew, particularly during unhooking of the lifting clutches from the external deck units after installation.

4.2.24 Install wing walls

The next action undertaken was installation of the precast abutment wing walls. Prior to the trucks arriving, plastic packers were installed to design level and the target perimeter marked with a chalk line as shown in Figure 4-31.

Figure 4-31 - Setting packers for placement of wing wall

The wing walls have a void containing horizontally projecting reinforcement which is designed to fit over the reinforcement projected from the sill beam and the footing. In anticipation of reinforcement clashes, the reinforcement projected from the sill beam was bent away slightly as shown in Figure 4-32 to increase clearance between nearby bars during installation of the wing wall.

Figure 4-32 - Adjusting sill beam projected reinforcement

The wing walls were delivered and unloaded in a consistent manner to all other precast elements for this bridge, the only difference was that due to their lesser weight they were able to be transported at a rate of two elements (one complete side) on each truck. The crane was still set up on the creek crossing from the installation of the deck units in the previous day. The crane lifted the first wing wall off the truck and moved it above Abutment A in preparation for lowering into its' final position. The wing wall was then lowered into position as shown in Figure 4-33.

Figure 4-33 - Lowering precast wing wall into position

The positioning of the wing wall was controlled by aligning the perimeter of the element with the chalk lines marked on the abutment slab and blinding layer as discussed earlier in this step. A centreline was not marked as the unit was not symmetrical so the centreline was difficult to determine and may have been marked inaccurately and result in installation error. Once lowered, the alignment was adjusted as needed using levers as shown below in Figure 4-34.

Figure 4-34 - Adjustment of precast wing wall position

After positional adjustment had been completed the wing walls were ready to be temporarily secured to allow the crane to release the load. Blocks of timber were screw bolted to both sides of the wing wall and an acrow prop installed on both sides and a 45˚ angle (approx.). The base of the acrow prop was braced using a piece of timber secured against star pickets (where the ground was gravel) or screw bolted to the abutment footing slab as shown below in Figure 4-35.

Figure 4-35 - Bracing of placed precast wing wall

The bridge was beginning to take shape at the completion of this step as shown below in Figure 4-36.

Figure 4-36 - Site after installation of deck modules and wing walls

4.2.25 Tie wing wall stitch reinforcement

After the wing walls were placed, the reinforcement projecting from the sill beam was lapped with the reinforcement projected from the wing wall using plain steel tie wire. N16 U-bars were placed over the now lapped reinforcement projected from the sill beams and wing walls and lapped to the reinforcement projected from the footing.

4.2.26 Pour wing wall stitch

With the wing walls now placed and secured and the reinforcement tied, the next step was to pour concrete into the stitch area between the Abutment footing, sill beam and wing wall. Each stitch pour only used 0.6 m^3 (approx.) of concrete, so it was considered uneconomical to have the concrete pump return to site for such a small job and, as such, the concrete was placed using a kibble lifted from the bridge truck as shown in Figure 4-37 instead.

Figure 4-37 - Placing wing wall stitch concrete

The concrete was placed and vibrated in one continuous operation to ensure that no cold joints would form. The concrete was then smooth trowel finished immediately after placement and again after reabsorption of the bleed water after which the Fosroc ConCure curing agent was applied to mimic wet curing.

4.2.27 Install bearings

The next step in the construction of this bridge was to install the bearing pads, plates and dowels. Each deck is supported on four individual laminated elastomeric bearings underneath the web or "tee" of the deck in the arrangement shown in Figure 4-38. At the piers, the bearings are all located away from the stitch area with the columns which meant that the recesses for the bearing pins were precast into the sill beam prior to delivery to site. At the abutments, the bearings beneath the two internal deck units are located within the stitch area with the footing which meant that the recesses had to be drilled on site once the stitch concrete had cured as discussed in Section 5.16.

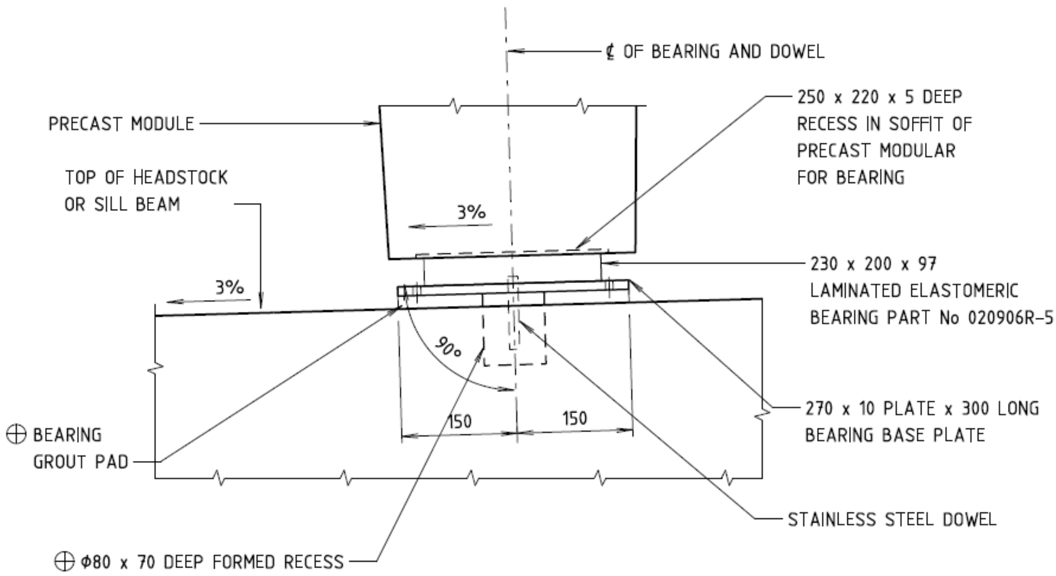

Figure 4-38 - Bearing layout

The bearings were intended to be installed before the deck units, however this was unable to happen as the bearing plates were not welded to the bearing plates. This would have meant that the embedment depth into the head stock or sill beam would be difficult to control as later discussed in Section 5.17.

In response to this issue, the bearing pins were welded to the bearing plates on site with a full length butt weld before being placed over the void in the head stock or sill beams.

The plates were then lifted slightly and 30 mm long DN10 nylon screws installed below the plate to provide height adjustment. The plate was then lowered completely using the screws, the bearing installed on top of the plate and then the whole assembly was raised back up using the screws to be in contact with the soffit of the deck.

4.2.28 Install bearing formwork

Once all of the bearing componentry is installed, the next step was to install formwork prior to grouting the bearings. Due to the chosen location of the packers and the limited space between the back of the bearing plate and the abutment curtain wall, the installation of this formwork was a time consuming and access restricted activity as discussed further in Section 5.18.

Individual deck units were jacked up and the packer temporarily removed to allow access to install the formwork. Leftover pieces of form ply were cut to size and installed tight around the bearing plate on three sides and offset by 40 mm on the uphill or higher side as shown in Figure 4-39. The outside edges were then sealed with silicon to prevent leakage of the grout.

Figure 4-39 - Bearing formwork

The purpose of the 40 mm gap was to allow additional area for air to escape and allow the construction team to easily monitor the depth of the grout in the next step. This worked about 70% of the time, however in about 30% of the bearings the grout did not penetrate the whole way under the bearing plate and needed to be injected with structural epoxy filler to ensure that the bearing plate was fully supported.

4.2.29 Seal gap between deck flanges

The deck units included non-structural 75 mm thick mass concrete flanges protruding from both sides of the internal modules and the internal side of the external modules. These flanges run the full length of the deck units (excluding the end diaphragm soffit) and act as permanent formwork during pouring of the deck close strip.

The drawings show a 30 mm nominal gap between adjacent units – on site gap varied between 30-60 mm – which needs to be sealed prior to pouring concrete. To seal this gap the edge of the flanges were painted with bitumen primer and bitumen impregnated tape (BITAC tape) was applied as shown in Figure 4-40.

Figure 4-40 - Sealing gap between deck module flanges

The flanges were rough finished to promote adhesion of the later poured concrete, however this resulted in difficulties in sealing the gap between units as discussed later in Section 5.10 . In some areas, the finish was so rough that the tape needed to be applied over thick silicon sealant to allow adequate adhesion.

4.2.30 Pour bearing grout

The bearings are supported by a 20 mm thick (nominal) grout pad between the top of the head stock or sill beam and the underside of the bearing plate. Once the formwork was installed, Renderoc BB bearing grout was mixed with water and installed via a gravity assisted funnel and tube arrangement from the bridge deck as shown in Figure 4-41.

Figure 4-41 - Pouring grout to bearings

A member of the construction team was below the deck working from scaffold and communicating with the team members on the deck about how much grout was required. The form work was regularly and vigorously tapped to encourage proper distribution of grout below the bearing plate to reduce the potential for them to be drummy (lacking full grout penetration) until grout covered filled the 40 mm area between the bearing plate and the formwork and covered the top of the bearing plate on the uphill side.

4.2.31 Remove formwork and load transfer

Once the grout for the bearing pads had reached sufficient strength (five days were allowed, the formwork was removed and the grout sprayed with curing compound. The decks were raised slightly using hydraulic jacks, after which the packers were removed and the deck lowered back down to be supported entirely on the bearing pads. This process transfers the load from the packers to the bearings, hence it is named load transfer.

4.2.32 Shear keys

This bridge has 12 shear keys which join the head stock or sill beam to the deck unit to provide lateral load restrain and resist transverse loads (shear). At the head stocks, the shear keys were in the form of 480 x 800 x 250 mm concrete plinth with two protruding N30 stainless steel dowels as shown below in Figure 4-42. The arrangement at the sill beam is similar, consisting of a 480 x 430 x 250 concrete plinth with a single protruding N30 stainless steel dowel.

Figure 4-42 - Shear key at Pier One

The stainless steel dowel(s) from the shear key protrudes through the deck soffit into the stitch area. After installation of the deck units, the protruding dowel was wrapped in Abelflex polyethylene foam and over which a 50mm diameter PVC cap was installed. This arrangement is different to the rubber ring in 60 x 5.4 mm CHS arrangement shown in the drawings, but was considered to be equivalent as the role of the shear keys is to provide shear restraint (load transfer and resist movement) for the decks, so the CHS would essentially be acting as formwork (same as the PVC cap) with the rubber rings allowing small amounts of movement (same as the Abelflex).

4.2.33 Tie deck steel

The next step in construction of this bridge was to tie the reinforcing steel in the deck closure strip area between adjacent deck units. The precast modules had two layers of N12 reinforcement projected at 120 mm centres from the side of each module directly above the concrete flange as shown in Figure 4-43. The last 450 mm of each module did not have a concrete flange and contained two layers of N16 transverse reinforcement spaced at 120 mm centres as shown in Figure 4-42 on page 67.

Figure 4-43 - Projected reinforcement above deck module flanges

The projected reinforcement above the precast flanges was connected together with 800 mm long N12 transverse reinforcement with minimum 350 mm lap length. A total of 1,350 stitch individual bars were tied during this step, an exercise that took about 60 man hours. Installing the bottom layer of longitudinal reinforcement was space constrained and is discussed further in Section 5.11. The final result is shown in Figure 4-44.

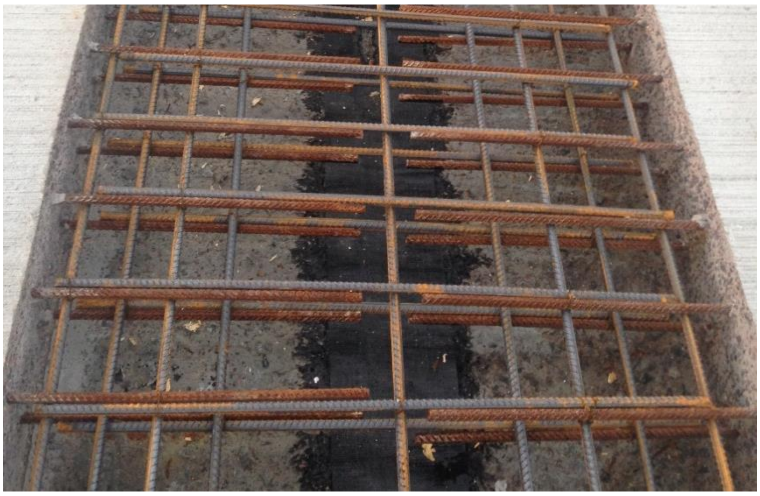

Figure 4-44 – Completed reinforcement above deck module flanges

The projected reinforcement within the end diaphragm was connected together with 800mm long N16 transverse bars before being encapsulated by N16 ligatures as shown below in Figure 4-45. This area is quite congested and is discussed in further detail in Section 5.13,

Figure 4-45 - Completed reinforcement in end diaphragm

4.2.34 Soffit formwork and falsework

As briefly mentioned in the previous step, the precast concrete flanges on the deck units stop 450 mm before the end of each module, with the end diaphragm section being cast in-situ. Formwork for the soffit in this area is required to retain the concrete and maintain dimensional conformity with the design.

The formwork and falsework was installed from below the deck and consisted of a single sheet of form ply braced off the top of the headstock or sill beam as shown in Figure 4-46. The edges of the formwork were sealed with silicon to prevent leakage of the concrete. During installation of the formwork, it was difficult to maintain the design cover from the ligature to the soffit. This difficulty is discussed further in Section 5.8.

Figure 4-46 - End diaphragm soffit formwork

4.2.35 Pour and finish deck concrete

With the steel reinforcement installed and the formwork complete, the next step is to pour the longitudinal closure strip between the deck units and the transverse joints between spans. It was mid-June by the time this stage was reached, so pouring started early in the day to make to most of the comparatively warm temperature to assist the concrete in reaching initial set before the temperature started dropping again overnight.

The finished deck has a 3% cross fall both ways with the apex on the centreline. To assist in creating the apex, a vibrating screed with 3% cross fall both way was manufactured and a pine guide rail bolted to the deck to run the screed along. Additional labourers were also engaged as the rate of concrete delivery was expected to result in a large and busy working area.

The concrete pump arrived and set up to receive concrete consistent with every other pour in this construction project. Concrete was pumped into the working area and vibrated and screeded into position.

Screeding the outer longitudinal closure strips was a relatively simple task as the cross fall was consistent so it was able to be screeded straight across, however finishing the centre apex was a lot more difficult as discussed in Section 5.12. The custom manufactured screed did not work as intended, so the construction crew removed the screed from the vibrating arms and used it to set the target profile every the metres (approx.) and hand screeded in between as shown below in Figure 4-47.

Figure 4-47 - Screeding deck central closure strip including cross fall apex

Each span is separated by cast in-situ full width transverse joints poured and finished in the same manner as the longitudinal closure strips. The purpose of the joint is to allow independent difference movement (expansion and contraction) of each of the spans; movement which requires a 25 mm gap to be present between adjacent spans. This gap was achieved by installing Styrofoam between the spans against which concrete was poured as shown in Figure 4-48, however this resulted in a joint that was not formed consistently straight as discussed in Section 5.19.

Figure 4-48 - Screeding transverse closure pour near expansion joint

The alignment issue was anticipated and the methodology changed to pour the two end spans first and fix the Styrofoam to a 45 x 90 mm length of timber in an attempt to hold the joint straight during the pouring of the end spans. Once the two end spans had been poured and the pouring of the central span had almost reached the joint, the timber was removed as shown in Figure 4-49 to allow concrete to be poured to the other side of the joint. The timber was left in place for as long as possible to minimise distortion of the joint.

Figure 4-49 - Removing timber formwork brace from transverse joint

After the concrete had been vibrated, screeded, bled and screeded again, it was time for finishing. The concrete just poured is part of the final running deck so it was rough broom finished to provide a slip resistant surface for traffic. As the name suggests, this method of surfacing is achieved by running a broom across the concrete to roughen the finish as shown in Figure 4-50.

Figure 4-50 - Broom finishing deck concrete

The final action in the step was curing of the concrete. The surface of the finished concrete was sprayed with Fosroc ConCure in the same manner and application rate as discussed in Section 4.2.9. Application of the curing agent served to mimic the wet curing process by minimising water loss, however it would not provide any protection against the cold weather experienced by the site in mid-June. To retain the heat of the concrete and reduce potential for damage due to cold weather, the finished concrete was covered with hessian, sarking and black plastic which was weighed down with lengths of timber as shown below in Figure 4-51.

Figure 4-51 - Thermal protection of deck closure strip concrete

4.2.36 Strip soffit and joint formwork

After the deck had been allowed sufficient curing time (minimum 48 hours), the formwork from the soffit was removed from below the bridge. Access to the soffit was gained by the use of mobile scaffold, and the timber falsework was removed followed by the plywood sheeting as shown in Figure 4-46 on page 70. Although curing was well underway and the risk of moisture loss through the exposed concrete surface was highly unlikely, a curing agent was applied to the stripped surface as a precautionary measure.

The joint formwork consisted of three independent layers of Styrofoam as discussed in step 4.2.35. To remove this formwork, the central layer of Styrofoam was cut and removed which loosened the external layers and allowed easy removal.

4.2.37 Install backing rod and joint epoxy

The transverse movement joints consist of a 25mm gap between adjacent spans into which a length of neoprene backer rod is installed and overlain by an elastomeric sealant as shown in Figure 4-52. After stripping of the joint formwork in step 4.2.37, the joint was cleaned to remove any loose material and a foam backing rod was compressed and inserted into the joint gap. Dow Corning 902 RCS joint sealant was then installed using a standard sealant gun and tooled to a finished level 5mm below deck level.

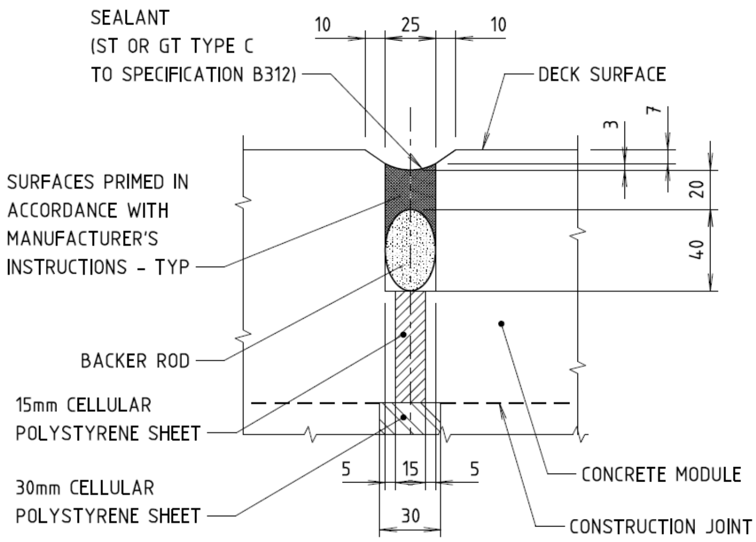

Figure 4-52 - Joint design

4.2.38 Adjust height and install grout under traffic barrier

The traffic barrier was dummy fitted to the external deck modules by the precast deck module manufacturer prior to delivery to site as briefly discussed in step 4.2.23. At this stage of construction, the bearing had been set and load transferred and the deck closure strip had been poured so any variations in deck hog had already occurred which meant that the kerb alignment was in its' final position.

The low performance traffic barrier consists of a galvanised steel upright posts at 2.50 m spacing connected by horizontal galvanised steel RHS. The barrier connected is to the kerb of the external unit by bolts through the barrier base plate as later discussed in step 4.2.42. The bolts have a nut above and below the plate which were used to adjust the height of the installed barrier, after which sleeved RHS connection pieces were used to connect adjacent railings.

Formwork was the installed around the perimeter of the base plate and filled with Renderoc BB grout. Once initial set had been reached, the formwork was removed and the traffic barrier was complete as shown in Figure 4-53.

Figure 4-53 - Completed traffic barrier

4.2.39 Tighten bolts and install grout under hold down brackets

The hold down brackets were loose fitted prior to installation of the precast deck modules as briefly mentioned in step 4.2.23. The deck modules are now in their final alignment, so the bolts were tightened but this resulted in snapping in two of the bolts as discussed further in Section 5.20.

The design shows a 33 mm gap between the underside of the traffic barrier base plate and the top of the head stock or sill beam. Formwork was installed around the perimeter of the base plate and filled with Renderoc BB grout. Once initial set had been reached, the formwork was removed and the hold down brackets were complete.

4.2.40 Install grout between abutment footing and sill beam

The top surface of the abutments was constructed 10 mm lower than design height to provide greater flexibility during installation of the sill beams as discussed in step 4.2.9. The design shows a nominal gap between the sill beam and the abutment footing of 22 mm, however with the additional gap this was closer to 30 mm. Leaving this gap open would not only look unsightly, but it would also cause water from behind the abutments to leak over the front of the abutment which may result in accelerated concrete degradation.

To close this gap, the join between adjacent sections on the back of the abutment was painted with primer and the gaps sealed with bitumen impregnated tape as shown in Figure 4-54.

Figure 4-54 - Sealing the back of the Abutment

Foam backing rod was then pushed approximately 20 mm in front the front face of the gap and dry pack grout installed for aesthetic reasons. At the end of the step, the gap was sealed smooth as shown below in Figure 4-55.

Figure 4-55 - Completed Abutment and wing wall face

4.2.41 Fill deck lifting points

The precast deck modules were lifted and installed using the lifting points as previously shown in Figure 4-26 on page 54. These lifting points were located below deck level and holding rainwater, so the first action in this step was to dry and clean the lifting points. Renderoc BB grout was then used to fill the recess around the lifting lug. Figure 4-56 shows two open lifting points and two partially grouted lifting points.

Figure 4-56 - Grouting of deck lifting points

4.2.42 Dummy fit traffic barrier to wing walls

The traffic barrier extends beyond the bridge deck to the end of the wing walls where it joins a traditional thrie beam railing by means of a connection plate. Installation of the thrie beam railing needs to take place after the completion of the approach pavement works so it would be impractical to permanently set the level of the traffic barrier on the wing wall when the connection level was not yet known.

In this step, the traffic barrier was lifted using the Hiab crane on the bridge truck and dummy fitted to each wing wall at the same general level as the barrier on the bridge. The bolts below and above the base plate were tightened to hold the barrier in this temporary position but the area under the base plate was not grouted. Taking this action not only meant that the barrier height could be adjusted as needed, it also made the barrier less likely to be stolen after site disestablishment as it was more difficult to access.

4.2.43 Install name plate

The final construction step was to install the bridge name plate. The name plate was made of brass by local supplier Phoenix Foundry in Uralla. Four holes were drilled in the kerb of the southbound external deck unit 300 mm (approx.) in from Abutment A to align with the holes in the nameplate. Construction adhesive and a M12 bolt was then installed into each hole as shown in Figure 4-57. When the construction adhesive had set, the bolts were tightened using a spanner making the hex head on top of the bolt shear off to leave a round imitation rivet finish.

Figure 4-57 - Bridge name plate

4.2.44 Site disestablishment

The final step in any construction job is to disestablish from the construction site. This step involved the removal of all sheds, material storage, ablution facilities, leftover materials and general construction waste from the site. Environmental controls such as sediment fences and coir logs were left behind to assist in stabilisation of the site consistent with standard industry practice. The creek crossing was left in place to assist in the later pavement works but will be removed when those works are finished.

The bridge itself was now complete and ready for commencement of approach pavement works by others.

4.3 Conclusion

This chapter has described the considerations, methodology and techniques used to construct the bridge over Bookookoorara Creek from the spread footing to placement of deck units a site disestablishment. A number of issues and areas of improvement were identified and will be discussed and analysed in the following chapters.

5 Issues identified during construction

This section contains a register of the issues raised during construction of the bridge and provides a brief discussion of each.

5.1 AFT001 / PFT001: Spacing of reinforcement in footings

The steel reinforcement cage for both Abutment footings and both Pier footings consists of a mix of N16 and N20 deformed bar at 200mm spacing. The cages were constructed on site, with all steel bent to fit with a handheld bender due to the highly variable rock location. Workers were required to walk on, over and around the cage during construction, with the distance to the ground below the cage ranging from 40mm (lower layer of all footings) to 3.32 metres (upper layer of Abutment A). As shown below in Figure 5-1, the 200mm spacing interval creates open holes that are large enough for a workers boot to slip through resulting in personal injury.

The response to this issue during construction was to provide timber planks and boards in set locations to create set walking routes over the cage. This removed the hazard and mitigated the risk in the defined walking locations, but the hazard was still present where planks or boards were not placed.

Figure 5-1 - Spacing of reinforcement

5.2 AFT-002: Width of Abutment B retaining wall

The design for Abutment B on this bridge consists of a cast in-situ reinforced concrete spread footing and 450 mm wide retaining wall which supports a 980 mm wide precast reinforced concrete sill beam. The width difference of the retaining wall and the sill beam results in the sill beam overhanging the retaining wall by 225mm on the front face and 305mm at the rear. The overhang area at the front face would be visually inconsistent with the appearance of Abutment A, however the construction issue associated with these different dimensions occurs in the back face. The abutment is to be backfilled with granular material to form a flexible pavement as is typical of walled abutments on local roads, but the overhang area would make it near impossible to adequately compact the material below the curtain wall. This inadequate compaction would likely lead to excessive settlement and/or of the approach over time. This design arrangement and issue is shown below in Figure 5-2.

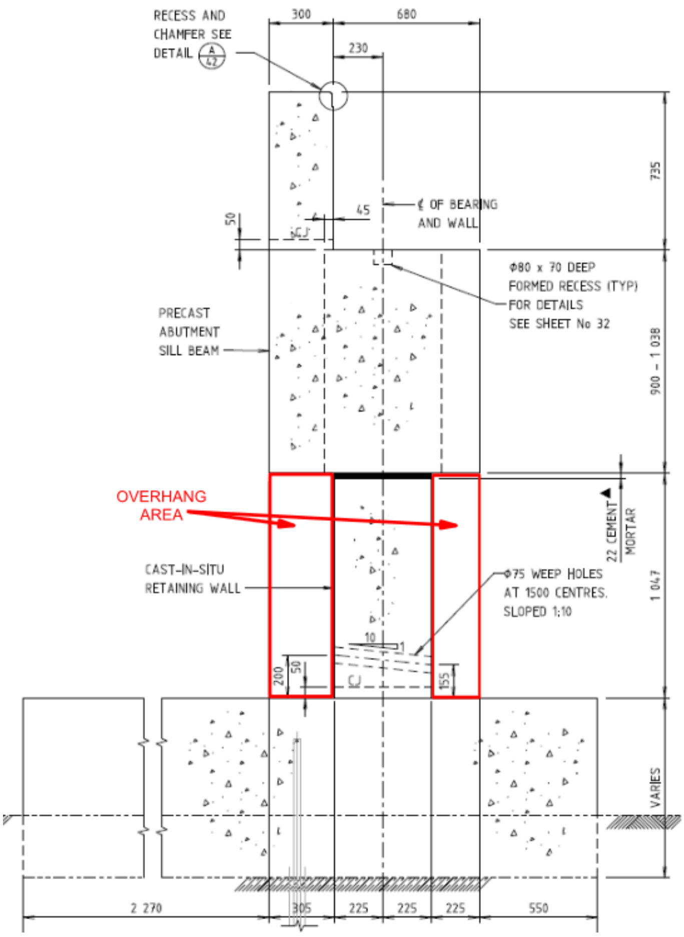

Figure 5-2 - Inadequate depth of Abutment B retaining wall (RMS [annotated], 2016)

5.3 AFT-003: Depth of Abutment B retaining wall

As discussed in Issue AFT-002, Abutment B includes a cast in-situ reinforced concrete retaining wall. This retaining wall extends 2575 mm behind the sill beam to provide support for the precast wing wall. In the initial consideration of constructability during the design phase of this project it was considered practical that, after construction of the retaining wall, earthworks would commence to place and compact material to the level of the top of the retaining wall to provide full length support for the wing wall.

Mobilisation of earthmoving plant is an expensive exercise and would have resulted in approximately one week added to the bridge construction program. As such, it was not considered to be economically or logistically feasible to place and compact this material before placing the wing walls. This would result in the wing wall being unsupported and potentially unstable if placed per the design drawings as shown in Figure 5-3.

The response to this issue during construction was to construct a mass concrete extension of the retaining wall to provide support for the wing wall. The extension needed to have similar or greater compressive strength characteristics as compacted granular gravel so no steel reinforcement was required.

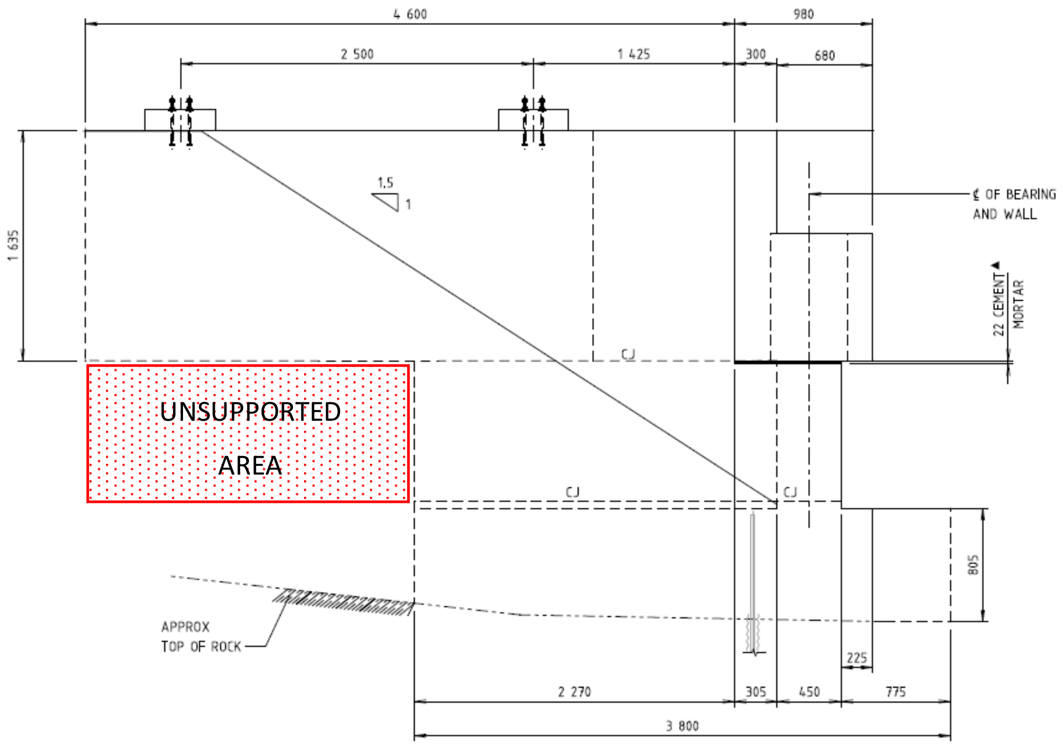

Figure 5-3 - Unsupported wing wall at Abutment B

5.4 AFT-004: Abutment A drainage

Drainage provisions are required to drain water thereby relieving hydrostatic pressure that may lead to eventual failure of walled abutments. Abutment A has no provision for drainage which may lead to future maintenance issues, whereas Abutment B incorporates 4 x 75mm diameter weep holes in to cast in-situ retaining wall. This issue is shown below in Figure 5-4.

A response to this issue during construction was to install drainage sheet (cord drain) and Megaflo subsoil drain to direct water out the end of the wing walls, however potential responses for future constructions are presented in Chapter Six.

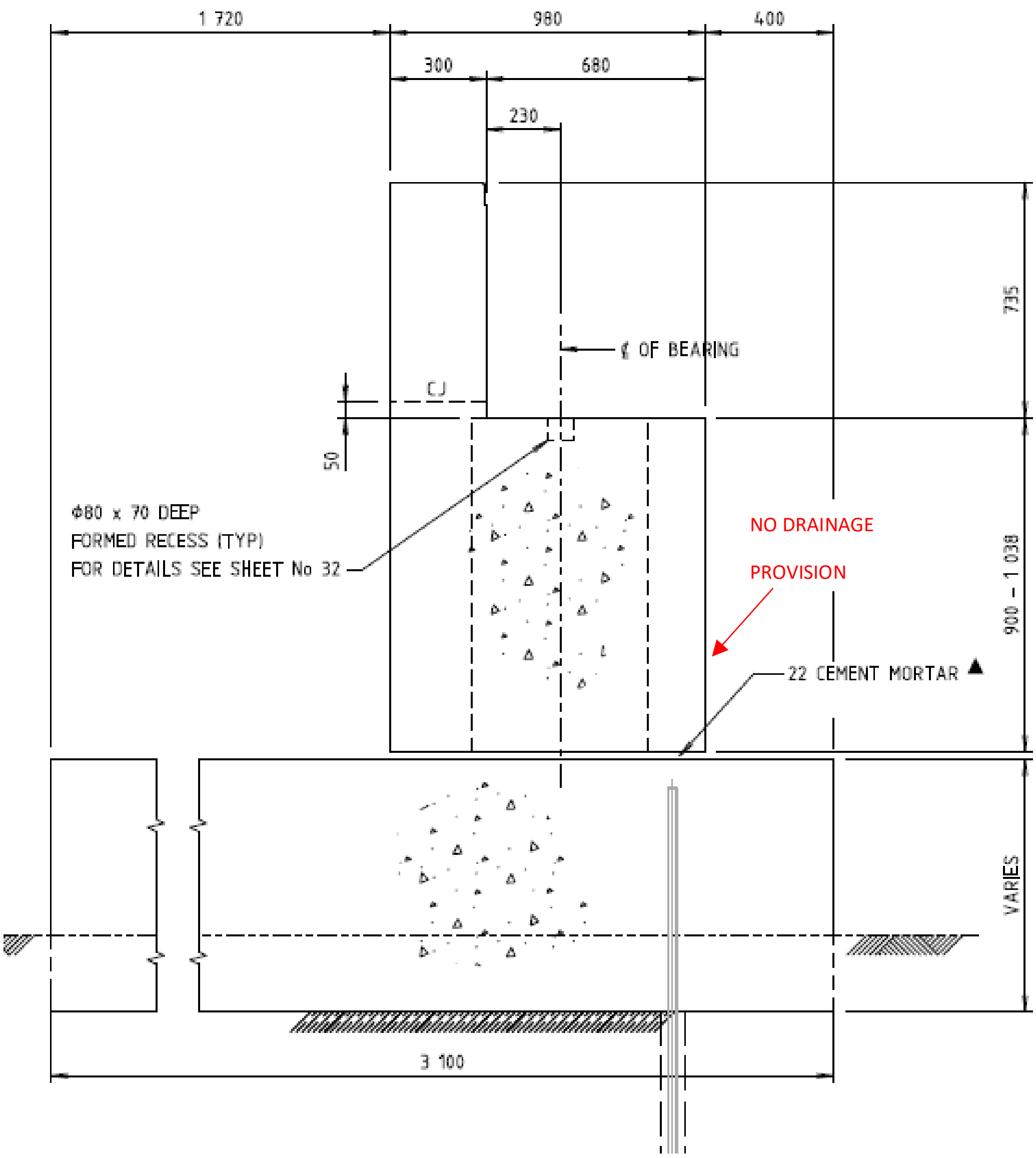

Figure 5-4 - Abutment A layout (RMS [annotated], 2016)

5.5 AFT-005 / PFT-002: Dowel hole size to footings

The pier and abutment spread footings are connected to bedrock by a series of galvanised steel dowels. Holes are drilled into the rock, the dowel is inserted and grouted into position prior to pouring of concrete and provide direct shear connection to the bedrock to prevent sliding of the footing. The design requires N36 galvanised steel dowels in 2050mm deep, 100 mm diameter drilled holes as shown in Figure 5-5.

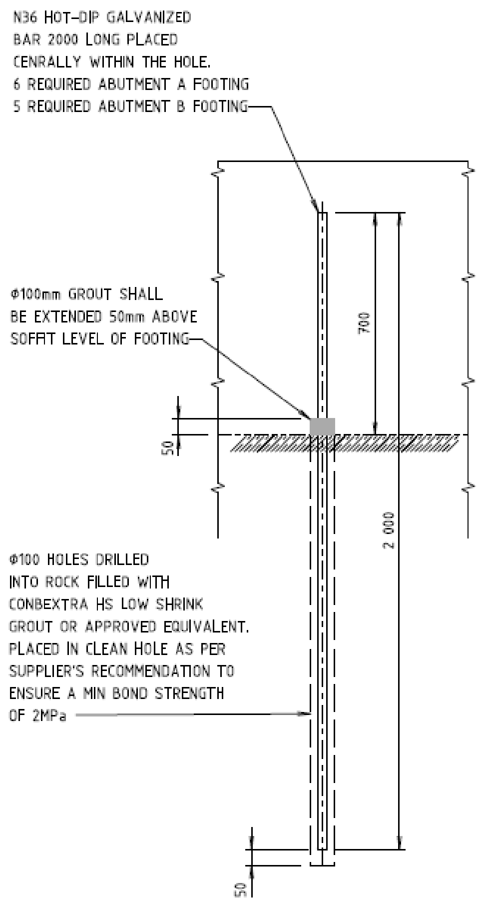

Figure 5-5 - Rock dowels (RMS, 2016)

The construction team were unable to locally source plant capable of drilling a 100 mm diameter hole to the design depth. Instead, the holes were drilled at 50mm diameter and inserted with N24 dowels. To compensate, the number of dowels was increased to provide the same cross sectional area and therefore shear capacity as the original N36 dowels. This and other potential responses for future constructions are presented in Chapter Six.

5.6 SB-001: Potential damage to Abutment Curtain Wall concrete

The Abutment design for the subject bridge consist of a cast in-situ reinforced concrete spread footing which supports a precast reinforced concrete sill beam. The pavement structure of the bridge approach is unspecified, but for this bridge and future constructions of the Country Bridge Solutions system it is likely that a sealed of unsealed flexible pavement will be constructed. It is reasonably foreseeable that the flexible pavement will settle and deform somewhat during service which will result in an uneven transition between the pavement and the bridge. This unevenness will result in high localised wheel impact loads that may cause damage to the concrete in the Abutment sill beam in the locations shown in Figure 5-6. It is not certain that the concrete will be damaged, but there is potential for damage to occur.

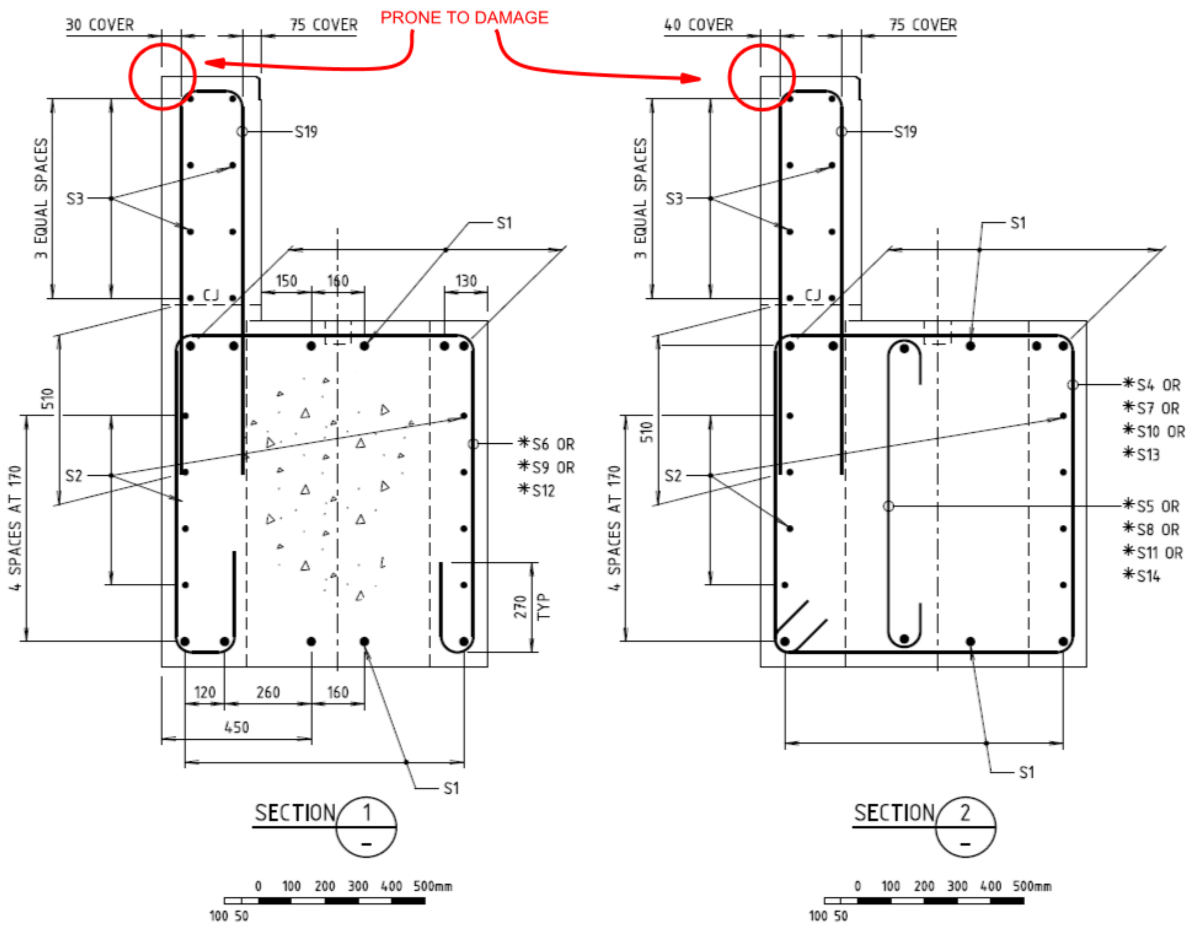

Figure 5-6 - Potential concrete damage to Abutment sill beam (RMS [annotated], 2016)

5.7 SB-002: Roughness of curtain wall running surface

The top surface of the Abutment sill beam forms part of the running surface as discussed in issue SB-002. The approach pavement, whether sealed or unsealed, and the bridge deck are sufficiently rough finished to allow tyres to grip, however the top surface of the sill beam was supplied to site smooth finished. This difference is roughness will create a localised smooth strip of transverse concrete which may result in increased potential for traffic accidents. This risk will not be realised in the trial bridge as the approach and the bridge deck will be sealed which will result in a consistent grip profile, however it is important that this issue is raised for future constructions where the deck may not be sealed.

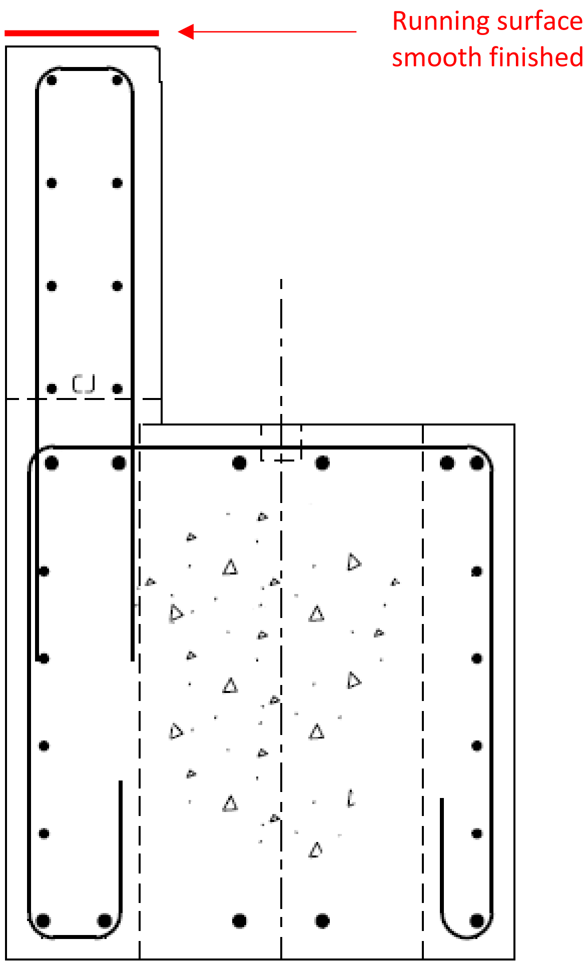

Figure 5-7 - Roughness of curtain wall (RMS [annotated], 2016)

5.8 DMI-001 / DME-001: End diaphragm cover

The deck units are joined by a reinforced concrete longitudinal stitch pour as discussed earlier in this document. At the end of each module, N16 reinforcement projects transversely as shown below in Figure 5-8 around which ligatures are installed (notated as D4 in Figure 5-9). Once steel tying is complete, soffit formwork is installed from below and concrete is poured to form the end diaphragm. The required cover for this element is 40mm (-0, +10), but after installation of the ligatures the maximum achievable cover is approximately 20mm.

A response to this issue during construction was not available (due to the benign environmental of this specific bridge no ongoing maintenance or operation issues are expected as a result of this situation), however potential responses for future constructions are presented in the Chapter Six

Figure 5-8 - End diaphragm projected reinforcement

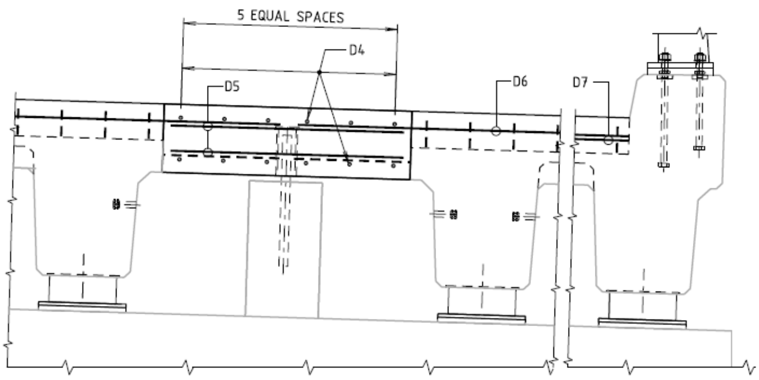

Figure 5-9 - End diaphragm reinforcement layout (RMS, 2016)

5.9 DME-002: Scupper height

The completed bridge has 3% transverse cross-fall in both directions which drains to a series of 125 x 75 x 4 galvanised steel RHS scuppers cast into the kerb on the external deck modules. The scuppers are included as the final drainage mechanism to remove water from the bridge and freely discharge onto the ground or waterway below. Unfortunately, the scuppers as detailed in the design plans are not located at the lower point on the deck as shown in Figure 5-10. It is surmised that the thickness of the RHS was not accounted for during design or drafting which has resulted in the scuppers being built 4mm higher than the finished deck level.

This issue was not realised until delivery of the deck units, at which stage it was too late to implement any form of change during construction. Potential responses for future constructions are presented in Chapter Six.

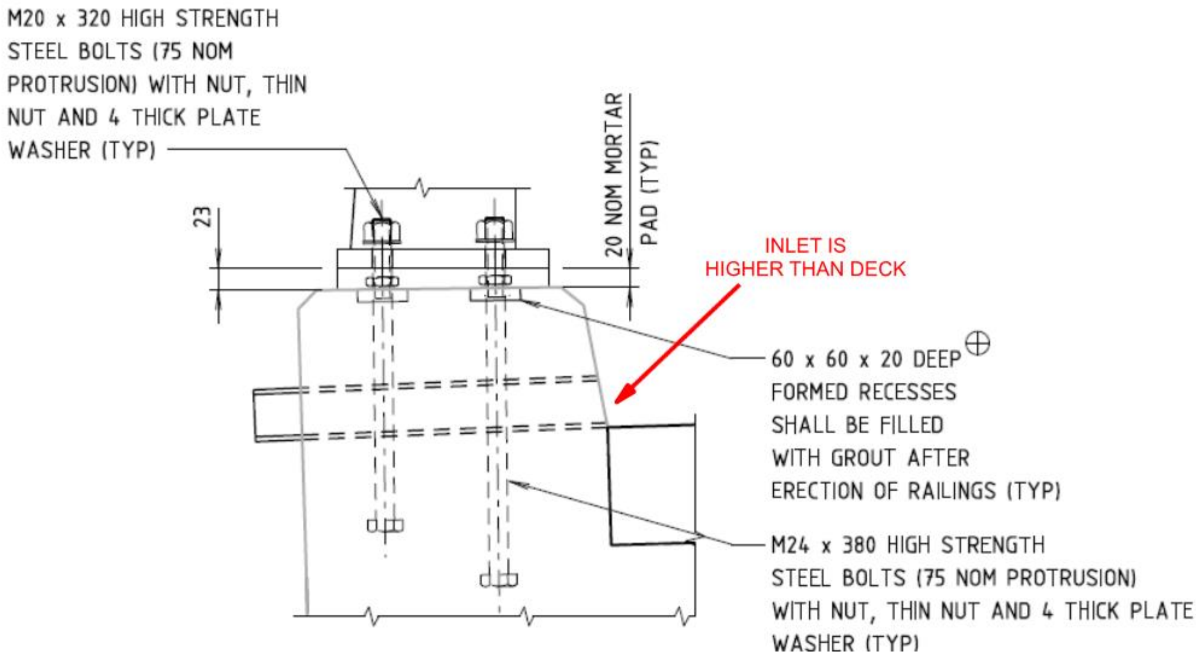

Figure 5-10 - Scupper layout (RMS [annotated], 2016)

5.10 DMI-002 / DME-003: Surface finish of precast deck flanges

The internal and external deck units have 410mm concrete flanges that project out the side of each unit as shown below in Figure 5-11. Concrete is poured in the area above adjacent flanges in order to form the closure strip that stitches the deck modules together. The flanges were rough finished in accordance with RMS B80 Specification to facilitate effective bonding in the formation of a construction joint as required on the design drawings, however the roughness caused issues in sealing the gap between adjacent units prior to placing and compacting concrete. Due to the small gap between units (30mm nominal specified on the drawings), bitumen impregnated tape was used to seal between adjacent flanges. This tape is applied by painting a primer over the application area and pressing the tape into it. Due to the uneven surface, constant adhesion was not obtained which, if left, would have resulted in leakage of the concrete.

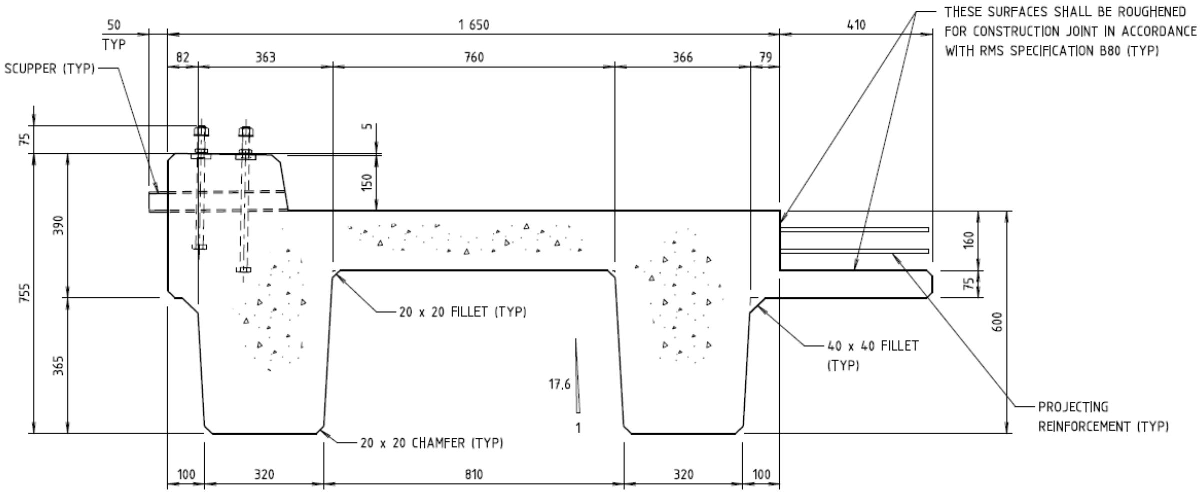

Figure 5-11 - External precast deck module (RMS, 2016)

The response to this issue during construction was to use silicon sealant instead of the specified primer to adhere the tape to the concrete. As the silicon was less viscous that the primer, it was able to be installed thicker and seal the edge more effectively. Other potential responses for future constructions are presented Chapter Six.

5.11 DS-001: Longitudinal deck stitch pour reinforcement

The deck for this bridge consists of four precast prestressed double-tee modular deck units placed side by side and stitched together with cast in-situ concrete closure strips (RMS 2016). The precast deck modules incorporate non-structural concrete flanges and two layers of projected N16 reinforcement running along the full length of each unit. The projected reinforcement acts as starter bars to provide continuity between the closure strip and the deck modules. Once the deck modules are placed, additional reinforcement is tied into position prior to concrete being poured and finished. Due to obstruction by the top layer, it is difficult to install the transverse reinforcement which is required to be tied to the underside of the bottom layer of projected reinforcement. This issue is shown below in Figure 5-12.

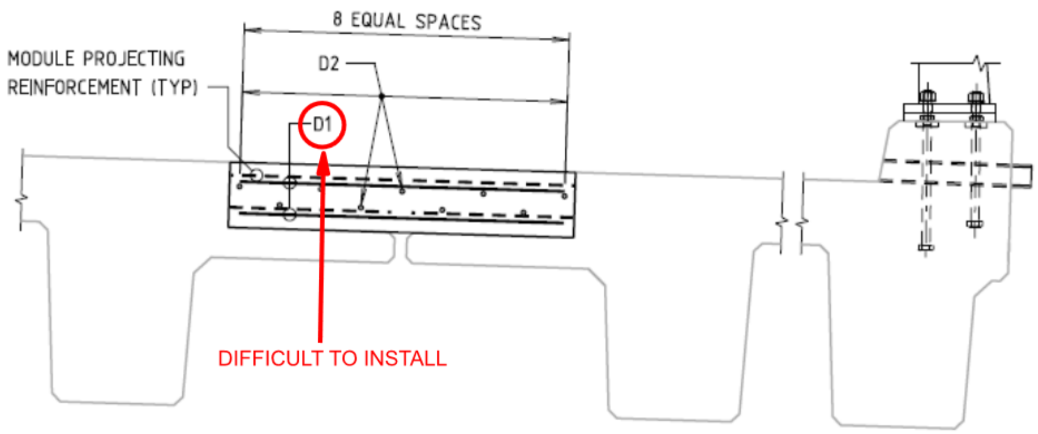

Figure 5-12 - Bottom layer of deck closure strip reinforcement (RMS [annotated], 2016)

5.12 DS-002: Deck cross-fall finishing

The finished bridge has a 3% cross-fall both ways to facilitate drainage of the deck to prevent pooling of water and reduce potential for vehicular aquaplaning. The cross-fall meets at a defined peak along the centre line of the bridge in the central longitudinal stitch pour as shown in Figure 5-13. This peak needs to be constructed and hand finished on site, an activity which may present difficulty for inexperienced works crews.

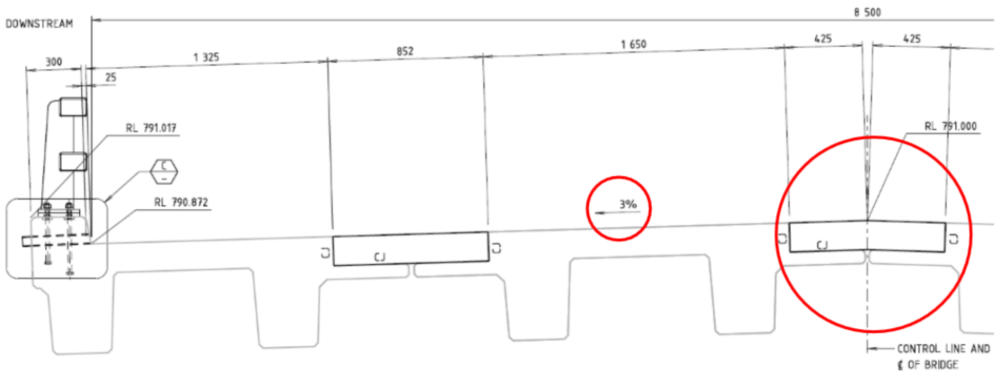

Figure 5-13 - Deck cross fall apex (RMS [annotated], 2016)

A screed with 3% cross fall was manufactured and brought to site .n an effort to achieve a uniform cross fall over the centre deck stitch. This proved ineffective and hand finishing was required, and while the end result was satisfactory (although still with inconsistencies as shown in Figure 5-14) it is likely a result of the skill of the staff rather than the ease of process. Potential responses for future constructions are presented in the following pages.

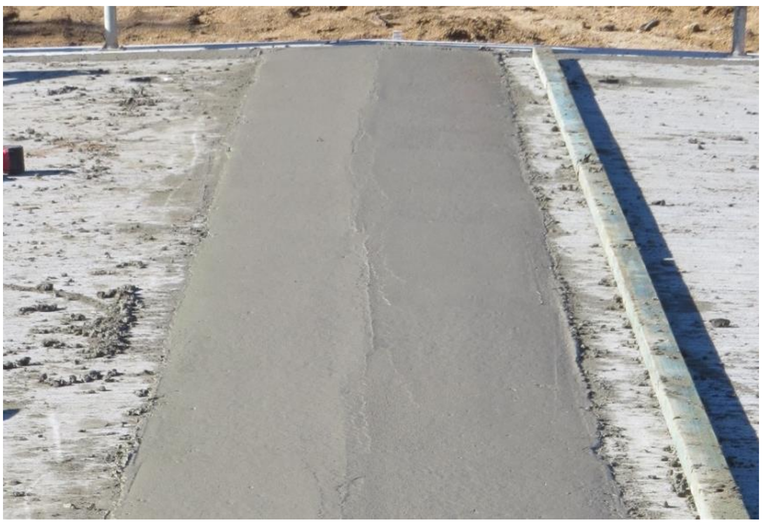

Figure 5-14 - Finishing of central deck closure strip

5.13 DS-003: End diaphragm reinforcement congestion

The end diaphragm reinforcement detail is very congested as shown below in Figure 5-15. When concrete is poured around congested reinforcement it finds it difficult to get around the steel and fill all the gaps, or the gaps fill with slurry only as the aggregate cannot pass through the gaps between the reinforcing steel. This may result in localised areas of weakness in the structure.

Figure 5-15 - End diaphragm reinforcement congestion

A response to this issue during construction was not available during construction but extra care was taken to ensure adequate vibration of the concrete to minimise potential for voids, however potential responses for future constructions are presented in the Chapter Six.

5.14 TB-001: Height of traffic barrier

The design for this bridge includes a 650mm high low performance traffic barrier atop a 150mm concrete kerb on the outside edge of the external modules to provide a top rail 800mm above the deck as shown in Figure 5-16. The barrier was installed in the precast yard and delivered to site pre-attached to the deck units in order to provide the foundation of edge protection under *AS/NZS4994.1:2009 Temporary Edge Protection*, however clause 3.6.2 of this standard requires the top rail of the edge protection to "be located at an effective height above the surface of not less than 900mm".

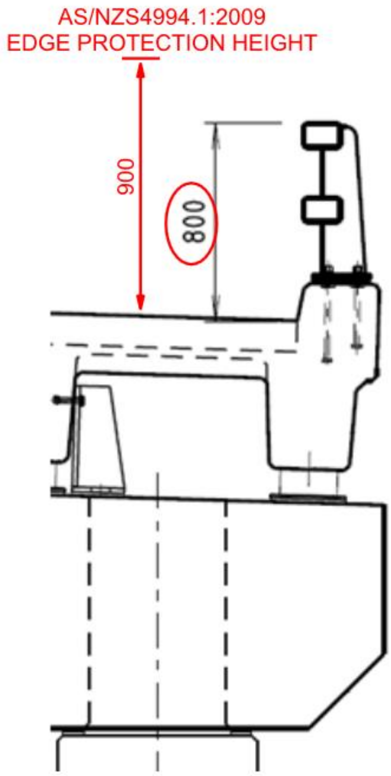

Figure 5-16 - Traffic barrier layout (RMS [annotated], 2016)

5.15 TB-002: Inconsistent bolt and hole sizes in traffic barrier base connection

The traffic barrier is fixed the M20 and M24 bolts projected from the kerb of the external precast deck unit as shown in Figure 5-17. The bolts pass through 22, 26 or 30 mm slotted holes in the base plate of the barrier as shown in Figure 5-18. There are five variables in the fixing arrangement which increases the potential for error during manufacture of the deck unit, manufacture of the traffic barrier and procurement of nuts for the site works. To reduce the potential for error, it would be beneficial to remove as many variables as possible from the fixing detail. A response to this issue during construction was not available during construction.

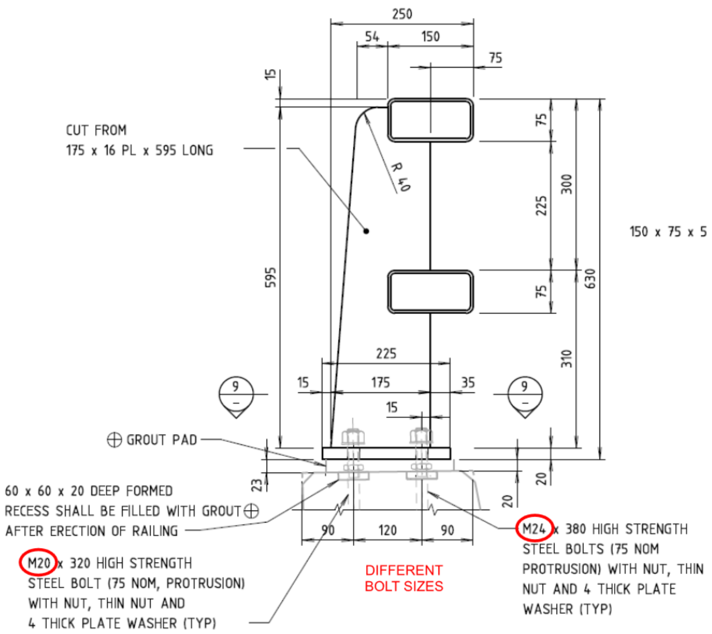

Figure 5-17 - Traffic barrier arrangement (RMS [annotated], 2016)

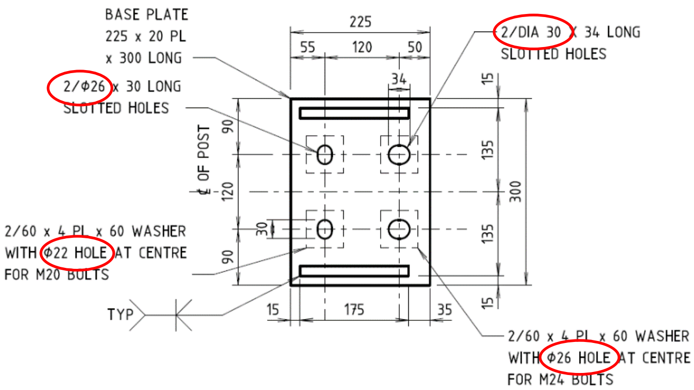

Figure 5-18 - Traffic barrier base plate (RMS [annotated], 2016)

5.16 B-001 / SB-004: Bearing clash with sill beam blockouts

The precast sill beams include 550 x 550 mm blockouts (areas free of concrete) which slide over the projected reinforcement from the abutment footing. Once installed on the footing, the blockouts are filled with concrete to provide connection between the two elements. The bearings sit atop the sill beam, and consist of a 20mm dowel recessed into the sill beam by an 80 mm diameter x 70 mm deep hole, followed by a galvanised steel plate and elastomeric bearing pad. Each sill beam has eight bearings, with the four central bearings being located in the same area as the blockouts as shown in Figure 5-19. This meant that in order to install the bearing, the blockouts had to be filled with concrete and allowed to cure, then a hole drilled and the bearings installed. If the bearing location did not clash with the blockout, the bearings could be installed which the blockout was curing, resulting in a time and cost saving.

A response to this issue during construction was not available during construction, however potential responses for future constructions are presented in Chapter Six.

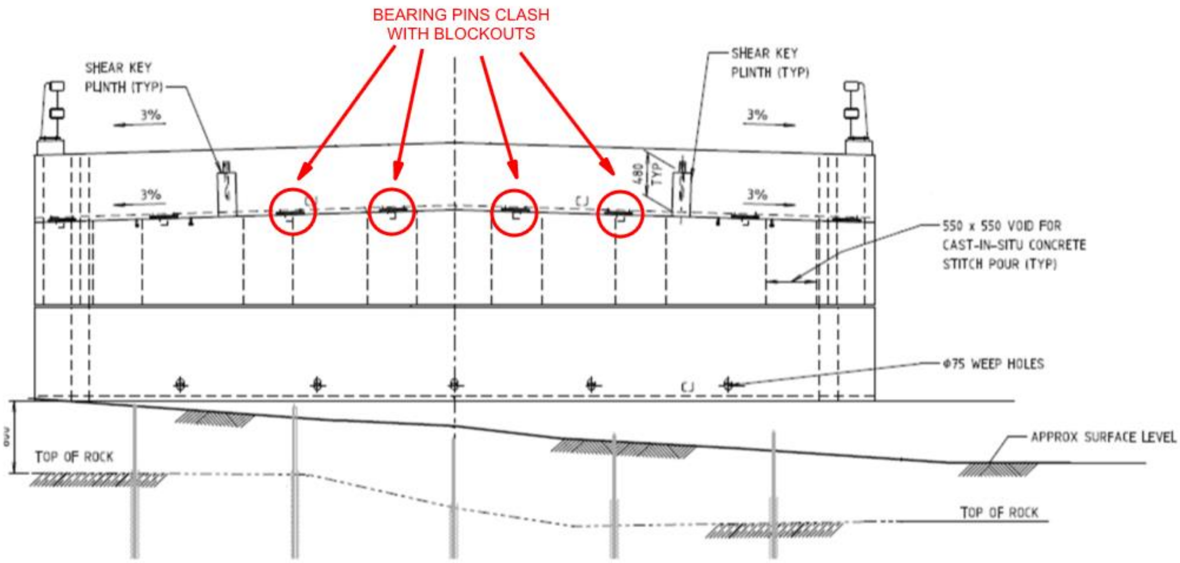

Figure 5-19 - Bearing and blockout clash (RMS [annotated], 2016)

5.17 B-002: Bearing pins not welded to bearing plate

The bearing arrangement includes a 20mm diameter stainless steel dowel which is cast into a recess in the headstock/sill beam to provide the shear connection as shown in Figure 5-20. The drawings specify a minimum 50mm embedment into the recess with 20mm cover below the dowel, as well as minimum 10mm embedment into the bearing pad.

The dowels are typically welded to the base plate, but the drawings for this bridge showed them as being independent which made it difficult to stop the dowel from falling into the recess and compromising the required 20mm cover depth. This issue was noticed during construction after the bearing components had arrived which resulted in the deck and bearing installation process occurring out of order (bearings were installed after the decks).

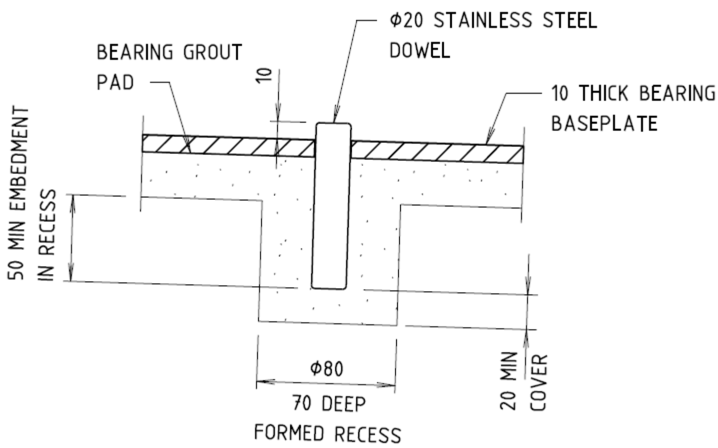

Figure 5-20 - Bearing layout (RMS, 2016)

The response to this issue during construction was to procure the dowel in galvanised steel and butt weld it to the bearing plate.

5.18 B-003: Bearing plate formwork access

The bearing arrangement consists of a laminated elastomeric bearing pad, stainless steel base plate and stainless steel pin with cementitious grout beneath the baseplate. The decks are landed onto a series of packers to design height grout and, after the deck levels are adjusted to design RL using jacks, the bearing plate/pad/pin is raised and the area below the plate is filled with shrinkage compensated high flow cementitious type grout. Formwork is installed at 10mm offset around the bearing plate and sealed with silicon to contain the grout. Due to the chosen located of the deck packers (in front of each bearing) and the limited space between the abutment curtain wall and the bearing plate (80mm to plate edge which needs to include 10mm gap, formwork and sufficient room to install and seal the formwork), it was difficult to install the formwork.

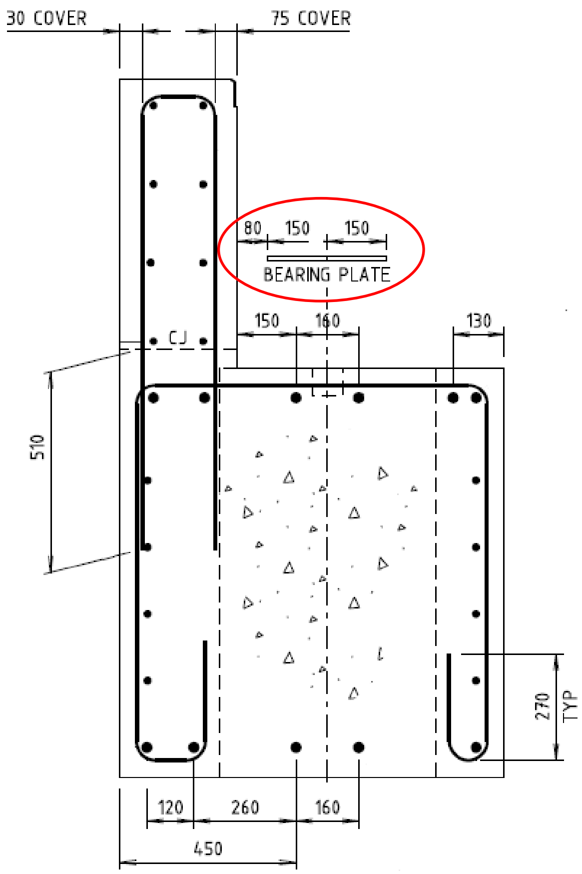

Figure 5-21 - Access for bearing installation (RMS [annotated], 2016)

A response to this issue during construction was not available during construction and as a result the formwork and grouting installation was prolonged and expensive, however potential responses for future constructions are presented in Chapter Six.

5.19 J-001: Alignment of transverse joint

The design for this bridge requires a small movement joint to relieve localised stresses and allow movement (expansion and contraction). The joint, as shown in Figure 5-22, comprises of cold-applied epoxy sealant over backing rod between the units. The concrete on either side of this joint needs to be poured on site and requires the use of a long formwork panel which is easily removable; in this construction flexible Styrofoam sheet was used. Due to the concrete being poured on both sides, it was difficult to keep the Styrofoam straight and resulted in the joint having slight bends as shown in Figure 5-23. This is unlikely to create any operation issues, but could be improved for the general appearance of the finished structure.

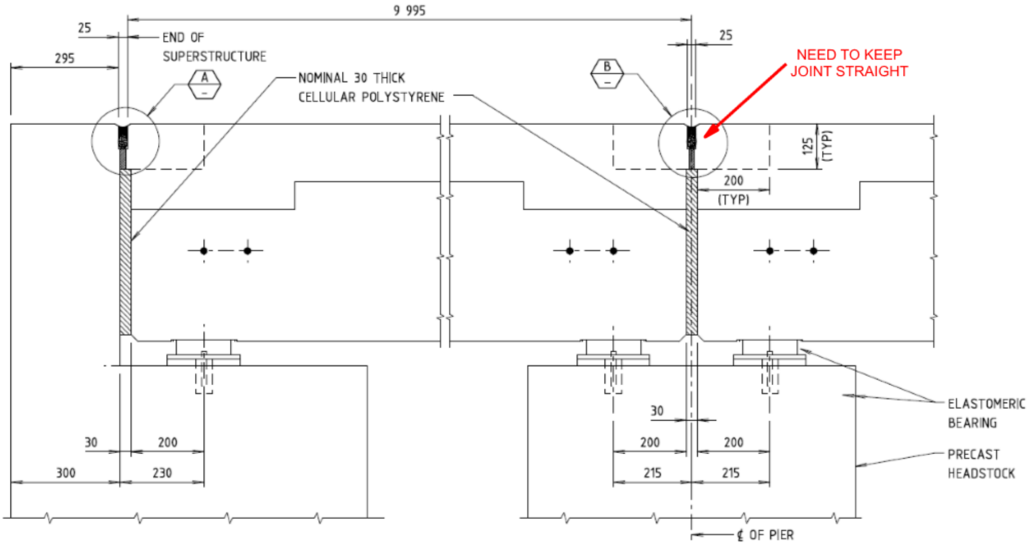

Figure 5-22 - Joint detail at Abutments and Piers (RMS [annotated], 2016)

Figure 5-23 - Joint alignment issue

5.20 HB-001: Binding and shearing of hold down bracket bolts

Hold down brackets have been included on this bridge to restrain the deck in the result of overtopping (design allows for two metre submergence). The hold down brackets comprise a section of 310UC118 fixed to the headstock/sill beam and deck units using cast-in stainless steel ferrules and M20 stainless steel anchor bolts. The brackets need to be fitted during module erection, then removed to allow grouting of the bearings and re-attached for final installation of the brackets. Unless adequately lubricated, stainless steel fixings are susceptible to galling (also known as cold welding - the seizing of threads due to applied force and/or friction) which makes the bolts difficult to remove during construction and bearing maintenance activities.

During the construction of this bridge, two bolts sheared off in the ferrules during removal and were unable to be extracted as shown below in Figure 5-24. As all required test certificates were supplied and the bolts were not lubricated prior to installation, the cause is believed to be thread galling.

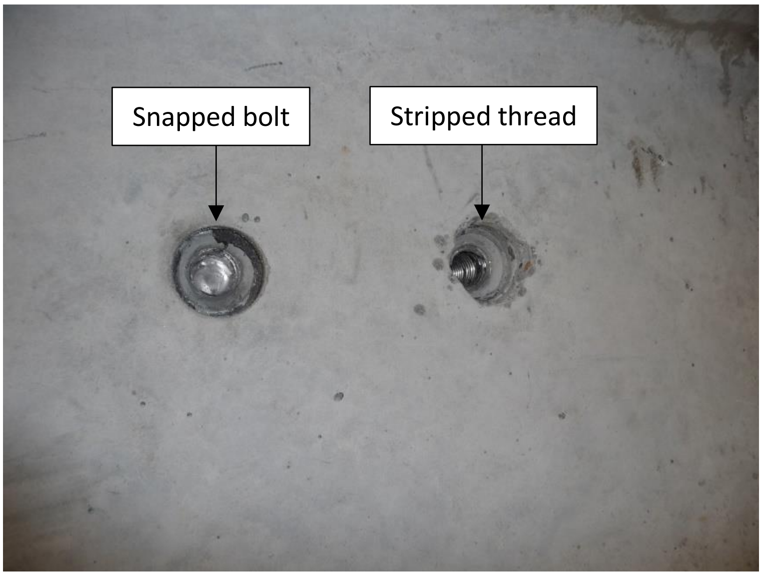

Figure 5-24 - Damage to hold down bracket bolts and ferrules

A response to this issue during construction was not available during construction, however potential responses for future constructions are presented in Chapter Six.

5.21 Conclusion

This chapter has introduced and discussed the issues identified during construction of the trial bridge. The issues were generally identified in all facets of the bridge, however it is noteworthy that the Abutment Footings (AFT) and Pier Footings (PFT) are over-represented when compared to other areas of the bridge. These elements are not part of the CBS system as the substructure of a bridge will always require a site specific design, however it is an important observation in terms of identifying troublesome areas generally. Table 5-1 contains a register of the issues identified as summation of this chapter.

Table 5-1 - Register of identified issues

Issue code	Description
AFT001 PFT-001	Spacing of reinforcement in footings presents risk of falling and injury to worker
AFT-002	Width of retaining wall results in visual inconsistency at front of Abutment B and compaction difficulties below sill beam at rear of Abutment B
AFT-003	Wing wall is not fully supported on retaining wall which leaves the wing wall unstable and prone to damage during pavement construction
AFT-004	No drainage provision at Abutment A to drain water and relieve hydrostatic pressure
AFT-005 PFT-002	Specified hole for fixing dowels to rock is too large for easy procurement of suitable drilling plant
SB-001	The top of the sill beam curtain wall may be subject to concrete damage and breakout due to high vehicle impact loads
SB-002	The top surface of the sill beam curtain wall forms part of the running surface but is provided smooth finished which has poor tyre grip properties
DMI-001 DME-001	Cover to steel at the underside of the end diaphragm is less than design
DME-002	Scupper inlet is located higher than the finished deck level which will prevent full draining of water from the bridge deck
DMI-002 DME-003	Flanges are rough finished which created issues with sealing between adjacent units

DS-001	Access to install bottom layer of longitudinal reinforcement is obstructed by top layer of precast projected transverse reinforcement
DS-002	Finishing of the central closure pour with two way cross-fall is difficult to maintain a straight defined apex
DS-003	Reinforcement layout in end diaphragm is congested which may result in inadequate concrete penetration
TB-001	The top rail of traffic barrier is 800 mm off deck height whereas the minimum height of a temporary edge protection system is 900 mm. It would assist in construction if the traffic barrier could also function as edge protection
TB-002:	The bolts projected from the deck kerb into the traffic barrier are different sizes, likewise the receiving holes in the traffic barrier base plate. These variations increase the chance of construction errors.
B-001	Half of the bearings on the sill beam are located within the stitch areas to the Abutment which results in extra site work that would be required if there was no clash
SB-004	
B-002	Bearing pin specified as independent to bearing plate caused issue achieving design penetration into head stock or sill beam void and elastomeric bearing pad
B-003	Access to install bearing grout pad formwork between the bearing and the Abutment curtain wall is limited
J-001	Difficult to ensure straight alignment of transverse expansion joints during pouring adjacent deck cross beams.
HB-001	Some stainless steel bolts fixing the hold down bracket sheared in the stainless steel ferrules.

6 Rectification concepts

This section contains concept designs devised to address the issues described in Chapter Five. The concepts have been arrived upon by consideration of the reason that each issue has been identified (time, quality, cost, safety or a combination of these areas). Constructability, safety, structural adequacy and overall viability are considered in an instinctual and practical sense, however the concepts not been formally moderated. Evaluation of selected issues is presented later in this report; the purpose of this chapter is only to introduce the concepts.

Each issue has a minimum of two concepts, with the "no action" being presented consistently across all issues. This is partly because it is a commonly considered option is industry, but also because it sets an important baseline during later analysis of the options.

6.1 AFT001 / PFT001: Spacing of reinforcement in footings

6.1.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.1.2 Reduce bar spacing

This option would involve reducing the spacing of the deformed bar in the reo cage to 150 mm centres in order to reduce the gap between adjacent bars, thereby reducing the size of the open hole and reducing the potential for a person to fall. To implement this option, it would be required to change the spacing of the steel over the entirety of both footings in order to provide continuity of spacing and facilitate proper alignment and lap length of joined bars. A preliminary check of size and spacing of new bars is below.

Abutment A: largest dimension is across the Abutment footing (9800mm) hence this shall govern. The largest reinforcement in this layer consists of 49 x N20 bars and 200mm c/c.

$$\text{For 49 off N20, } A_{ST} = 49\left(\frac{\pi \text{ x } 200^2}{4}\right) = 1{,}539{,}380 \text{ mm}^2$$

$$\text{N16} = \left(\frac{\pi \text{ x } 160^2}{4}\right) = 20{,}106 \text{ mm}^2$$

$$\frac{1{,}539{,}380}{20{,}106} = 76.6 \therefore 77$$

$$\therefore 49 \text{ x N20 bars} = 77 \text{ x N16 bars}$$

Check dimension

$$\frac{9800}{77-1} = 128 \text{ mm}$$

Therefore, adopt N16 bars at 130mm centres as a concept for this option.

6.1.3 Pre-fabricated reinforcement cages

Pre-fabricated reinforcement cages involve off-site construction of the reo cage which is then delivered to site intact and installed into position. If a reinforced concrete element is to be constructed with known dimension and bounds, pre-fabricated reinforcement cages are often an economical and efficient method of construction that reduce manual handling and construction waste on site (Ausreo 2012 and Natsteel 2013). Pre-fabricated cages were used for the pier columns only in this project as they were the only elements whose dimension (800mm diameter) was known, whereas the variable rock depth in the footings meant that the full subsurface profile was not known until excavation was complete.

6.1.4 Construction joint to reduce fall height

This option would involve specifying construction joints at 1/3 and 2/3 of footing height for Abutment A and 1/3 height at Abutment B (as shown in Figure 6-1 for Abutment A) in order to limit the maximum fall height and reduce the potential for or severity of an injury if a worker loses their footing. Whilst this option does not address the root cause of the issue (being large gaps), it does make some progress towards mitigating the impact if the risk is realised at Abutment A and Abutment B only as the pier footings are already less than one metre deep.

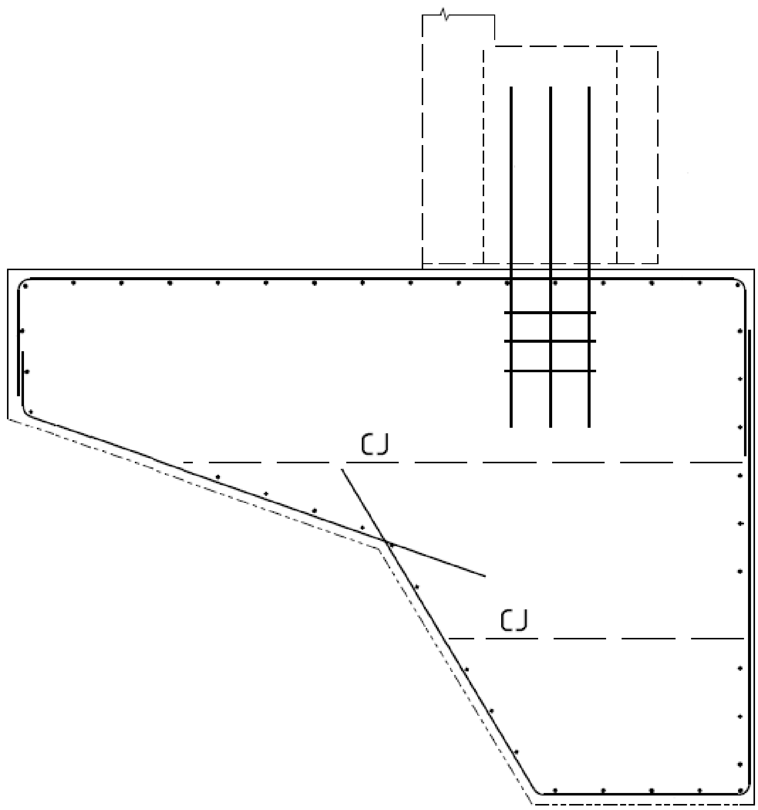

Figure 6-1 - Construction joint option to Abutment A

6.2 AFT-002: Width of Abutment B retaining wall

6.2.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.2.2 Increase thickness of retaining wall

This option is consistent with the change made during construction where the retaining wall was increased to 1080mm thickness to remove the potential compaction issue and provide visual continuity with Abutment A. A drawing of the retaining wall as constructed is shown below in Figure 6-2.

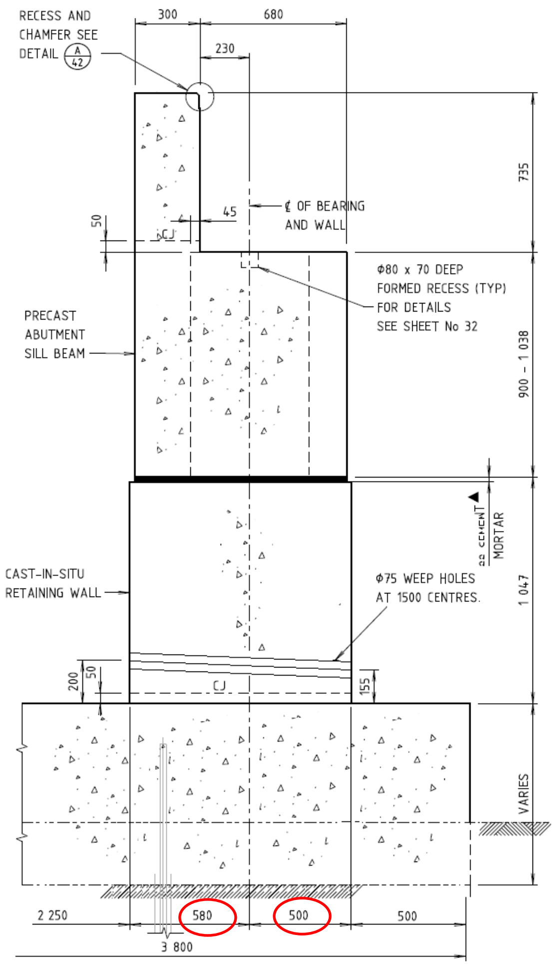

Figure 6-2 - Increased retaining wall depth option

6.2.3 No fines concrete backfill

This option would not involve any change to the dimension of the retaining wall or sill beam, rather it would use a controlled material to backfill the area of the overhang at the rear of the abutment. No fines concrete is, as the name suggests, concrete that contains little to no fine aggregate such as sand (CCAA, 1999). Unlike granular backfill, no-fines concrete can be easily compacted with no additional skills or plant than that which would already be on site for the bridge construction. The no-fines concrete is also free draining and could form part at the Abutment drainage system. A concept drawing is shown in Figure 6-3.

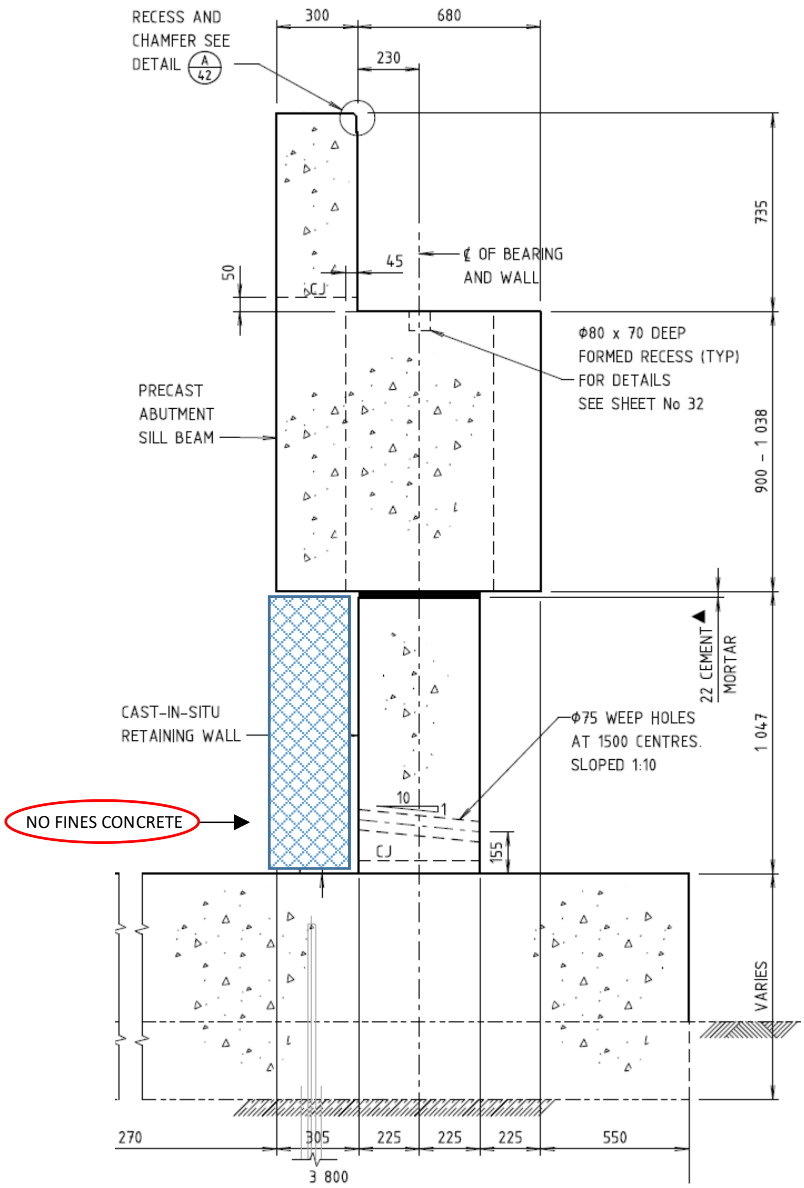

Figure 6-3 - No fines concrete backfill option

6.2.4 Remove retaining wall and increase slab footing height

This option would involve removal of the retaining wall and increasing the height of the spread footing by a commensurate dimension. To maintain the design level of the bridge deck, an additional 18.6 m^3 (approx.) of concrete would be required (1025mm x 8200mm x 2270mm). A concept drawing is shown below in Figure 5-12.

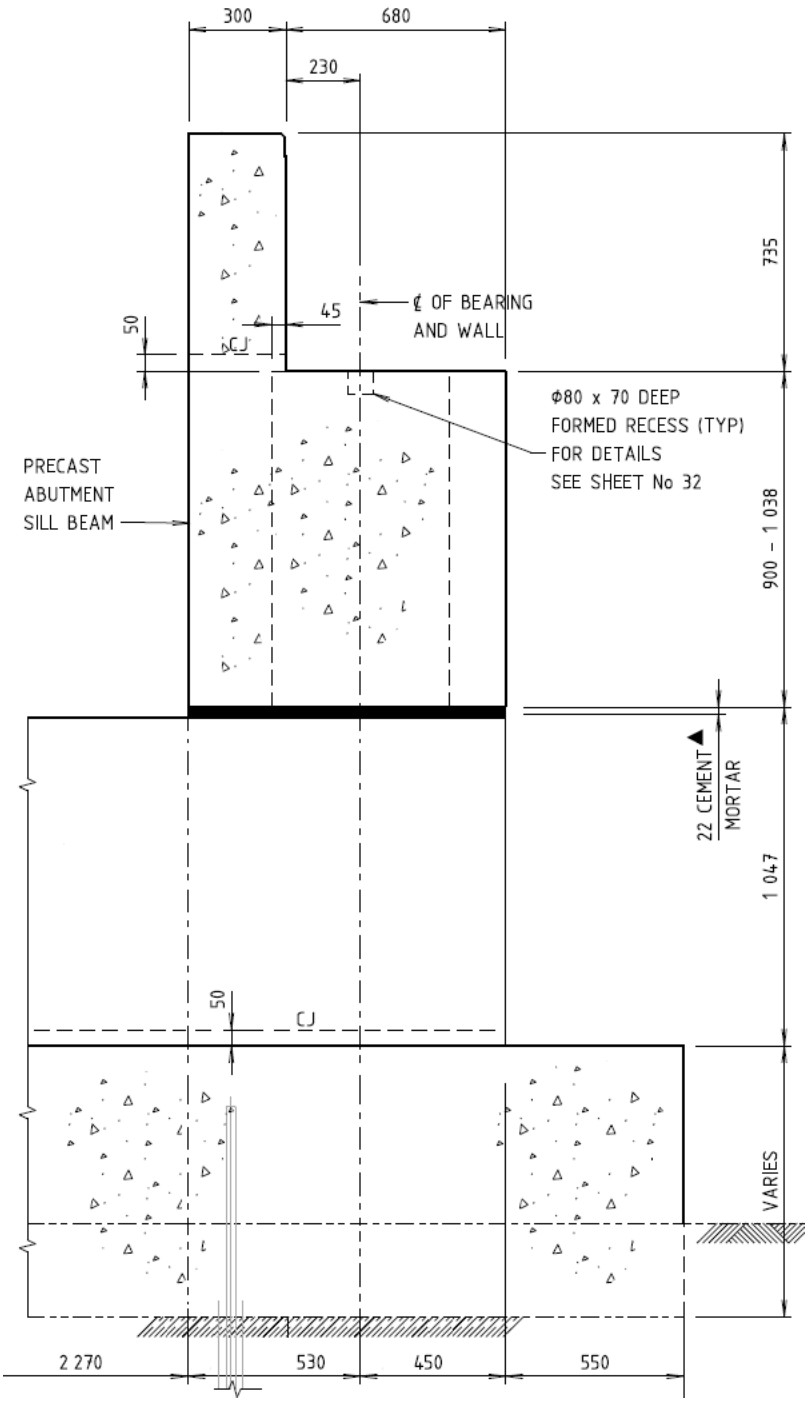

Figure 6-4 - Increase height of Abutment footing option

6.3 AFT-003: Depth of Abutment B retaining wall

6.3.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.3.2 Extend retaining wall

This option requires extension of the retaining wall to be 4,925 mm to provide support under the full length of the precast wing wall. This would increase the stability of the wing wall and, by removing the need to construct granular fill beneath the wing wall, reduce the potential for earthmoving equipment to cause damage to the bridge. This option is consistent with the change implemented during construction of the trial bridge, a drawing of which is shown below in Figure 6-5.

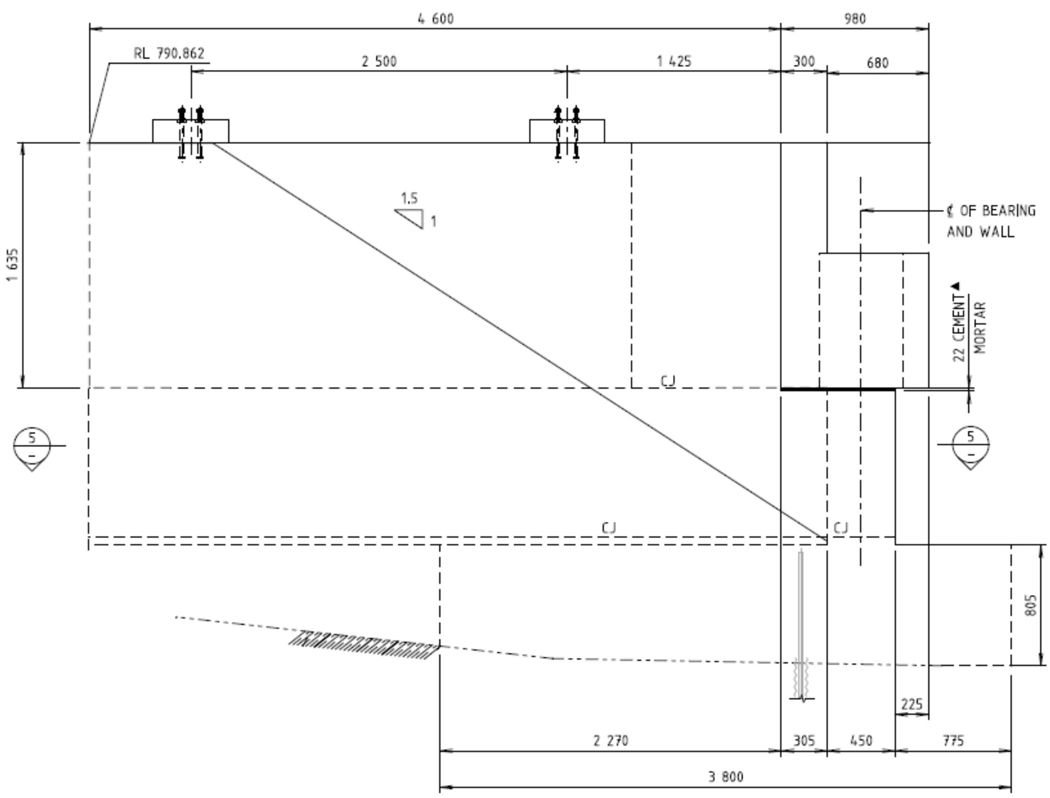

Figure 6-5 - Extend retaining wall option

6.4 AFT-004: Abutment A drainage

6.4.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.4.2 Specify proprietary drainage system on the drawings

Bridge construction drawings typically specify the drainage requirements behind the abutment to ensure that the assumed design conditions are being fulfilled. The drawings or annotations include specifications the products must meet and/or recommend a proprietary product or equivalent. This information or other direction was not present on the plans, so the construction team adopted the drainage detail recently used in construction on the bridge over Tangaratta Creek on the Oxley Highway as reproduced below in Figure 6-6. This option involves adopting the drainage detail as installed on site.

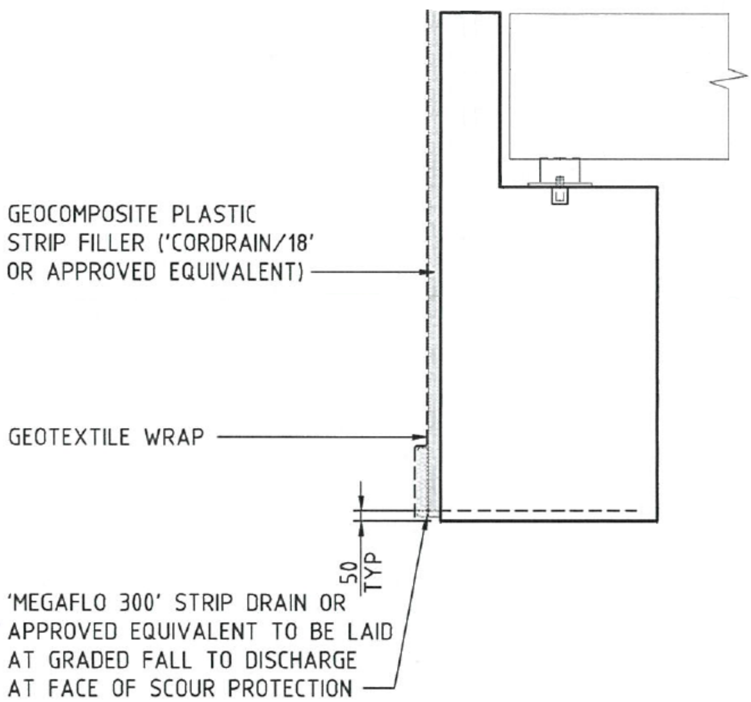

Figure 6-6 - Proprietary drainage system option (RMS, 2014)

6.4.3 Cast weep holes into sill beam

This option would involve the inclusion of 4 x 75mm diameter weep holes in the precast sill beam in the same location and spacing as specified for the Abutment B retaining wall. This would allow water to drain and provide pressure relief and could be combined with the no-fines concrete option for AFT-002 / SB-001 or the previous proprietary drainage system option for greater effectiveness. A drawing of this option is shown below in Figure 6-7.

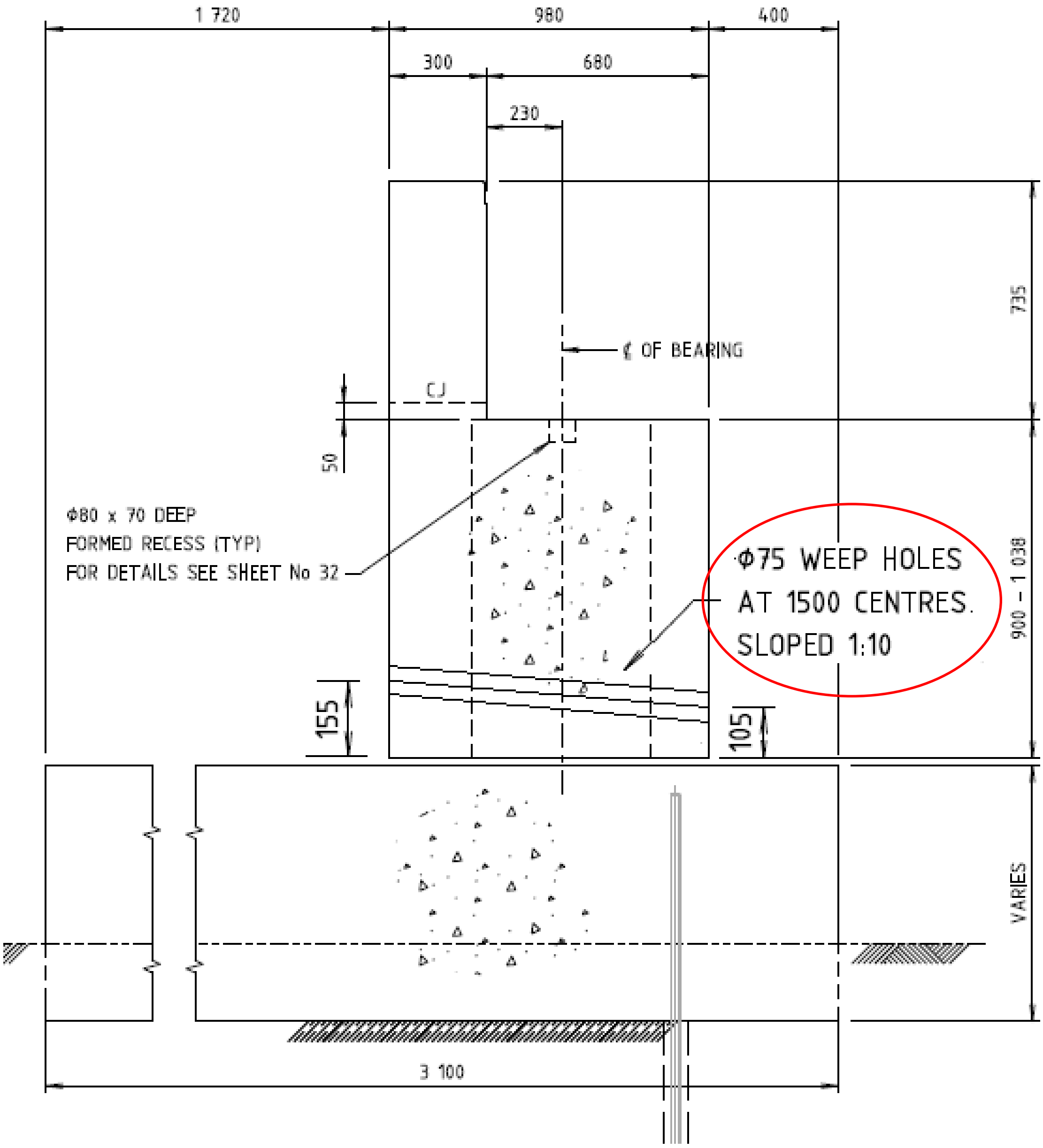

Figure 6-7 - Weep hole in sill beam option

6.4.4 Cast weep holes into wing wall

This option would involve the inclusion of one 75mm diameter weep hole in each precast wing wall located approximately 1/3 in from the end of each wing wall. The holes in either side of the wing wall blockout would be joined by a sacrificial PVC pipe form prior to pouring the in-situ closure concrete.

This would allow water to drain and provide pressure relief, but it must be combined with the no-fines concrete option for AFT-002 / SB-001 or the previous proprietary drainage system option in order to draw water away from the back side of the sill beam an into the weep holes. A drawing of this option is shown below in Figure 6-7.A drawing of this option is shown below in Figure 6-8.

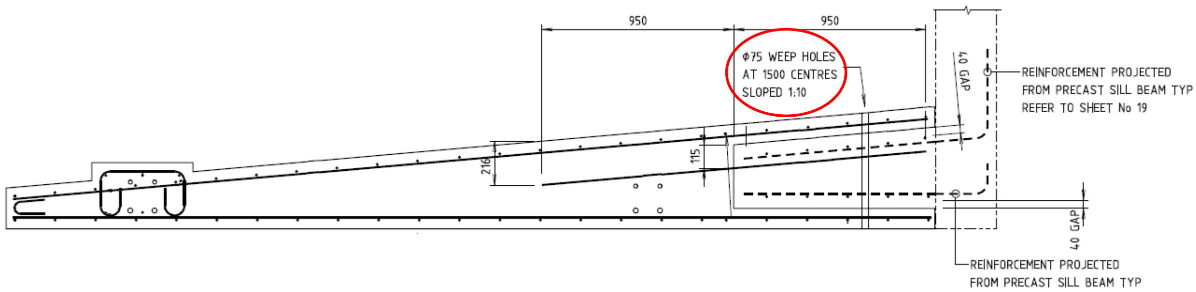

Figure 6-8 - Weep hole in wing wall option

6.5 AFT-004 / PFT-002: Dowel hole size to footings

6.5.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.5.2 Decrease diameter of holes and dowels

This option is consistent with the site response, and would involve specifying the holes to be drilled at 50mm diameter and inserted with N24 stainless steel dowels. The substructure for each future CBS bridge is to be site specific hence the design used for Bookookoorara Ck is unlikely to be replicated verbatim, however Table 6-1 (below) was used for determining the quantity of N24 bars required to replace the N36 bars. This simple design aid is included in this report as it may be useful for future constructions.

Table 6-1 - N36 to N24 conversion table

N36 bar	N24 bar
1	3
2	5
3	7
4	9
5	12
6	14
7	16
8	18
9	21
10	23

The table was calculated as a rounded up ratio of cross sectional areas for the two bar diameters. The following calculations are presented by means of example

$$\mathrm{A_S}\ (5 \text{ off N36}) = 5 \text{ x} \frac{\pi \text{ x } 36^2}{4} = 5089.38 \text{ mm}^2$$

$$\mathrm{A_S}\ (1 \text{ off N24}) = \frac{\pi \text{ x } 24^2}{4} = 452.39 \text{ mm}^2$$

$$\therefore \text{equivalent N24 to 5 off N36} = \frac{5089.38}{425.39} = 11.25 = 12 \text{ bars}$$

6.6 SB-001: Potential damage to Abutment Curtain Wall concrete

6.6.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.6.2 Install protection angle

This option would involve installing a full width steel protection angle on the leading edge of the curtain wall to provide impact resistance and reduce the potential for damage to the concrete, similar to the arrangement that is sometimes installed on the soffit of low overhead clearance concrete bridge members (RailCorp, 2009). This method of protection was widely used in the past and generally consisted of 75 x 75 or 90 x 90 x 10 EA steel equal angle fastened to 16mm shear connectors cast into the concrete member. This arrangement is shown in RMS Standard Drawing ANGLES.dgn as reproduced below in Figure 6-9.

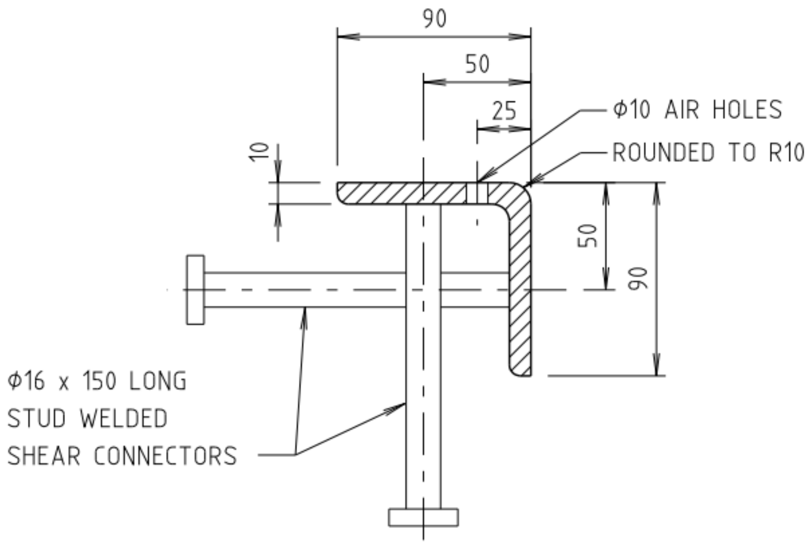

Figure 6-9 - Protection angle option (RMS, 2012)

6.6.3 High strength concrete

This option would not involve the installation of any additional physical protection to the curtain wall, but specifying concrete with a higher compressive strength (say 50 MPa) be used in the curtain wall to reduce the potential for breakout as a result of vehicle impact.

6.6.4 Approach slab

This option would involve the installation of a concrete approach slab between the pavement and the abutment. This approach would reduce the magnitude of the vehicle impact load experienced by the bridge (VicRoads, 2001), thereby reducing the potential for damage to the curtain wall concrete. To protect the leading edge of the approach slab, a 90 x 90 x 10 EA protection angle will need to be provide in the same manner as discussed on page 114. This arrangement is shown in RMS Standard Drawing AS6FPA.dgn as reproduced below in Figure 6-10. The same design is proposed for consideration for this bridge, however it is noted that this will also require modification of the sill beam to provide end support for the approach slab.

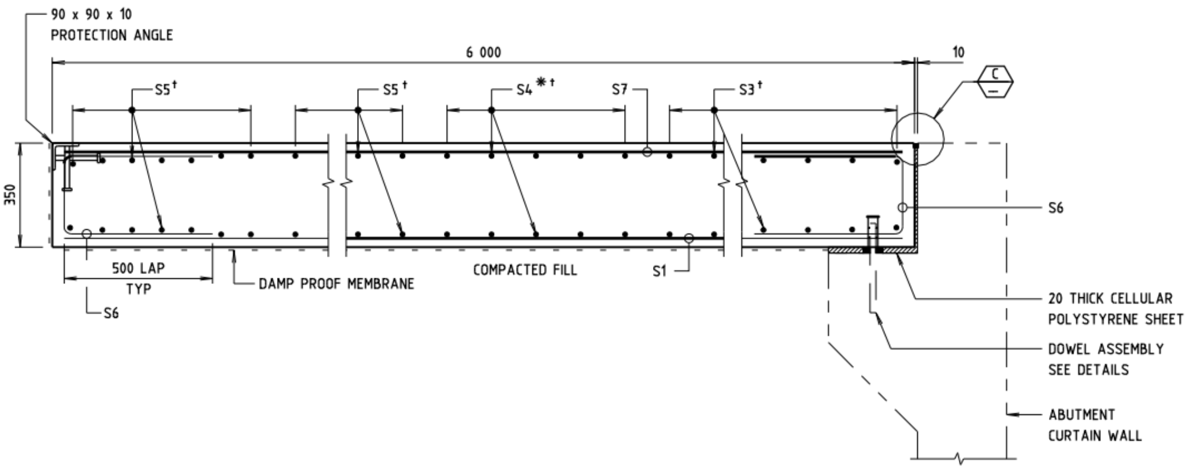

Figure 6-10 - Approach slab option (RMS, 2011)

6.6.5 Rigid pavement

This option would not involve changing the design of the Abutment sill beam in any way, rather it would control the pavement and approach conditions by specifying a rigid pavement. A rigid pavement has a higher compressive strength and is less prone to deformation than flexible pavement; properties that are typically achieved by the addition of a cementitious binder (USQ CIV3703, 2014.2). If this type of pavement were specified, it is less likely that the transition between the pavement and the bridge will experience grade separation, hence the impact load and potential for concrete damage experienced by the edge of the curtain wall will be substantially reduced.

6.7 SB-002: Roughness of curtain wall running surface

6.7.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.7.2 Mandatory seal of completed bridge deck

This option would not require any change to the precast process, rather it would include sealing of the bridge deck including the top of the sill beam as a mandatory part of the CBS system. This is only likely to be a feasible option if the adjacent road and approaches were already scheduled to be sealed as mobilising plant for such a small area is unlikely to be economical.

6.7.3 Rough finish the top of curtain wall

This option would involve annotating the design plans to specify that the finished surface of the sill beam shall be rough finished as shown below in Figure 6-11 in order to provide consistent roughness of the running surface. This would result in a minor increase in working time (less than one man hour) during the precasting process.

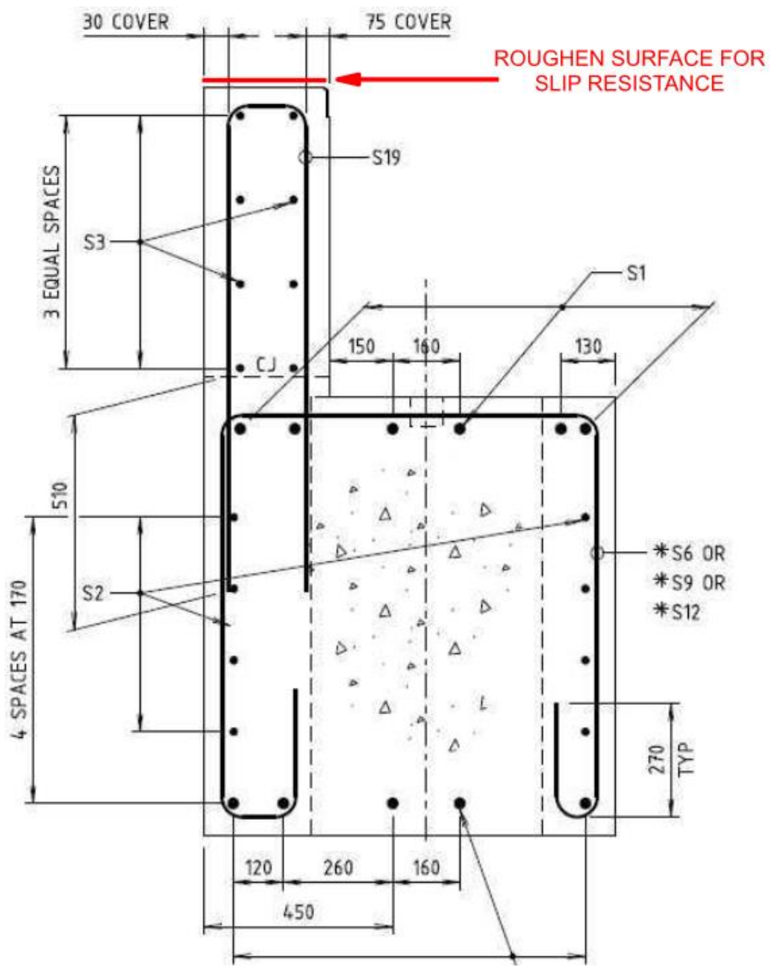

Figure 6-11 - Roughen curtain wall option

6.8 DMI-001 / DME-001: End diaphragm cover

6.8.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.8.2 Modify reinforcement layout

This option would involve increasing the cover by modifying reinforcement layout. The change would likely involve raising the projected reinforcement (D6 in Figure 6-12) by 20mm and modifying the ligatures (D4 in Figure 6-12) by a commensurate amount. When read in conjunction with issue DS-003 on page 126, it is noted that reducing modifying this layout may result in increased congestion and need additional controls implemented to ensure a high level of construction quality is maintained.

6.8.3 Specify corrosion resistant/protected reinforcement

This option would not require any change to the location or layout of either the projected reinforcement or the reinforcement that is installed on site, rather it would specify that corrosion resistant or corrosion protected material (such as galvanised steel or stainless steel) shall be used in the end diaphragm area. In Figure 5-9 on page 88, the bars designated as D4 and D5 in the bottom layer of site installed steel, as well as the bottom layer of the projected reinforcement from the precast module, would be specified as being either galvanised steel or stainless steel. This option would only require an additional annotation on the drawings rather than redesign of any component, but would increase the cost of construction as these materials are more expensive that common steel deformed bar.

6.8.4 Increase depth of diaphragm

Rather that modifying reinforcement detail or materials, this option would achieve the required cover by lowering the soffit of the end diaphragm until the minimum 40mm is achieved as shown in Figure 6-12. The formwork for the soffit is supported off the headstock or sill beam as shown in Figure 6-13, hence modification is a simple exercise that would result in a negligible change to working time, materials and cost. Each end diaphragm is 0.05m^3 (450mm x 435mm x 245mm) so this option would add 0.85 m^3 of concrete to the project. With concrete costing approximately \$290/m^3, the resultant increase in material cost would be less than \$250.

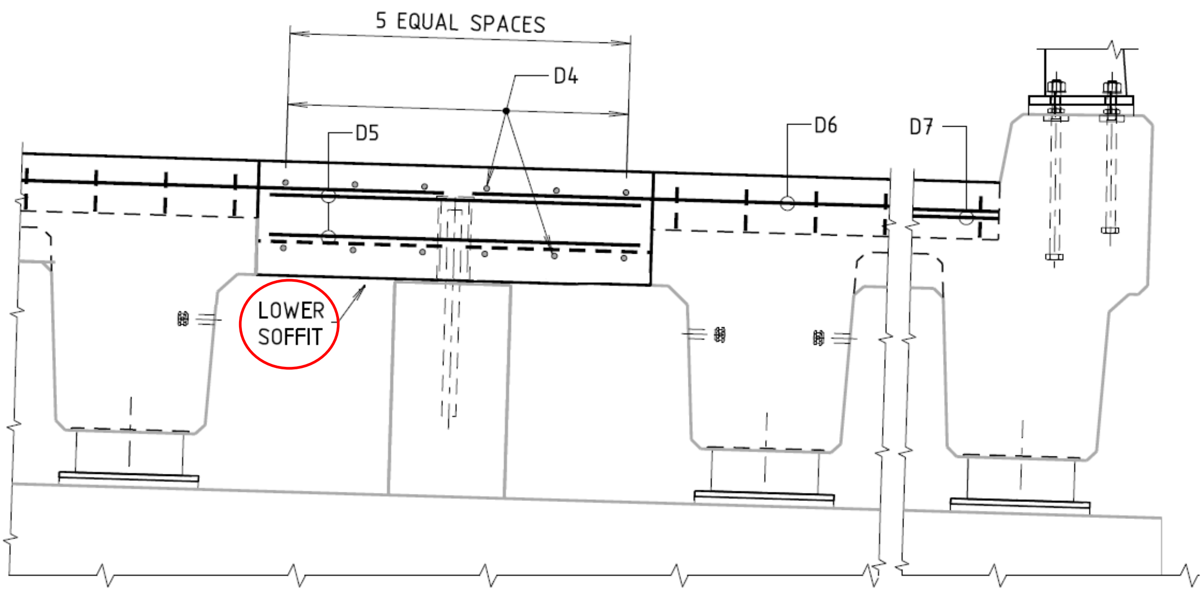

Figure 6-12 - Lower end diaphragm soffit option

Figure 6-13 - Soffit formwork around shear key

6.9 DME-002: Scupper height

6.9.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.9.2 Lower the scupper inlet

This option would simply involve lowering the scupper by at least 4mm to allow the inlet to be level with the deck. This would allow proper drainage and requires only a minor detailing change on the design plans with no increased construction cost. A drawing of this option is shown below in Figure 6-14.

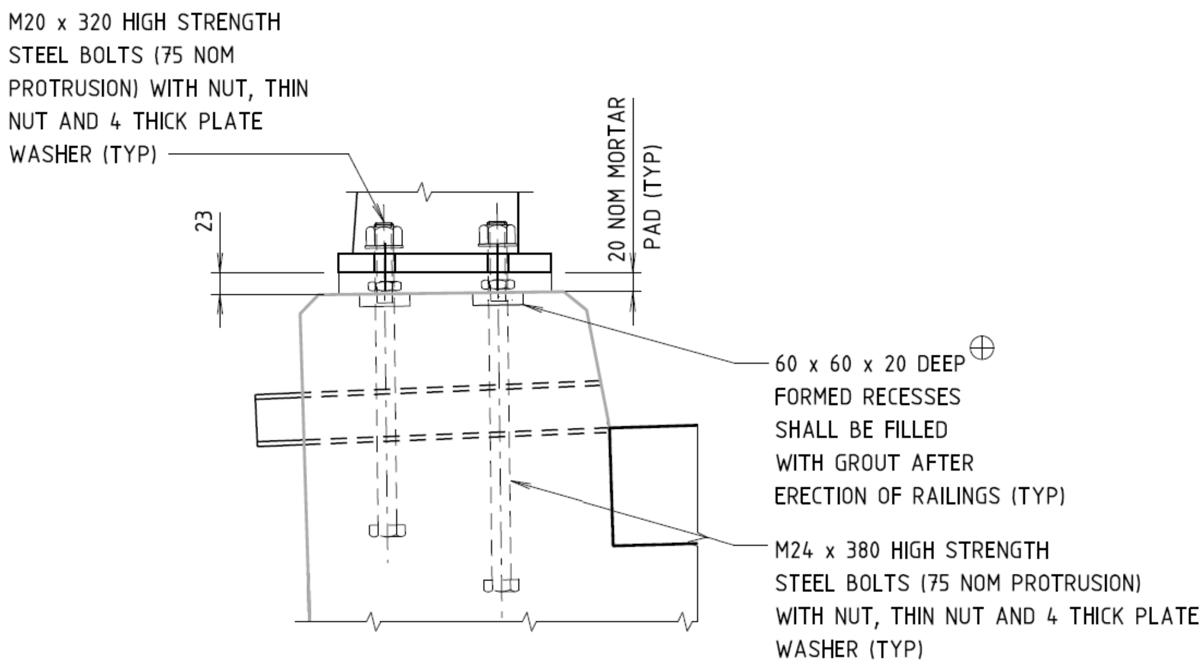

Figure 6-14 - Lower scupper inlet option

6.10 DMI-002 / DME-003: Surface finish of precast deck flanges

6.10.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.10.2 Remove flanges and construct cast in-situ soffit

This option would involve the removing the projected flanges as shown above in Figure 5-11 and casting the section of the soffit between adjacent units in-situ. This would remove the issue experienced with sealing the flanges as there would be no flanges to seal. A drawing of this option is shown below in Figure 6-15.

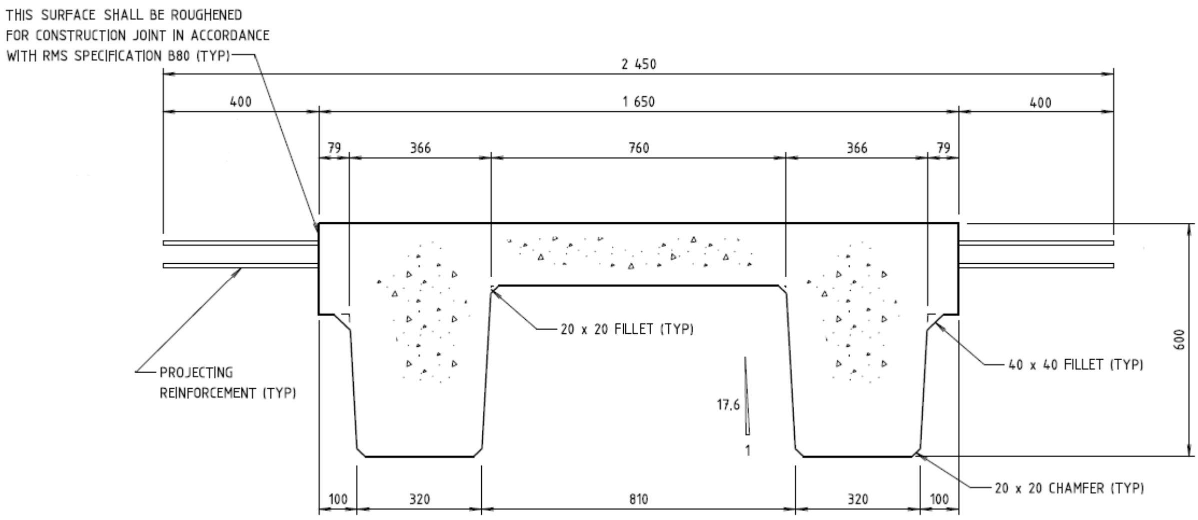

Figure 6-15 - Cast in-situ soffit

6.10.3 Smooth finish the edge of the flange

This option would simply involve roughening the majority of the flange surface in preparation for the construction joint but smooth finishing the last 50mm (approx.). This would provide a consistent and smooth surface to which the tape can be fixed, removing the potential leakage issue. A drawing of this option is shown below in Figure 6-16.

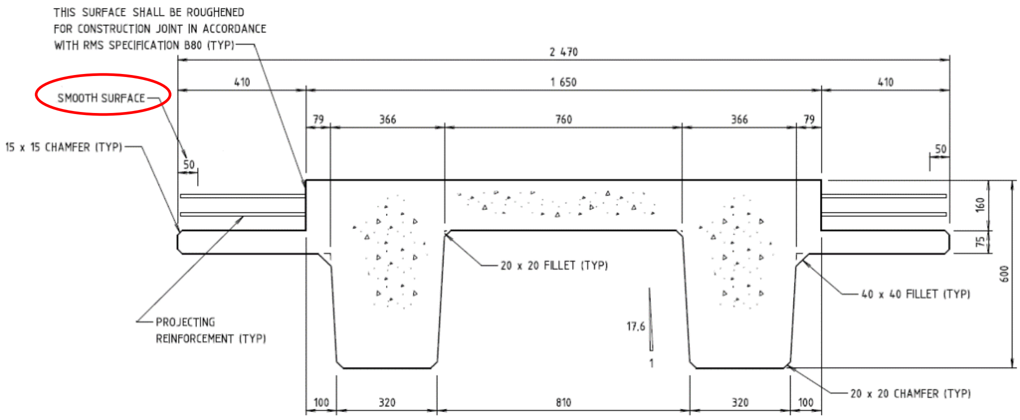

Figure 6-16 - Smooth finish of flange option

6.10.4 Provide recess on the edge of the flange

Similar to the last option presented, this option would also involve smooth finishing the last 50mm (approx.) of the flange to provide a consistent and smooth surface. This option differs from the previous option in that the smooth surface would be recessed by 9mm so that a strip of compressed fibreboard or similar can be installed and sealed as sacrificial formwork. A drawing of this option is shown below in Figure 6-17.

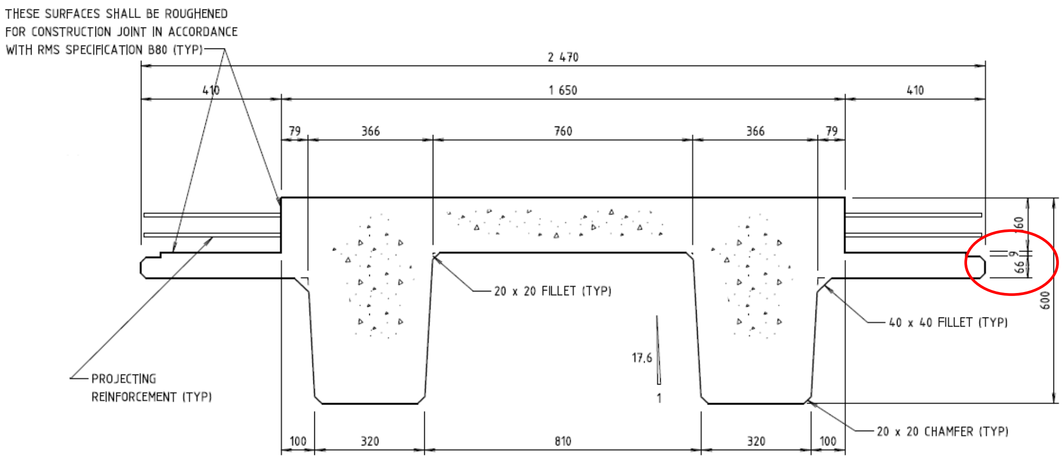

Figure 6-17 - Recessed flange option

6.11 DS-001: Longitudinal deck stitch pour reinforcement

6.11.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.11.2 Increase length of projected reinforcement

This option would involve increasing the length of the projected reinforcement in order to remove to requirement to install additional transverse bars to tie the bottom layer of transverse steel together across the full stitch width. The bars would be increased to project 880mm from the side of each module, being the sum of a 2 x 410mm flange, 2 x 15mm chamfer and 1 x 30mm nominal unit gap. These bars would line up between adjacent deck modules and be tied together. Note that this would only be an option for the bottom layer, as if the top layer were extended it would not be possible to install the bottom layer of longitudinal reinforcement. A drawing of this option is shown below in Figure 6-18.

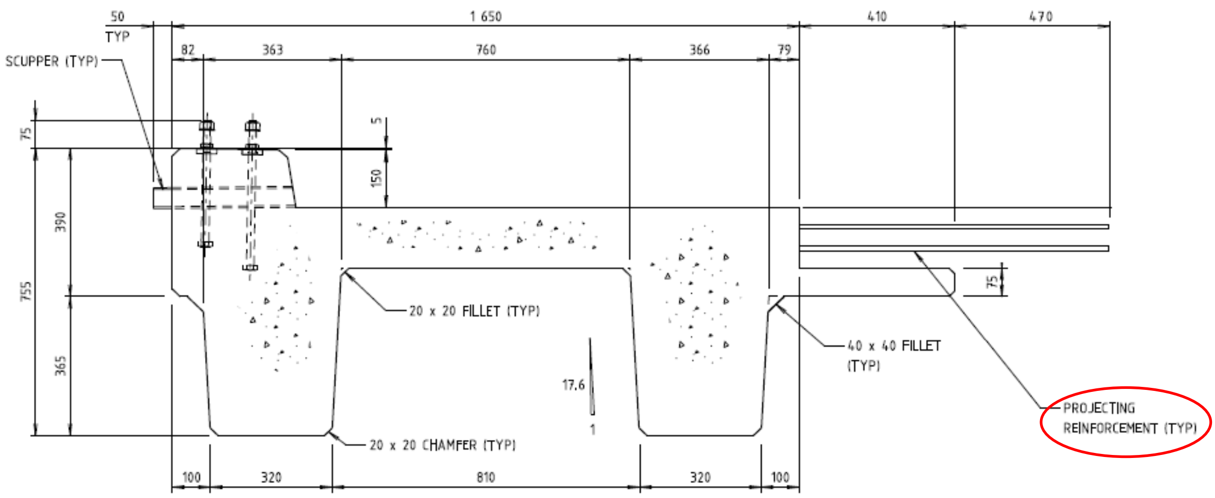

Figure 6-18 - Increase projected reinforcement length option

6.11.3 Loose fit bars prior to deck install

This option proposes a methodology change rather than a design change. In order to make the bottom layer easier to install, the bars will be loosely tied in bunches to the projected prior to installation (when access is good) and then moved into position and set in place after installation of the deck modules.

6.12 DS-002: Deck cross-fall finishing

6.12.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.12.2 Remove apex and have flat stitch pour area

This option would involve removing the apex/peak from the central stitch pour and allowing the area to be flat with the 3% cross-fall starting at the deck units. There is a possibility of water ponding if this were implemented, but more troublesome is that unless change, cover to reinforcement in the stitch area will reduce by 12mm (400mm width at 3% cross-fall). A drawing of this option is shown below in Figure 6-19.

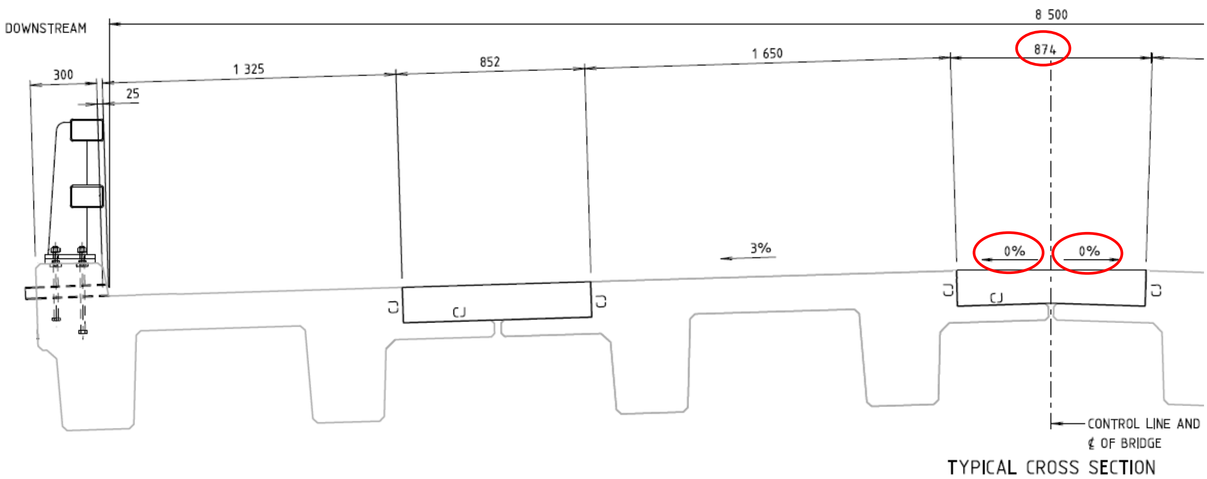

Figure 6-19 - Flat central stitch option

6.12.3 Rounded central stitch apex

This option would involve a simply detailing change to specify that the central stitch area shall be rounded with a target 3% cross-fall rather than having a defined apex. This option would allow water to drain better than a flat stitch but without the labour intensive finishing work of a defined apex. A drawing of this option is shown below in Figure 6-20.

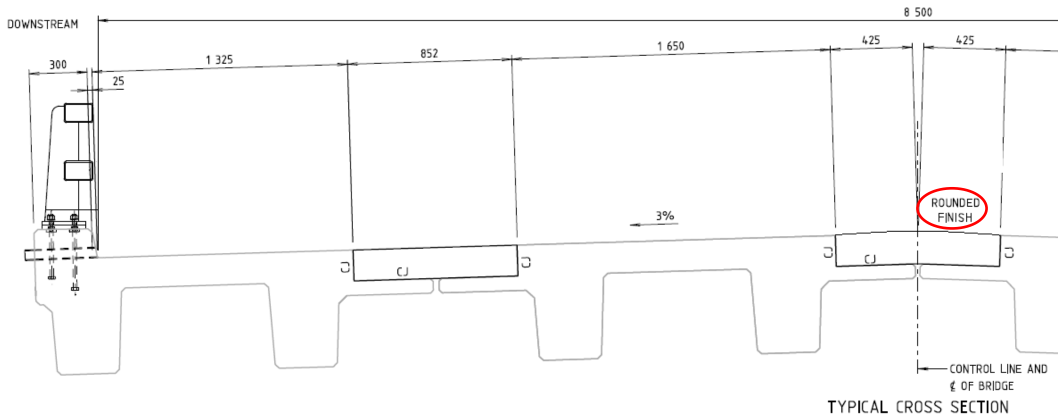

Figure 6-20 - Rounded central stitch option

6.12.4 One way cross-fall

This option would remove the central apex/peak and two way cross fall and specify one way cross-fall instead. This would allow controlled drainage of the deck and may deliver cost savings as the scuppers on the high side could be deleted. A drawing of this option is shown below in Figure 6-21.

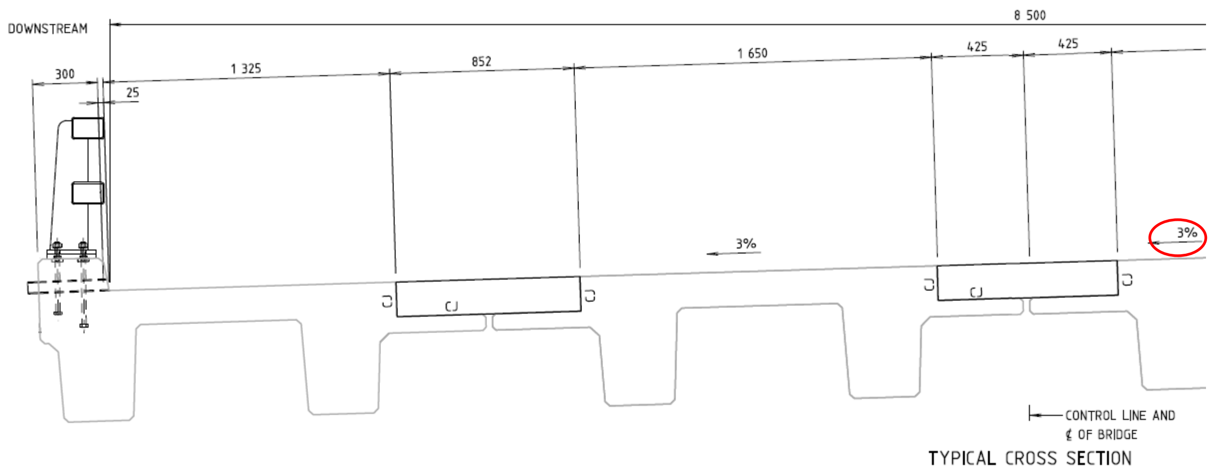

Figure 6-21 - One way cross fall option

6.13 DS-003: End diaphragm reinforcement congestion

6.13.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.13.2 Change reinforcement layout

As shown above in Figure 5-15, N16 transverse bars are lapped minimum 470mm with N16 bar projected from the precast deck unit. These transverse bars are the contained by N16 longitudinal ligatures and concrete poured. This option would aim to reduce reinforcement congestion by removing one or more of the transverse bars, thereby increasing the gap between adjacent bars and allowing better concrete flow and full encapsulation of the reinforcing steel.

The easiest way to reduce congestion is to minimise lapping of the N16 bars. A single pair of N16 bars has a cross sectional area of 402mm^2 ($2 \times \frac{\pi \times 16^2}{4}$) which is equivalent to a single N24 bar. Every second N16 bar projected out of the deck modules could be replaced with a N24 bar, staggered for adjacent units. This would remove every second lapped joint and increase the space between adjacent bars by 4mm. A drawing of this option is shown below in Figure 6-22.

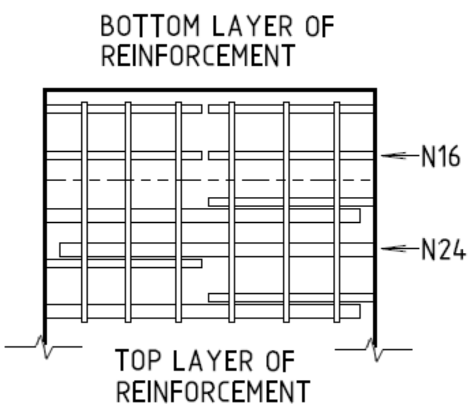

Figure 6-22 - Changed reinforcement layout option

6.13.3 Modify concrete properties

All concrete poured on site for this bridge was 40 MPa 28 day compressive strength, 80mm slump with 20mm coarse aggregate. To increase the ease with which the concrete can flow around the reinforcement and reduce the potential for voids, this option would involve either increasing the slump to in excess of 150mm or reducing the size of the coarse aggregate to 10mm.

6.14 TB-001: Height of traffic barrier

6.14.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.14.2 Increase height of barrier top rail

This option would involve increasing the clear distance between the barrier rails by 100 mm (still within the maximum clear distance of 475mm clear distance between each adjacent rails per Clause 3.6.2 (c)) in order to create a minimum barrier top rail of 900mm above deck level. A concept for this option is shown below in Figure 6-23.

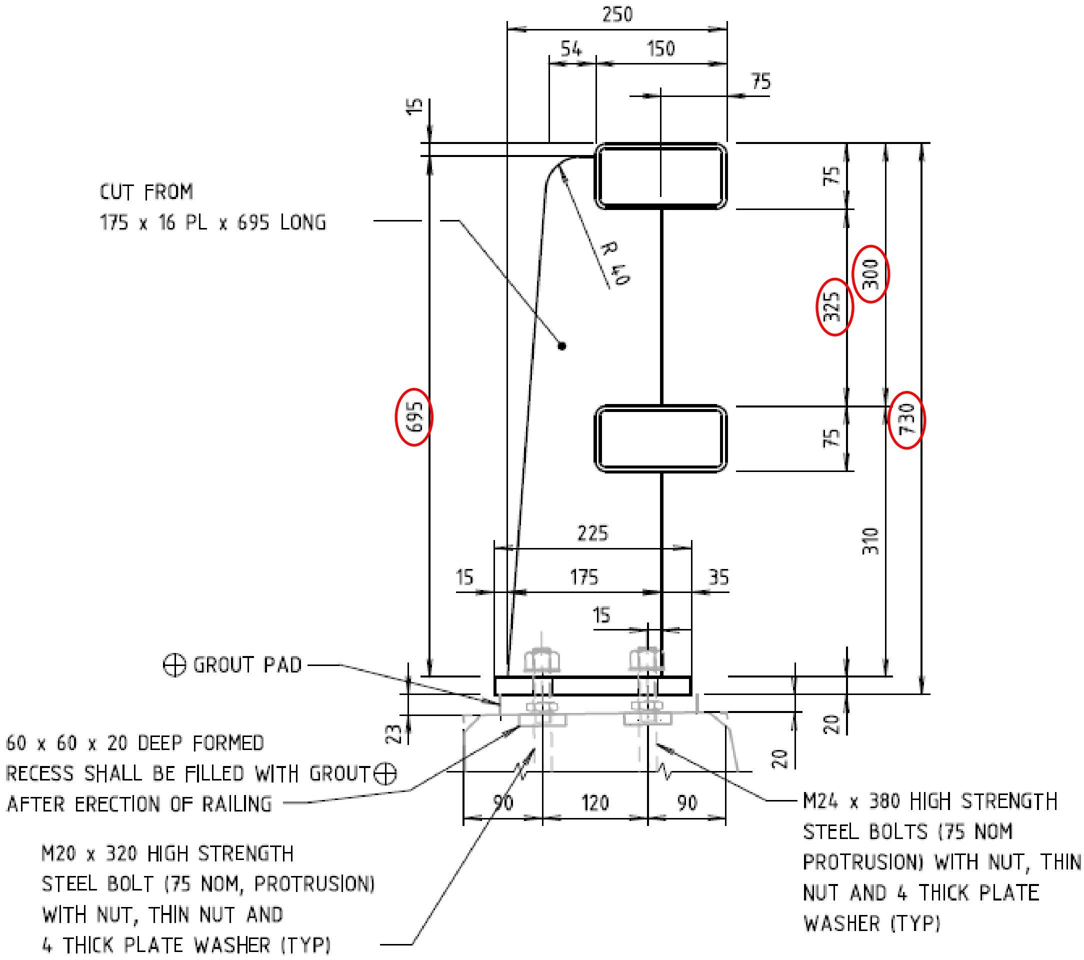

Figure 6-23 - Modified traffic barrier height option

6.14.3 Temporary top rail

This option would not involve no change to the dimension of the existing barrier rail, rather it would include provision to temporarily fix a higher top rail to the barrier for construction purposes only. The temporary rail could take the form of a bicycle rail as be fixed to the barrier through the use of pre-attached connection points on the top rail in a similar manner to QLD TMR Standard Drawing 453123A-C as reproduced in Figure 6-24. Note that only the bicycle safety rail component of the QLD barrier rail system, not the full detail of this barrier design, is proposed for investigation.

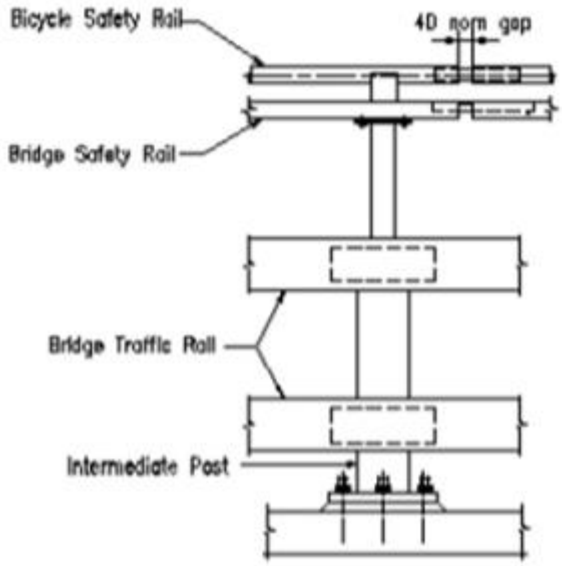

Figure 6-24 - Temporary top rail option (TMR, 2011)

6.14.4 Increase kerb height

This option would not involve increasing the height of the kerb on the precast external deck module by 100mm in order to achieve the minimum 900mm height between the bridge deck and the top rail of the barrier. Assuming a concrete density of 2450 kg/m^3, this option would add approximately 0.325m^3 (10m x 100mm x 325mm) or 800kg of concrete to the deck module. Whilst this weight is still within the carrying limits of a standard semi-trailer and does not exceed T44 axle loads, the implications on weight distribution and lifting point locations would need to be considered during detailed design. A concept drawing is shown below in Figure 6-25.

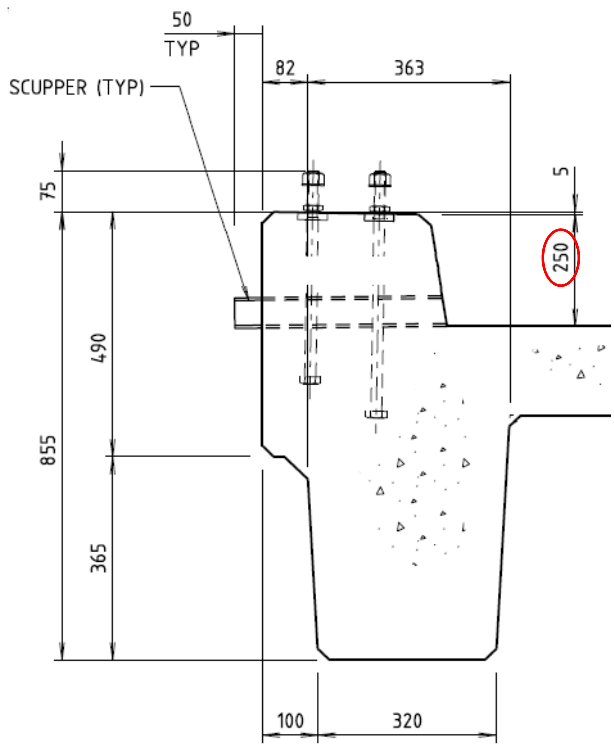

Figure 6-25 - Increased kerb height option

6.14.5 Provide plinths on kerb

This option would involve the installation of localised 100mm high concrete plinths cast into the kerb at the location of the barrier connections. This would achieve a similar outcome to the previous full length kerb height increase option but with a lesser increase to the deck module weight. Assuming a concrete density of 2450 kg/m^3 and four plinths per unit with dimension 100mm x 325mm x 500mm (to allow 100mm each side of the connection plate), this option would add approximately 0.065m^3 (10m x 100mm x 325mm) or 160kg of concrete to the deck module. Whilst this weight is still within the carrying limits of a standard semi-trailer and does not exceed T44 axle loads, the implications on weight distribution and lifting point locations would need to be considered during detailed design. A concept drawing is shown in Figure 6-26.

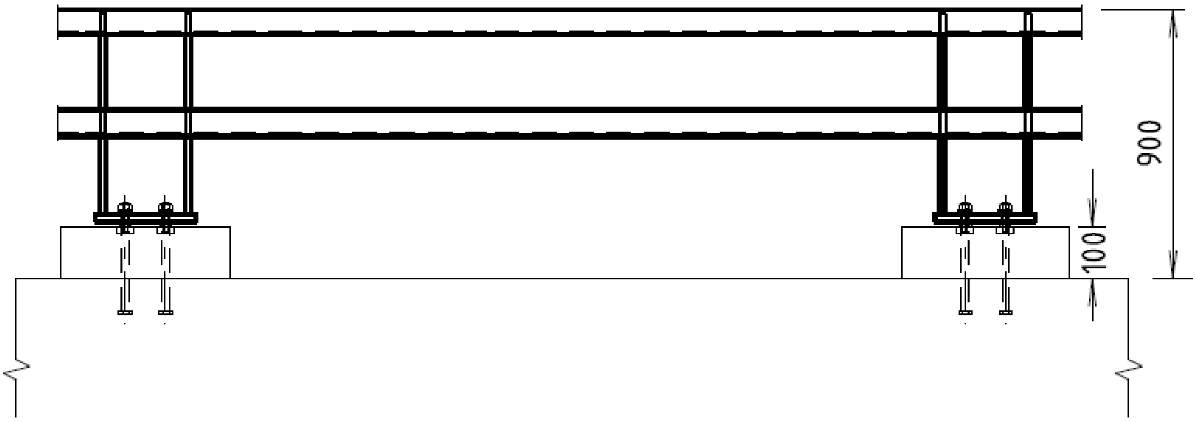

Figure 6-26 - Kerb plinths option

6.14.6 Increase length of protruding bolts

The traffic barrier is presently fixed to 4 bolts which protrude 75mm through the top surface of the kerb. The barrier is placed on top of the kerb, held in position using a top and bottom nut and grouted in secure the final position. This option would involve extending the bolts to protrude 175mm from the kerb, temporarily installing the barrier to be 900mm high during construction, then lowering the barrier to 800mm design height, cutting off the excess bolt length and grouting. A concept drawing is shown below in Figure 6-27.

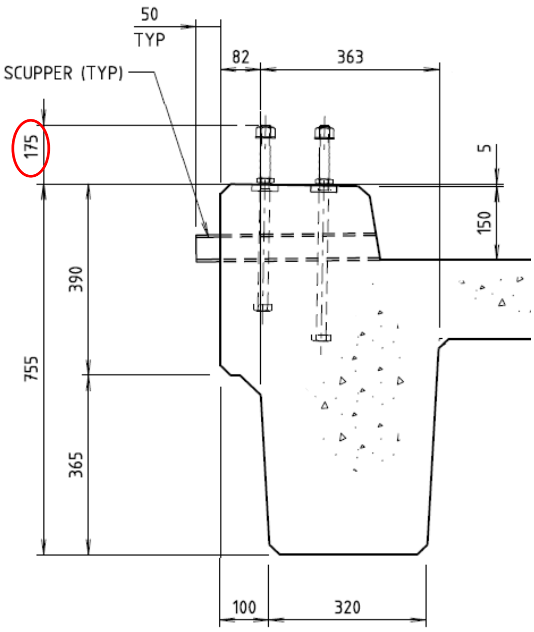

Figure 6-27 - Increased projected bolt length option

6.15 TB-002: Inconsistent bolt and hole sizes in traffic barrier base connection

6.15.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.15.2 Make both bolts the same size

This option would involve making both bolts projected from the kerb of the external deck unit the same size. The largest bolt is N24 which, in the event of impact, would act in single shear hence this size is assumed to be critical. As such, the back bolts would be increased in size from N20 to N24 as shown below in Figure 6-28.

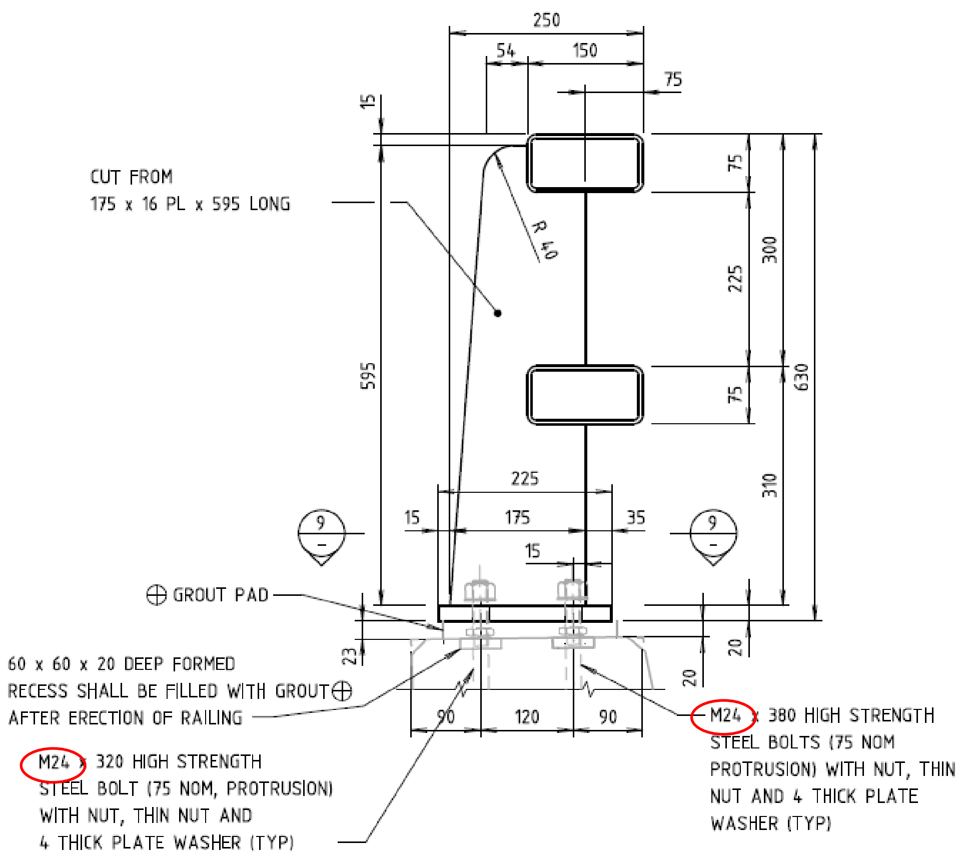

Figure 6-28 - Consistent bolt size option

6.15.3 Make holes in the base plate the same size

This option would involve making all four holes drilled in the base plate a consistent diameter. The largest bolt is M24 and the smallest hole larger than a M24 bolt is 26mm diameter hence this size will be adopted. Oversize washer would be used for the M20 bolts to account for the 3mm either size of the bolt when it passes through the 26mm hole. A drawing of this option is shown below in Figure 6-29.

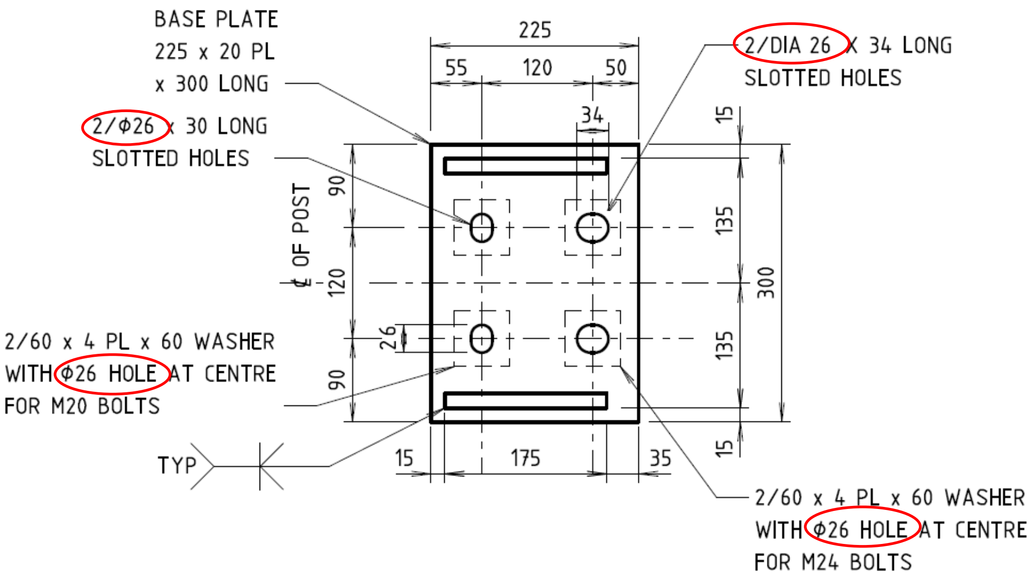

Figure 6-29 - Consistent base plate hole size option

6.15.4 Change both the bolt size and hole size

This option is a combination of both of the previous options. It would involve standardising the bolts to both be 24mm as shown in Figure 6-28 and the holes to all being 26 mm diameter as shown in Figure 6-29.

6.16 B-001 / SB-004: Bearing clash with sill beam blockouts

6.16.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.16.2 Change location of blockouts

This option would simply involve moving the location of the blockout in the sill beam away from the location of the bearing pads, and the moving the projected reinforcement from the abutment to suit. The column to head stock connection does not result in any dimensional clash with the bearings, so this option proposes to use the same spacing for the sill beams. A drawing of this option is shown below in Figure 6-30.

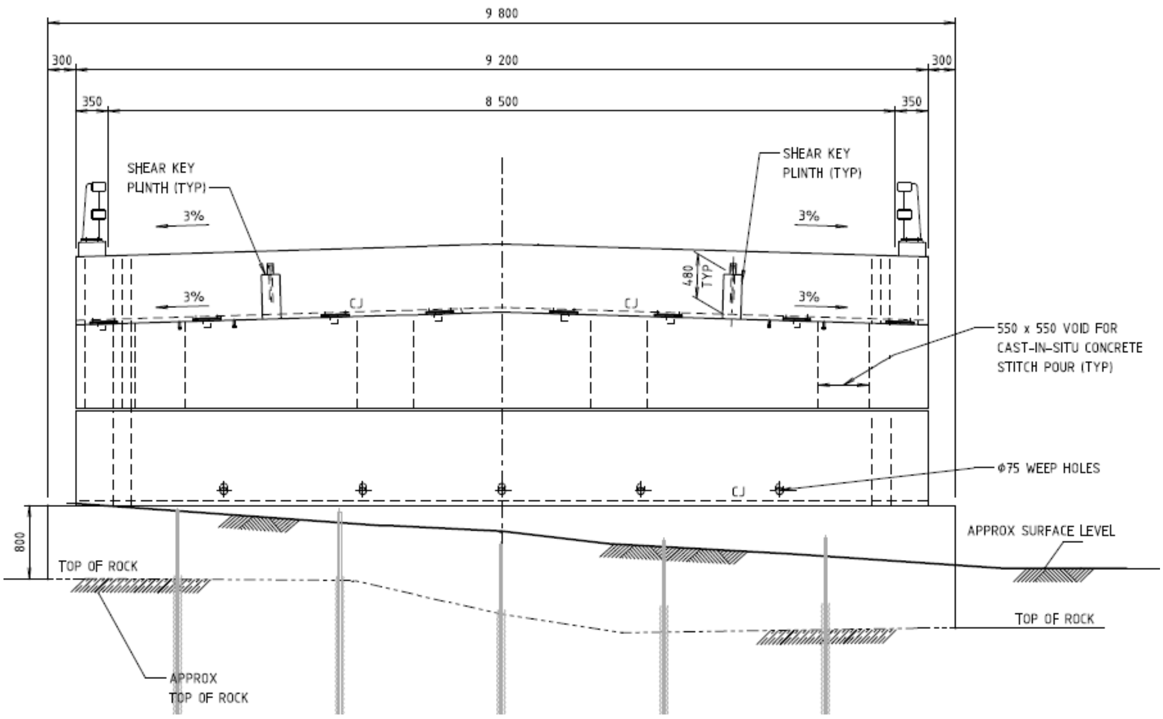

Figure 6-30 - Changed blockout location option

6.16.3 Bearing strip rather than bearing pad

The precast deck modules are supported on 230 x 200 x 97mm laminated elastomeric bearing pads, all of which have an individual grout pad, bearing plate and dowel contained in 80 mm diameter x 70 mm deep recess. It is the recesses that are causing the issue raised herein, hence removing the requirement for recesses and replacing the individual bearings with a full width bearing strip will remove the issue. The thickest bearing pad compliant with RMS B280 Specification *Unreinforced elastomeric bearing pads and strip* is 25mm thick whereas the individual bearing pad current specified is 97mm thick so either the sill beam shelf will need to be raised on the height of the curtain wall reduced. Additional design work would also be required to incorporate the shear restraint currently provided by the shear key plinth and the uplift restraint currently provided by the hold down bracket, both of which are secured on the head stock of sill beam where the bearing pad would run. A drawing of this option is shown below in Figure 6-31.

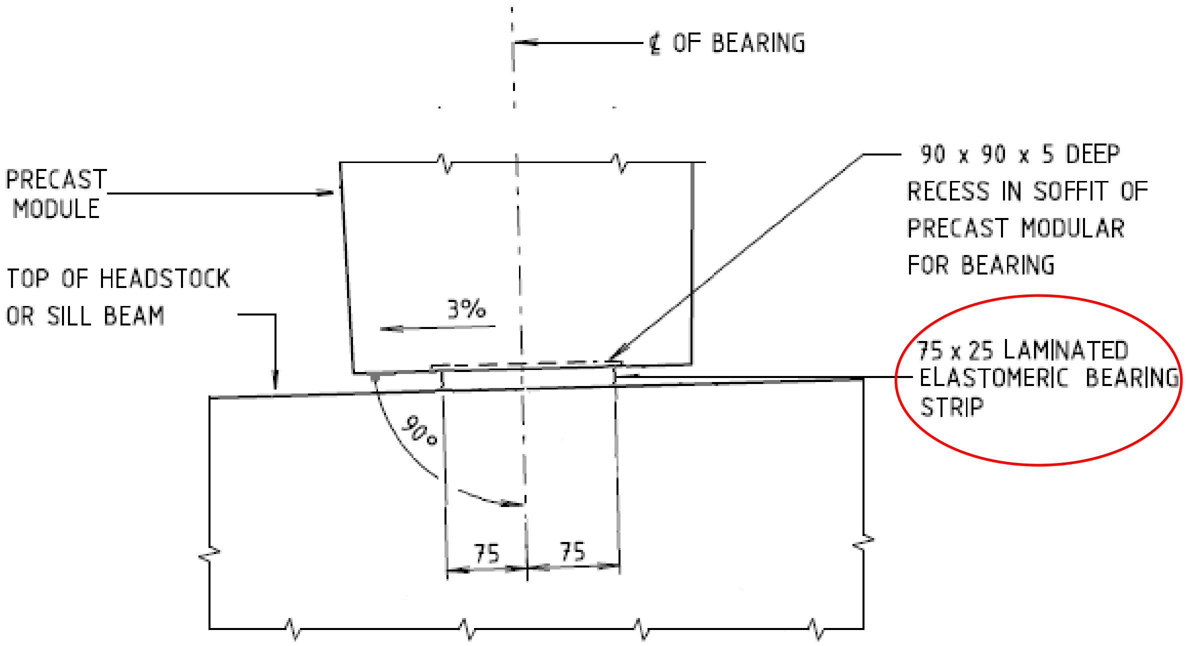

Figure 6-31 - Bearing strip option

6.16.4 Cast void into blockout

This option does not involve any design change, rather a change to the methodology to specify that the recesses occurring in the blockouts shall be cast during pouring not drilled after the concrete has cured. The recesses were drilled during construction as half of the blockout clashes occurred near the edge of the blockout area hence it was envisaged that issues would be encountered getting proper movement and compaction of the concrete between the edge of the precast unit and the recess formwork. Nonetheless, it is reasonable to assume that as the recess will be filled with grout the given dimensions of the recess are the minimum requirement hence larger recesses can be cast to remove this potential issue.

6.17 B-002: Bearing pins not welded to bearing plate

6.17.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.17.2 Weld the pin to the bearing plate

This option is the same as was done on site, and requires the dowel to be procured in galvanised steel and welded to the galvanised steel base plate. This would provide benefit by stopping the dowel from falling into the recess into the headstock or sill beam and ensuring the minimum embedment into the bearing pad is achieved. A drawing of this option is shown below in Figure 6-32.

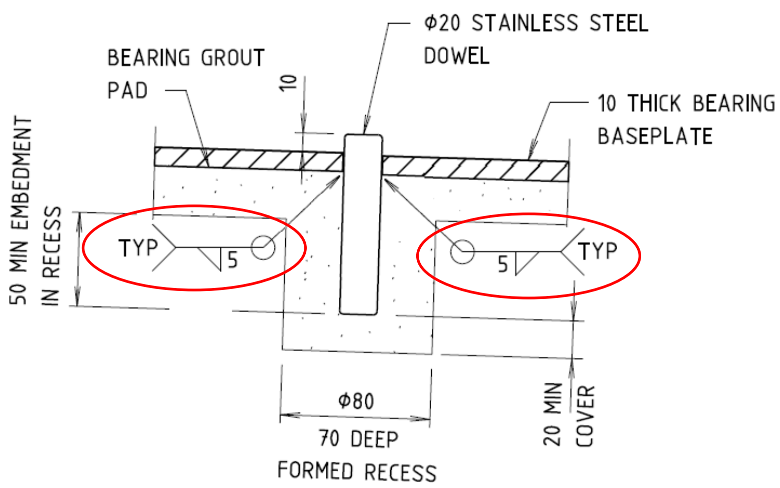

Figure 6-32 - Welded bearing pin and plate option

6.18 B-003: Bearing plate formwork access

6.18.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.18.2 Change packer location

The packers were installed in front of the bearing beneath the web of each deck as shown below in Figure 6-33 prior to installation of the bearings. Alternatively, the packers could be installed beneath the internal flanges of the deck units. This would improve the working area in front of the bearing but not between the bearing and the curtain wall. A drawing of this option is shown below in Figure 6-34.

Figure 6-33 - Packer location

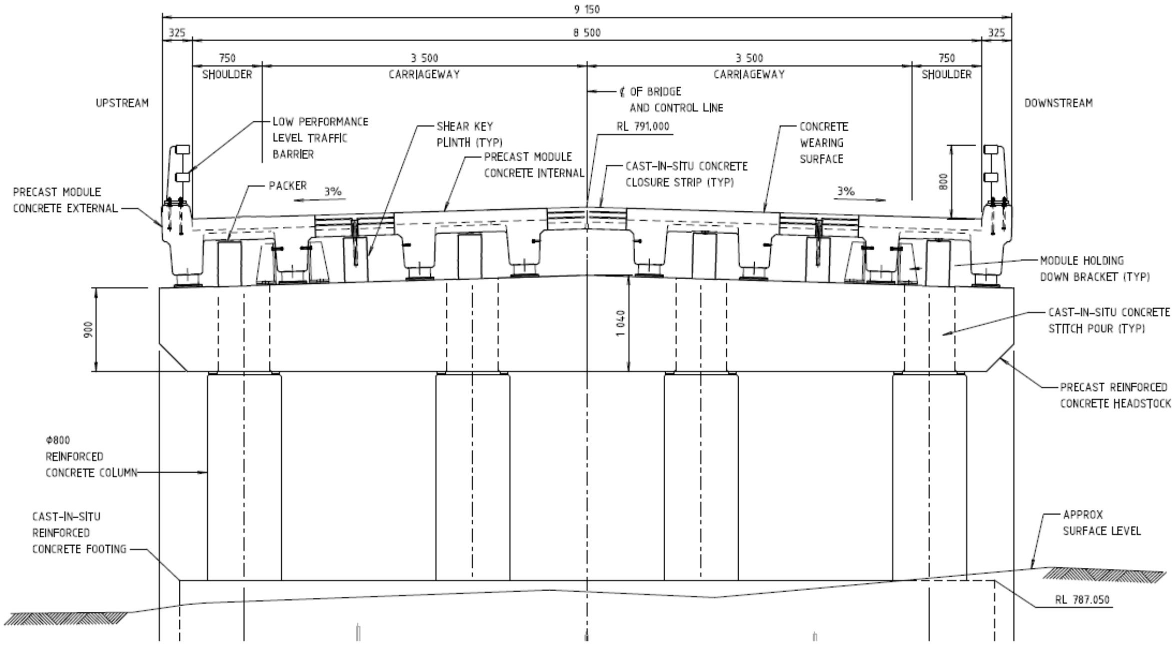

Figure 6-34 - Change packer location option

6.18.3 Install bearings before deck modules

The bearings for the bridge were installed after the deck units were landed onto packers. The bearings were intended to be installed before the decks as is typical is construction of the vast majority of concrete bridge, however this was unable to occur due to late realisation of issue B-002 and resultant re-ordering and welding of the bearing pins. It was not possible to delays installation of the deck units as the site needed to keep working and re-booking the crane may have added weeks onto the construction schedule. Installing the bearings after the deck units magnified the lack of room available for sealing of the formwork as the working space above the bearings was not available. This response proposed installing the bearings before the deck units so that the construction team can use the free room above the bearings to access the formwork area which will make installation easier.

6.18.4 Change bearing adjustment mechanism

The bearings are designed to be adjusted by M10 nylon screws below the bearing plate. The heads of the screws are located beneath the plate as shown in Figure 6-35, which means that the bearing needs to be adjusted to its final position before the formwork can be installed.

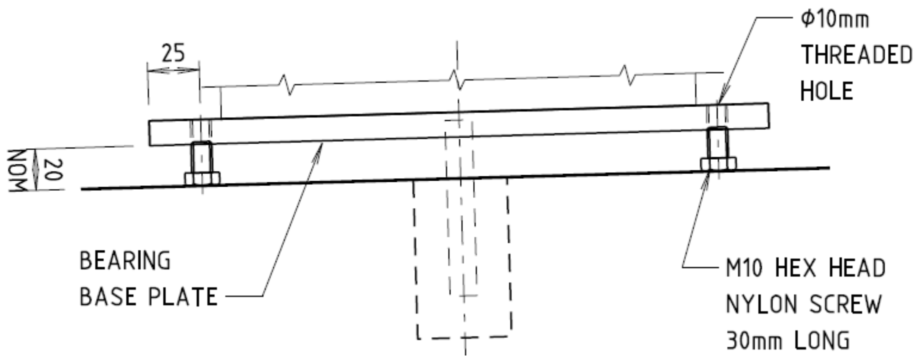

Figure 6-35 - Bearing adjustment mechanism (RMS, 2016)

This option would change the orientation of the screws such that the heads are above the plate as shown below in Figure 6-36. This would allow the full formwork frame to be installed and the bearing height adjusted later.

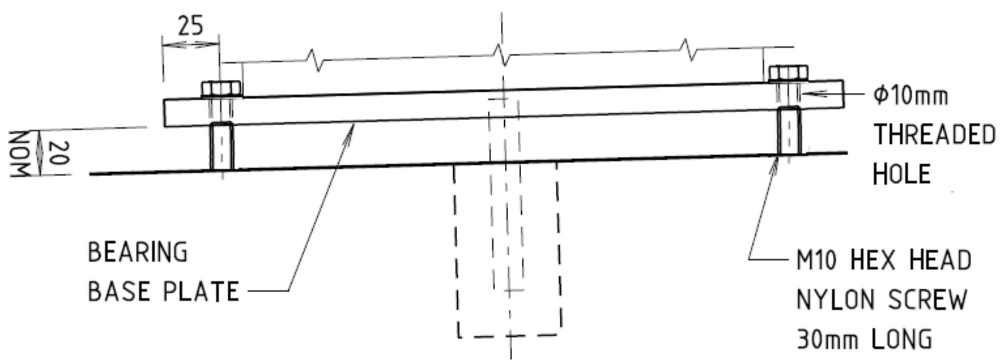

Figure 6-36 - Changed bearing adjustment mechanism option

6.18.5 Install back face formwork before the bearings

This option would require a change to construction methodology whereby the formwork between the bearing plate and the sill beam would be installed prior to installation of the bearings. The formwork would be sealed on the inside face of the formwork, removing the restricted working area. A drawing of this option is shown below in Figure 6-37.

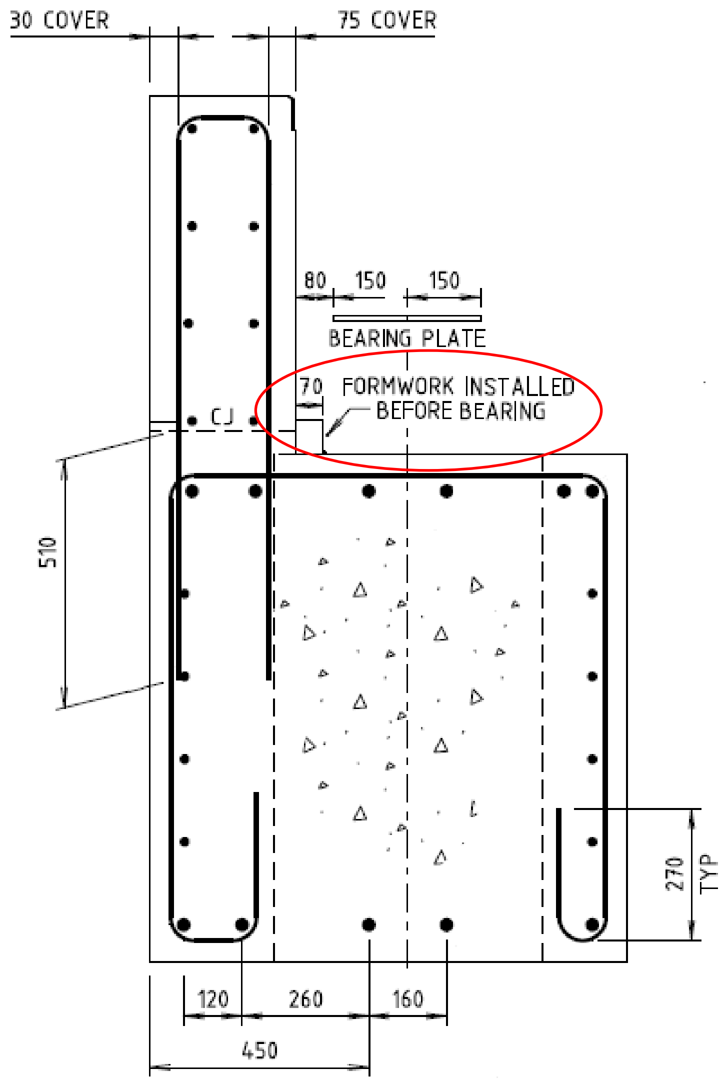

Figure 6-37 - Prior formwork installation option

6.19 J-001: Alignment of transverse joint

6.19.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.19.2 Precast full length and butt joint

This option would increase the length of the precast running deck in order to remove the requirement for a transverse concrete pour. Rigid formwork would be installed between the decks and held in place by the precast units and a full length longitudinal closure strip would be poured to stitch the deck units together and form a joint. Once poured, an unchanged small movement joint would be installed. A drawing of this option is shown below in Figure 6-38 and Figure 6-39.

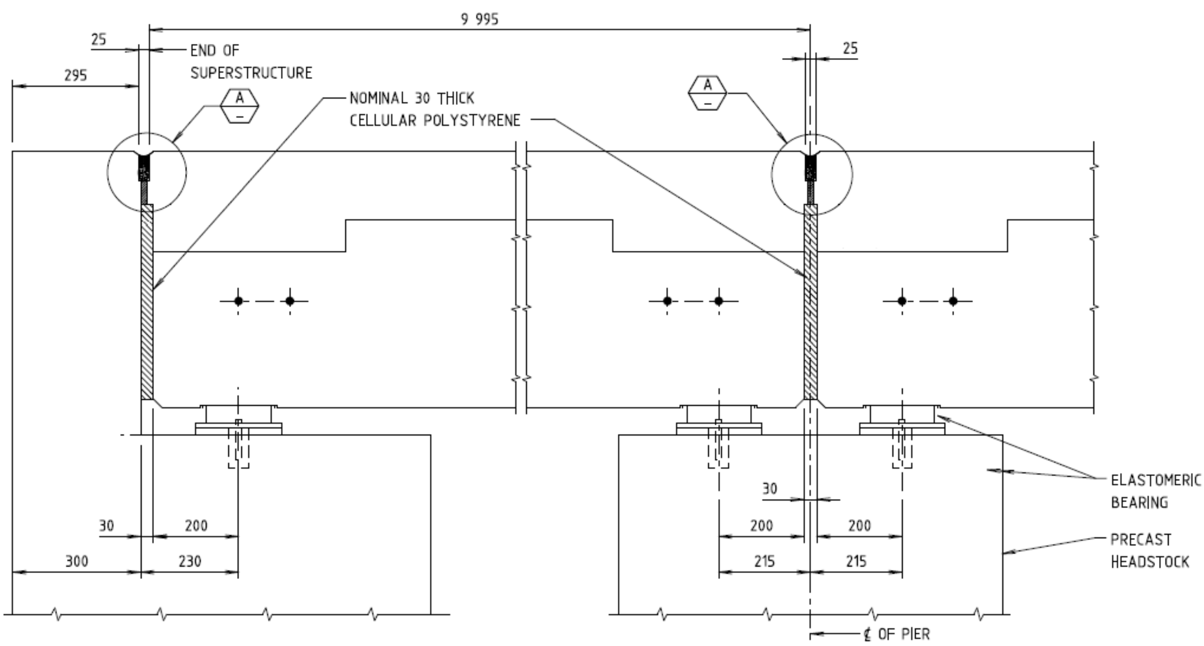

Figure 6-38 - Butt joint option overview

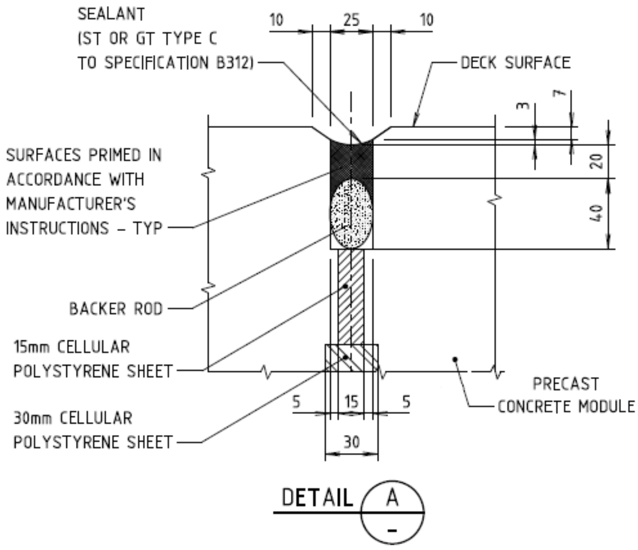

Figure 6-39 - Butt joint option detail

6.19.3 Precast one side of the joint only

This option is similar to the previous, except that only on side of the joint would be included in the precast unit. This would allow rigid formwork to be installed and fixed to the precasted side prior to pouring both a full length longitudinal closure strip and a single sided transverse closure strip. Adopting this option would result in a rigid, consistent surface to affix the formwork to in order to allow a straight joint, whilst retaining the transverse pour would provide a mechanism to correct any dislevellment between adjacent precast units. A drawing of this option is shown below in Figure 6-40 and Figure 6-41.

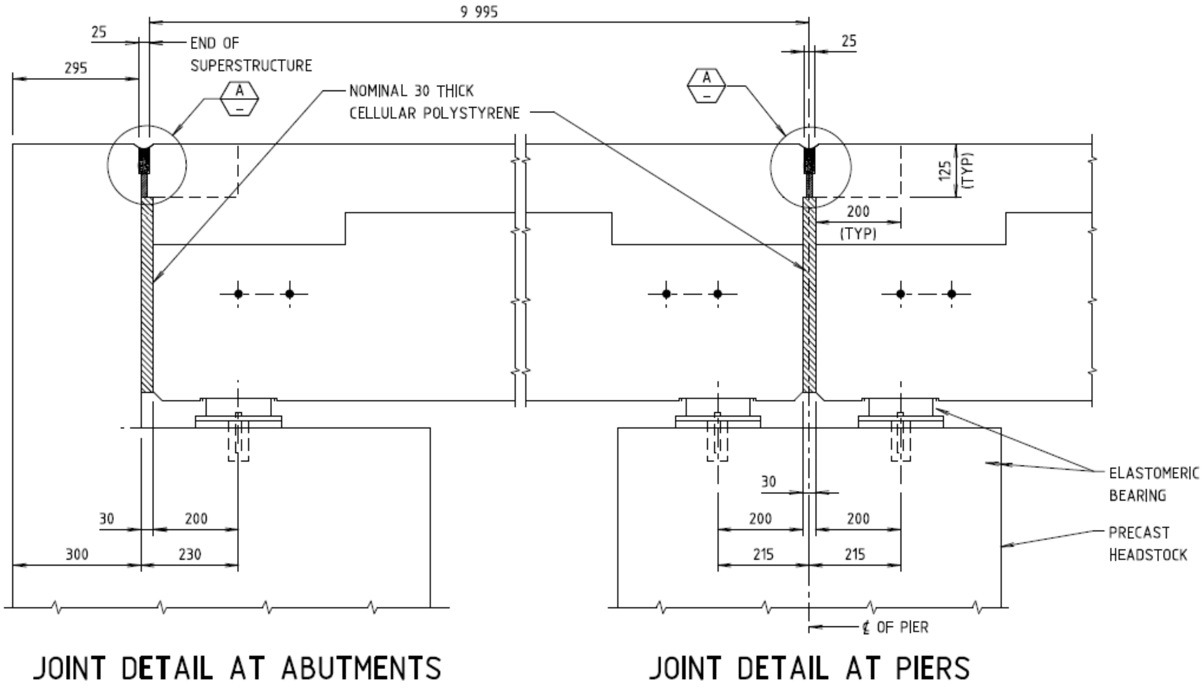

Figure 6-40 - Single side precast option

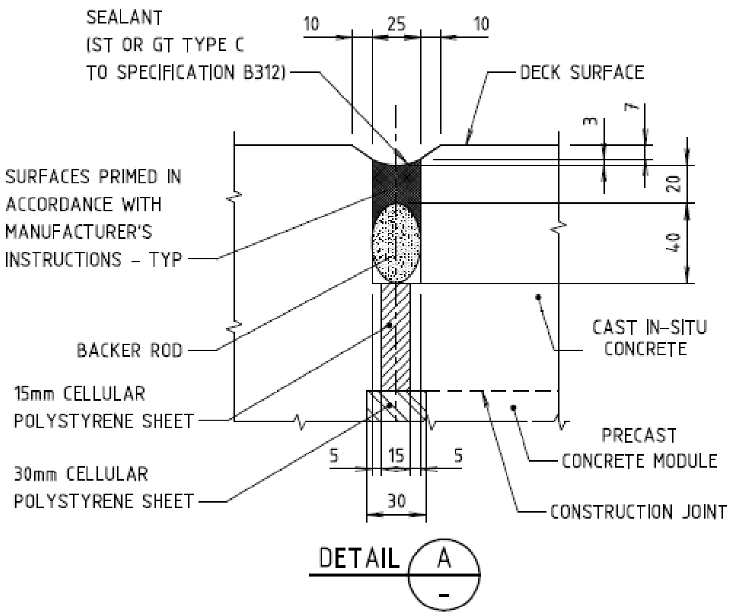

Figure 6-41 - Single side precast option joint

6.19.4 Delete joint

This option would remove the requirement for a joint between adjacent deck and allow the transverse closure strip to be poured without any requirement for formwork (except for on the deck soffit). The joint at the Abutment would be butt joint as discussed in the first option above for this issue. This would remove the issue with joint alignment between decks as there would be no joint to align, however it would also remove the mechanism by which the bridge allows expansion and contraction. A drawing of this option is shown below in Figure 6-42.

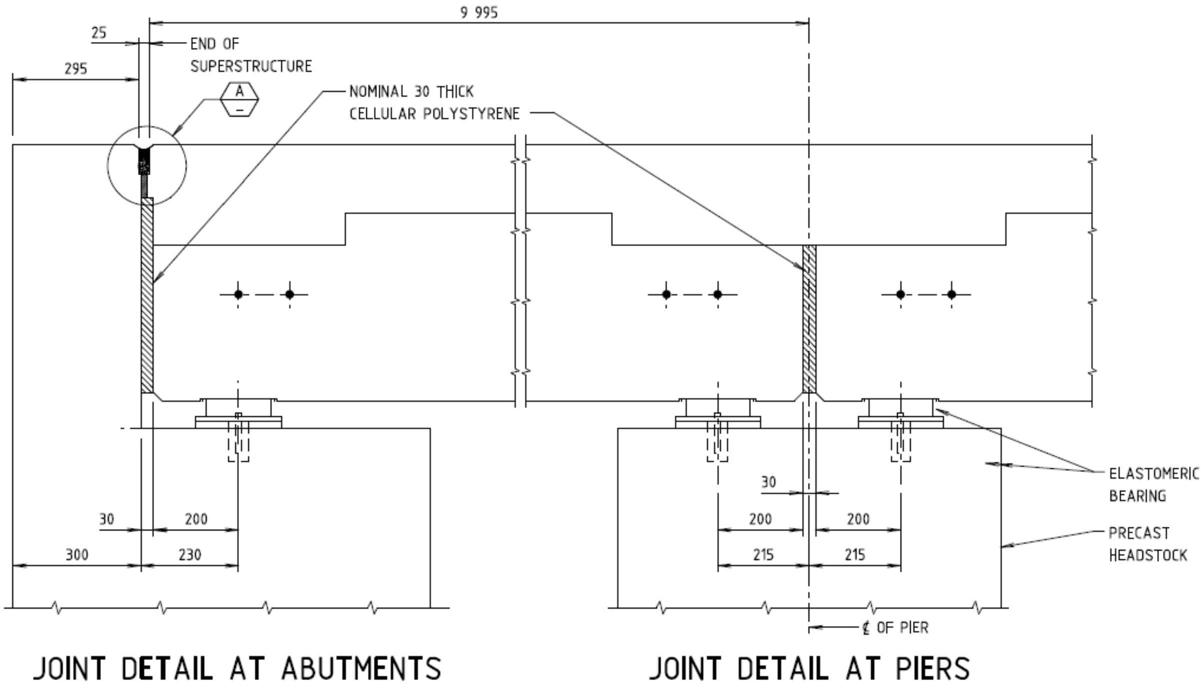

Figure 6-42 - Remove joint option

6.19.5 Multistage deck pour

This option proposes a methodology change rather than a design change. The deck at Bookookoorara Creek was poured in a continuous operation on one day, as having the concrete pump return a second time to pour the deck in two operations would have added an additional avoidable cost as well as lost time whilst waiting to be able to strip the joint formwork from the first pour. For future constructions which are able to utilise relatively local plant (it was an approximately 500km round trip for the concrete pump to Bookookoorara Creek), it may be feasible to pour the deck in two operations. This methodology change would allow rigid formwork to be fixed and propped from the central span deck (as this would not be poured) whilst pouring the end decks, then removed and formwork fixed to the poured decks to allow pouring of the central span.

6.20 HB-001: Binding and shearing of hold down bracket bolts

6.20.1 No action

The no action option simply maintains the status quo i.e. it does not propose any change to the current arrangement.

6.20.2 Specify requirement for thread lubricant

This option would require the inclusion of notes on the design drawings that instructs the construction team to apply lubricant/anti-seize to the threads of the bolts prior to installation such as Loctite 771 nickel based anti-seize (Loctite, 2016). Lubrication is the most effective and easily applied control to reduce the potential for thread galling (ASSDA, 2013). Implementation of this option would result in a negligible increase to project time and cost.

6.20.3 Use galvanised steel instead of stainless steel

This option would simply involve using galvanised steel (steel with a protective zinc coating) ferrules, bolts and fixings instead of stainless steel. Unlike stainless steel, galvanised steel is rarely susceptible to thread galling and does not need lubrication. It would be possible to use a stainless steel ferrule and galvanised steel bolt however this would result in accelerated corrosion of the galvanised steel due to galvanic corrosion of dissimilar metals (ASSDA, 2013). Black steel was also considered instead of galvanised steel, however, due to shrinking due to cooling due to manufacture, it is unable to be manufactured to the right tolerance required for threaded bolts hence is unsuitable (Metal Supermarkets, 2016)

7 Concept evaluation and recommendation

This section commences the analysis of the issues and concepts design presented in Chapter Six to determine the recommended concept. It is noted that this is an optional activity with reference to Appendix A: Project Specification. The method of analysis was consistent with Section 3.5, and involved implementation of the comparative evaluation matrix reproduced below as Table 7-1.

Table 7-1 - Evaluation matrix

Weighting		Option 1	Score	Option 2	Score	Option 3	Score
Time	- - %						
Quality	- - %						
Cost	- - %						
Safety	- - %						
Sum							

To use this matrix, the weightings are assigned based on the underlying driver of the issue. For the purposes of example, time shall receive a 70% rating and quality, cost and safety shall receive 10% each. The impact of each change is then assessed on the -5 to +5 scale as discussed earlier in this report. For the purposes of example, time, quality, cost and safety shall receive scores of +2, -2, +2 and -2 respectively.

The analysis then proceeds as follows

Time: 70% x +2 = +1.4

Quality: 10% x -2 = -0.2

Cost: 10% x +2 = +0.2

Safety: 10% x -2 = -0.2

The total score is the sum of these options (1.4 + (-0.2) + 0.2 + (-0.2)) = +1.6, therefore the option is assessed to have an overall minor to moderate positive impact. This process is repeated for all options, with the highest scoring option deemed the most suitable and recommended for further investigation and design by others.

7.1 AFT001 / PFT001: Spacing of reinforcement in footings

7.1.1 Evaluation

The primary driver of this issues is safety, namely the potential for worker injury by falling or slipping through the large spacing of the reinforcement cage. To reflect this, safety was assigned the highest weighting at 60% of total, followed by quality at 20% and cost and time at 10% each.

The evaluation matrix for this issue is shown in Table 7-2 and is discussed further in Section 7.1.2 .

Table 7-2 - AFT001 / PFT001 Evaluation Matrix

Weighting		No action	Score	Reduce spacing	Score	Construction joints	Score	Prefabricate reo cage	Score
Time	10%	0	0	-3	-0.3	-2	-0.2	3	0.3
Quality	20%	0	0	0	0	0	0	-5	-1
Cost	10%	0	0	-3	-0.3	-2	-0.2	-1	-0.1
Safety	60%	0	0	4	2.4	1	0.6	0	0
Sum		0		1.8		0.2		-0.8	

7.1.2 Discussion

As identified by Table 7-2, the evaluation indicated that reducing the spacing of the steel reinforcement is considered to be the most viable option for addressing this issue. The evaluation criteria and scores are discussed below, however the “no action” option is not discussed as the reasons for its scoring are considered self-explanatory.

7.1.2.1 Construction time

Reducing the spacing of the reinforcement would result in an increase in the required man-hours to tie the steel as a result of the increase in steel volume that is required to be tied. If spacing is reduced to 130 mm as indicated in Section 6.1.2, the volume of steel required will increase by approximately 55% which, for approximately three weeks of steel work, would result in approximately eight additional days being added to the construction program. The additional time associated with this option has resulted in a score of -3.

Inclusion of construction joints would disjoint the steel tying activity and result in lost time while waiting for each layer of concrete to cure to the point that it is able to be walked on. Additional time would also be required to strip formwork in three stages for Abutment A and two stages for each Pier footing. The additional time associated with this option is estimated to be six working days which has resulted in a score of -2.

Fabrication of the reinforcement cage off-site would result in significant time savings for the site work as the cage would be constructed off-site by others resulting in a saving of approximately 15 working days, however adjustment and installation of the cage is estimated to require up to five days works. This option would result in a reduction in site works by ten days and has received a time score of +3.

7.1.2.2 Quality

The steel tying activity per the current design resulted in no quality issues with regards to spacing, cover, lap length or quality of tie hence decreasing the spacing is not expected to result in any issues either. Similarly, inclusion of construction joints would not impact the reinforcement design and has also resulted in a “no change” assessment to the quality implications if this option were to be implemented.

The foundation conditions encountered at this site, particularly with regard to the size, depth and variability of rock, resulted in all the steel for the footings being bent on site. It would not be practically feasible to attempt to bend and fabricate a bespoke reinforcement cage for these conditions off-site as such a cage would be unlikely to fit the footing whilst achieving the required tolerance for cover and dimension. This difficulty resulted in this option receiving a -5 rating for quality to reflect the significant negative impact on quality.

7.1.2.3 Cost

Decreasing the reinforcement spacing and the subsequent increase in the quantity of steel to be tied and the resultant increase in time and man-hours would result in a negative cost impact if this option were to be implemented. Increasing the steel volume by 55% would also result in a material cost increase, the value of which has been estimated from the supply cost of steel under the current design. Additional site time would also result in increased hire durations for the site facilities (office, shed, ablution amenities) which has actual cost of approximately $100/day. The estimated cost for implementation of this option is shown in Table 7-3 and represents a 2.26% increase in the project budget which resulted in a comparative cost assessment rating of -3.

Table 7-3 - Cost estimate for reducing bar spacing option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	8	9.5	228	$ 59	$ 13,452
Steel	1	1	1	1	$ 10,000	$ 10,000
Site facility recovery	1	3	1	3	$ 100	$ 300
					TOTAL	$ 23,752

Implementation of the construction joint option would result in an increase in construction as discussed earlier (six days estimated), as well as needing the concrete pump to travel to site an additional four times to pump the concrete in individual layers (two additional visits for Abutment A and one additional visit per Pier). The estimated cost for implementation of this option is shown in Table 7-4 and represents a 1.52% increase in the project budget which resulted in a cost assessment rating of -2.

Table 7-4 - Cost estimate for construction joint option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	6	9.5	171	$ 59	$ 10,089
Concrete pump work	1	4	6	12	$ 180	$ 4,320
Concrete pump travel	1	4	1	2	$ 300	$ 1,200
Site facility recovery	1	3	1	3	$ 100	$ 300
					TOTAL	$ 15,909

Fabrication of the reinforcement cage off site by others is estimated to add an additional $20,000 to the cost of procuring this item, however it would result in a decrease in the amount of site works required as the cage would not be tied in-situ. The steel tying activity for the footings in question took three weeks, however five days are still expected to be needed to install and adjust the cage prior to commencing formwork activities. A $5,000 freight allowance has been added as the cages are substantial and, due to the need to be delivered to site intact, would not fit on a single standard truck like the plain bar used on site did. A mobile crane would also be required to unload the cages and lift them into position. The estimated cost for implementation of this option is shown in Table 7-5 and represents a 0.79% increase in the project budget which resulted in a cost assessment rating of -1.

Table 7-5 - Cost estimate for prefabrication cage option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	-10	9.5	-285	$ 59	$ -16,815
Freight	1	1	1	1	$ 5,000	$ 5,000
Steel fabrication	1	1	1	1	$ 20,000	$ 20,000
Crane	1	1	5	5	$ 230	$ 1,150
Site facility recovery	1	-10	1	-10	$ 100	$ -1,000
					TOTAL	$ 8,835

7.1.2.4 Safety

This issue was noted due to safety concerns, hence safety is the most important issue to consider and has received the highest weighting in Table 7-2. Reducing the spacing of the reinforcing steel is expected to result in positive safety implications as it addresses the key safety risk by reducing the potential for a worker to lose their footing and fall (+5 impact). There is a slight increase in the risk of abrasion or similar minor injuries due to the increase in volume of steel to be tied however this risk is expected to be minimal (-1 impact). The resultant score for this option is +4, representing a significant safety improvement if this option were to be implemented.

The inclusion of construction joints does not address the underlying mechanism of the safety risk (potential for fall), but it does limit the size of the fall. This option may result in a decreased severity of the injury if the falling risk is realised and has received a score of +1 to reflect this minor safety improvement.

Fabricating the reinforcement cage off site is not expected to result in any safety benefits as, although the amount of time the workers are working on the cage is reduced, the reinforcement layout and size of the potential fall has not changed. This situation has resulted in a score of zero.

7.1.3 Recommendation

The recommended option to address this issue is to reduce the size and spacing of the reinforcement to N16 bars at 130 centres. This option would result in a reduction of the safety risk by reducing the size of the gap between adjacent reinforcing bars which reduces the potential for a worker to lose their footing and fall causing injury. Additional time and cost is expected to result if this option is implemented, however there are no anticipated impacts on quality.

7.2 AFT-002: Width of Abutment B retaining wall

7.2.1 Evaluation

The primary driver of this issues is quality, namely the adequacy of compaction of pavement material beneath the back of the sill beam. To reflect this, quality was assigned the highest weighting at 60% of total, followed by cost at 20% and time and safety at 10% each.

The evaluation matrix for this issue is shown in Table 7-6 and is discussed further in Section 7.2.2.

Table 7-6 - AFT-002 Evaluation matrix

Weighting		No action	Score	Increase wall thickness	Score	No fines concrete	Score	Increase height of slab	Score
Time	10%	0	0	-1	-0.1	0	0	-3	-0.3
Quality	60%	0	0	5	3	3	1.8	5	3
Cost	20%	0	0	-1	-0.2	0	0	-1	-0.2
Safety	10%	0	0	0	0	0	0	0	0
Sum		0		2.7		1.8		2.5	

7.2.2 Discussion

As identified by Table 7-6, the evaluation indicated that increasing the thickness of the retaining wall to the same or greater width as the sill beam is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the “no action” option is not discussed as the reasons for its scoring are considered self-explanatory.

7.2.2.1 Time

Increasing the thickness of the retaining wall resulted in one additional day of site work during construction of the subject bridge. The additional time is reflective of the time spent installing additional four transverse N16 bars and pouring and finishing an additional 7.4 m^3 of concrete. The transverse retaining wall is cast monolithically with the longitudinal retaining wall that supports the wing wall, hence no additional time installing or removing formwork is expected. These changes have resulted in a time score of -1.

Using no fines concrete as backfill does not require any change to the retaining wall arrangement hence there will be no additional time in construction of this element. The no fines concrete will be installed in lieu of pavement gravel below the sill beam, and action that is expected to take 0.5 days, however the time taken to install this material will be offset by a reduction in time required to place and compact the rest of the pavement material. This balancing of time has resulted in a “no change” assessment of construction time for this option and a time score of 0.

Increasing the height of the footing to remove the retaining wall will require a significant increase in the quantity of reinforcing steel to be bent and tied; a change that is expected to add two days to the construction program. An additional two days (estimated) will also be required to construct and remove a larger and more robust formwork system to contain the additional 38 m^3 of concrete required for this option. There is not expected to be any overall increase in construction time due to the need to pour the larger footing as additional time taken to pour this footing would be offset by the time that is no longer needed to pour the retaining wall. The changes discussed in this paragraph have resulted in this option receiving a time score of -3.

7.2.2.2 Quality

All of the options presented remove the quality concern associated with adequacy of compaction as there is no longer any pavement material to compact beneath the sill beam. There were no issues observed with placing, compacting and finishing any of the concrete during the bridge construction hence no issues are expected to occur as a result of the changes required by the retaining wall or slab modification options. This quality environment has resulted in a +5 score for both options.

Using no fines concrete as backfill does not require any change to the retaining wall arrangement, however it has a positive impact on quality by using a controlled material to backfill below the sill beam. No fines concrete is self-compacting hence it is practically assured of meeting the specifications required for backfill of the abutments. Additionally, use of this high permeability material will assist in facilitating proper drainage of water from behind the abutment to relieve hydrostatic pressure and reduce the risk of wall failure. Nevertheless, when compared to the other option, there is potential for use of no-fines concrete to be overlooked and the quality benefits not be realised in future construction. Uncertainty has resulted in a +3 quality score for this option

7.2.2.3 Cost

As discussed previously, increasing the thickness of the retaining wall is expected to result in an additional one day of site work to tie the additional reinforcement and place the additional concrete. For the purposes of quantity estimation, a 15% wastage allowance has been included when calculating the quantity of concrete required to reflect the concrete lost in clearing the concrete pump lines. Placing the concrete is expected to take an additional one hour, a reality that has resulted in an additional hour of concrete pump hire. While implementation of this option will require additional materials, their cost is offset somewhat by reducing the quantity of pavement gravel that need to be purchased, placed and compacted – this is reflected by the “Gravel cost reduction” line. The estimated cost impact of this option is shown in Table 7-7 and represents a 0.44% increase in total project cost which has resulted in a score of -1.

Table 7-7 - Cost estimate for retaining wall thickness option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	1	9.5	28.5	$ 59	$ 1,682
Concrete	7.4	1	1	7.4	$ 286	$ 2,103
Steel allowance	1	1	1	1	$ 1,500	$ 1,500
Concrete pump work	1	1	6	6	$ 180	$ 1,080
Gravel cost reduction	7.4	1	1	7.4	$ -250	$ -1,838
Site facility recovery	1	1	1	1	$ 100	$ 100
					TOTAL	$ 4,626

Using no fines concrete in lieu of gravel to fill the area below the sill beam overhang does not require any change to the existing footing arrangement, hence the only costs associated with this option are minimal. It has been estimated that 6.9 m^3 of no fines concrete will be required based on a triangular placement arrangement behind the retaining wall. The concrete will be placed using the chute from the concrete agitator truck, removing the need for a concrete pump. This activity is expected to take about 2 hours, or 0.25 days. The estimated cost impact of this option is shown in Table 7-8 and represents a 0.07% increase in total project cost, an impact which is so small that it has resulted in a score of 0.

Table 7-8 - Cost estimate for no fines concrete option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	0.25	9.5	7.125	$ 59	$ 420
No fines concrete	6.9	1	1	6.9	$ 286	$ 1,973
Gravel cost reduction	6.9	1	1	6.9	$ -250	$ -1,725
Site facility recovery	1	0.25	1	0.25	$ 100	$ 25
					TOTAL	$ 694

Increasing the height of the footing is estimated to result in an additional four days of working time after consideration of the time that would no longer be required to tie, form and pour an independent retaining wall. Additional reinforcing steel would be required at an estimated cost of $3,500. This option would also require in an additional 32.2 cubic metres (including 10% wastage allowance) being poured, placed and finished, however the concrete would be placed in one operation which would remove the travel charge that would have been incurred if the footing and retaining wall were poured separately. The estimated cost impact of this option is shown in Table 7-9 and represents a 1.11% increase in total project cost which has resulted in a score of -1.

Table 7-9 - Cost estimate for increase to footing height option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	4	9.5	114	$ 59	$ 6,726
Concrete	38.0	1	1	38.0	$ 286	$ 9,206
Steel allowance	1	1	1	1	$ 3,500	$ 3,500
Concrete pump work	1	1	1	1	$ 180	$ 180
Concrete pump travel	1	1	1	1	$ -300	$ -300
Gravel cost reduction	38.0	1	1	38.0	$ -250	$ -9,511
Site facility recovery	1	4	1	4	$ 400	$ 400
					TOTAL	$ 11,266

7.2.2.4 Safety

There is no appreciable change to safety through implementation of any of the identified options, hence all options have received a safety score of 0.

7.2.3 Recommendation

The recommended option to address this issue is to increase the thickness of the retaining wall. This option will result in a minor increase in project time and cost but will have a significant positive impact on quality.

7.3 AFT-003: Depth of Abutment B retaining wall

7.3.1 Evaluation

The primary drivers of this issues quality and safety, namely ensuring that the wing wall is adequately supported to ensure that it is not damage during pavement construction or liable to fall during bridge or pavement construction. To reflect this, quality was assigned the highest weighting at 60% of total, followed by cost at 20% and time and safety at 10% each.

The evaluation matrix for this issue is shown in Table 7-10 and is discussed further in Section 7.3.2.

Table 7-10 - AFT-003 Evaluation Matrix

Weighting		No action	Score	Extend retaining wall	Score
Time	10%	0	0	-2	-0.2
Quality	40%	0	0	5	2
Cost	10%	0	0	-2	-0.2
Safety	40%	0	0	5	2
Sum		0		3.6	

7.3.2 Discussion

As identified by Table 7-10, the evaluation indicated that using increased the thickness of the retaining wall to the same or greater width as the sill beam is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the "no action" option is not discussed as the reasons for its scoring are considered self-explanatory.

7.3.2.1 Time

Increasing the depth of the retaining wall to support the full length of the wing wall resulted in a two day increase to the construction project. This time was needed to excavate and pour a blinding layer (one day), install formwork (one day), pour and finish concrete (0.5 days) and strip formwork (0.5 days). This change has resulted in a time score of -2.

7.3.2.2 Quality

Constructing a mass concrete retaining wall extension will assure that the wing wall is fully supported on adequately compacted material, thereby reducing the potential for instability. Removing the need to place and compact material beneath a suspended wing wall will also reduce the potential for damage to the structure during roadworks. These positive quality changes have resulted in a score of +5.

7.3.2.3 Cost

Extending the retaining wall on both sides of Abutment B resulted in an additional three days of site work as discussed earlier. The wall is mass concrete hence no reinforcement is needed, however 2.5 m^3 of concrete (inclusive of 10% wastage allowance) will be needed. It is assumed that existing formwork panels will be used at no cost, however an allowance has been included for fixings such as nail, screw bolts and sealant. The estimated cost impact of this option is shown in Table 7-11 and represents a 0.55% increase in total project cost which has resulted in a score of -2.

Table 7-11 - Cost estimate for increasing retaining wall option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	3	9.5	57	$ 59	$ 5,045
Concrete	2.5	1	1	2.5	$ 186	$ 468
Fixings	1	1	1	1	$ 100	$ 100
Site facility recovery	1	2	1	2	$ 100	$ 200
					TOTAL	$ 5,812

7.3.2.4 Safety

Increasing the depth of the retaining wall but casting a mass concrete extension does not introduce any new activities of safety risks into the site hence there will be no negative change to construction safety. Supporting the wing wall for its full length will increase its stability and reduce the potential for it to fall over and crush a worker. This represents a significant increase to construction safety and has resulted in a safety score of +5.

7.3.3 Recommendation

The recommended option to address this issue is to increase the depth of the retaining wall. This option will result in a minor increase in project time and cost but will have a significant positive impact on quality and safety.

7.4 AFT-004: Abutment A drainage

7.4.1 Evaluation

The primary driver of this issues is quality, namely ensuring proper drainage from behind the Abutment to reduce future maintenance issue. To reflect this, quality was assigned the highest weighting at 60% of total, followed by cost at 20% and time and safety at 10% each.

The evaluation matrix for this issue is shown in Table 7-12 and is discussed further in Section 7.4.2.

Table 7-12 - AFT-004 Evaluation matrix

Weighting		No action	Score	Proprietary drainage	Score	Weepholes in sill beam	Score	Weephole in wing wall	Score
Time	10%	0	0	-1	-0.1	-1	-0.1	-1	-0.1
Quality	60%	0	0	5	3	3	1.8	3	1.8
Cost	20%	0	0	-1	-0.2	-1	-0.2	-1	-0.2
Safety	10%	0	0	0	0	0	0	0	0
Sum		0		2.7		1.5		1.5	

7.4.2 Discussion

As identified by Table 7-12, the evaluation indicated that installing a proprietary drainage system to direct water out the back of the abutment is considered to be the most viable option for addressing this issue. This response was also implemented on site during construction of the trial bridge in response to this issue. The evaluation criteria and scores are discussed below, however the “no action” option is not discussed as the reasons for its scoring are considered self-explanatory.

7.4.2.1 Time

Installing a proprietary drainage system as discussed earlier took a team of three workers approximately two hours or 0.25 days. Including weep holes in the sill beam outside of the stitch area with the abutment would not result in any increase in site construction time, however a proprietary drainage system would still be required to direct the water to the weep holes which would take 0.25 days. This minor increase in time has resulted in both of these options receiving a time score of -1.

The proposed weep holes in the wing wall would be included within the stitch area to the sill beam and footing in an attempt to minimise the distance water would need to travel through the subsoil drainage system. This would result in a minor increase in site construction time as, in addition to installing the drainage system, it would take time to cut and install the 75 mm PVC pipe into the stitch area. The combined increase in construction time for these actions is estimated to by 0.5 days which has resulted in this option receiving a time score of -1.

7.4.2.2 Quality

All of the options presented address the underlying issue associated with relieving hydrostatic pressure which may result from water build up behind the Abutment. Installing the proprietary drainage system alone resulted in a significant quality increase without compromising any of the other activities or elements on site, hence this option has received a rating of +5.

The inclusion of weep holes cast in to either the sill beam or wing wall in conjunction with the proprietary drainage system would deliver similar benefits to those discussed in the prior paragraph, however a quality risk is present in that it may be difficult to achieve proper concrete penetration and compaction in the area below the weep hole. This concern has resulted in both of these options receiving a quality score of +3.

7.4.2.3 Cost

Purchasing of the materials required for the proprietary drainage system as shown in Figure 6-6 has a cost of approximately $2,000 (slightly rounded to obscure supplier rates from actual quote). This option was implemented during the bridge construction and took a team of three workers approximately 0.25 days, the cost of which is included in the estimate contained in Table 7-13. Implementation of this option will result in an estimated 0.23% increase to the total project cost which has led to a cost score of -1.

Table 7-13 - Cost estimate for proprietary drainage option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	0.25	9.5	7.125	$ 59	$ 420
Cord drain and megaflo	1	1	1	1	$ 2,000	$ 2,000
Site facility recovery	1	0.25	1	0.25	$ 100	$ 25
					TOTAL	$ 2,445

Inclusion of weep holes into sill beam is estimated to result in an increase to the cost of the precast element of $420 to account for the estimated 0.25 days required to install the weep hole formers. The cost estimate for this option is shown in Table 7-14 and includes the cost of the proprietary drainage system as discussed previously. Implementation of this option will result in an estimated 0.27% increase to the total project cost which has led to a cost score of -1.

Table 7-14 - Cost estimate for sill beam weep hole option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	0.25	9.5	7.125	$ 59	$ 420
Precast cost increase	1	1	1	1	$ 420	$ 420
Cord drain and megaflo	1	1	1	1	$ 2,000	$ 2,000
Site facility recovery	1	0.25	1	0.25	$ 100	$ 25
					TOTAL	$ 2,865

Inclusion of weep holes into the precast wing wall would result in the same cost increases as discussed for the sill beam weep hole option, however additional costs would also be incurred to supply and install weep hole formers into the stitch pour area. The forms would consist of 4 x 600mm of 75 mm PVC pipe which would take an estimated 30 minutes each to install. The resultant cost increase for this option is shown below in Table 7-15 and represents a 0.30% increase to total project cost and a cost score of -1.

Table 7-15 - Cost estimate for wing wall weep hole option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	0.5	9.5	14.25	$ 59	$ 841
75mm PVC pipe	2	1	1	2	$ 5	$ 12
Precast cost increase	1	1	1	1	$ 250	$ 250
Cord drain and megaflo	1	1	1	1	$ 2,000	$ 2,000
Site facility recovery	1	0.25	1	0.25	$ 100	$ 25
					TOTAL	$ 3,128

7.4.2.4 Safety

There is no appreciable change to safety through implementation of any of the identified options, hence all options have received a safety score of 0.

7.4.3 Recommendation

The recommended option to address this issue is to specify an indicative proprietary drainage system on the CBS drawings to use behind Abutments where no weep holes are provided. This option will result in a significant quality improvement by providing a mechanism for controlled drainage of water and relief of hydrostatic pressure to reduce the potential for Abutment wall failure. Minor increases in cost and time would result, however this change is considered to be of lesser importance that the quality benefits. There is no foreseeable impact on construction or operation safety as a result of implementation of this option.

7.5 AFT-004 / PFT-002: Dowel hole size to footings

7.5.1 Evaluation

The drivers of this issues are quality and cost, specifically ensuring that the specified hole size is able to be drilled on site at an economical rate. To reflect this, quality and cost are equally weighted at 40% each, followed by time and safety at 10% each.

The evaluation matrix for this issue is shown in Table 7-16 and is discussed further in Section 0.

Table 7-16 - AFT004 / PFT002 Evaluation matrix

Weighting		No action	Score	Reduce hole and dowel size	Score	Remove dowels	Score
Time	10%	0	0	-1	-0.1	1	0.1
Quality	40%	0	0	2	2	-5	-2
Cost	40%	0	0	-1	-0.4	1	0.4
Safety	10%	0	0	0	0	0	0
Sum		0		0.3		-1.5	

7.5.2 Discussion

As identified by Table 7-16, the evaluation indicated that reducing the hole size to 50 mm and installing and increased quantity of N24 dowels is considered to be the most viable option for addressing this issue. This response was also implemented on site during construction of the trial bridge in response to this issue. The evaluation criteria and scores are discussed below, however the “no action” option is not discussed as the reasons for its scoring are considered self-explanatory.

7.5.2.1 Time

As discussed earlier, the site response to this issue was to implement the reduced hole size and dowel size option. Drilling of these 44 x 50 diameter holes took a full day, or 9.5 hours, however this is expected to be the same time as would have been taken to drill the larger holes per the design as drilling of larger diameter holes is less efficient.

Nonetheless, there is expected to be one additional day of work associated with this option to reflect the time taken to place and grout the additional dowels; a change which has resulted in a time score of -1.

Removal of the dowels would result in a two day decrease to construction time as there would no longer be a requirement to drill the holes (one day) or insert and grout the dowels (one day). This change has resulted in a time score of +1.

7.5.2.2 Quality

Reducing the diameter of the hole to allow it to be easily drilled will result in a better quality outcome than if the holes were otherwise unable to be drilled. When considered in absolute terms, it would have been possible to procure an appropriately sized rock drill from an alternate supplier to install the dowels per the design, however this would have meant working with an unfamiliar contractor with potentially inferior quality systems. There is advantage in construction to working with experienced contractors – a reality which has resulted in this option receiving a quality score of +2.

Without being privy to the design assumptions, it is reasonably concluded that the designers were concerned about sliding of the abutment and resultant bridge failure which lead to the decision to specify dowelled connection to rock. Removal of the dowels would result in a significant negative impact on quality due to loss of shear connection between the footing and the natural rock, hence this option has received a quality score of -5.

7.5.2.3 Cost

As discussed in Section 7.5.2.1, reducing the size of the holes and dowels is expected to result in a time increase of one working day, however increasing the quantity of these items will result in an increase to the quantity of dowel and grout materials required. A cost estimate for implementation of this option is provided in Table 7-17, a change which represents a 0.24% increase in project cost and is reflected in a cost score of -1.

Table 7-17 - Cost estimate for reduced hole size option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	1	9.5	28.5	$ 59	$ 1,682
Grout (m^3)	3	1	1	3	$ 65	$ 195
N24 dowels	22	1	1	22	$ 25	$ 550
Site facility recovery	1	1	1	1	$ 100	$ 100
					TOTAL	$ 2,527

Removal of the dowels would result in a decrease in construction time and material cost as the holes would no longer need to be drilled or the dowels installed. Table 7-18 shows the change in cost associated with this option, a change which represents a 0.71% reduction in project cost and is reflected in a cost score of +1.

Table 7-18 - Cost estimate for removal of dowels

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer reduction	3	-1	9.5	-28.5	$ 59	$ -1,682
Grout reduction	3	-1	1	-3	$ 65	$ -195
Drilling reduction	1	-1	1	-1	$ 5,000	$ -5,000
N24 dowel reduction	-19	1	1	-19	$ 25	$ -475
Site facility recovery	1	-1	1	-1	$ 100	$ -100
					TOTAL	$ -7,452

7.5.2.4 Safety

There is no appreciable change to safety through implementation of any of the identified options, hence all options have received a safety score of 0.

7.5.3 Recommendation

The recommended option to address this issue is reduce the size of the holes and dowels in conjunction with increasing the quantity of same. This option will result in minor increase to construction time and cost, however the positive quality impacts are significant enough to warrant the change when compared to the option of removing the dowels altogether. There is no foreseeable impact on construction or operation safety as a result of implementation of this option.

7.6 SB-001: Potential damage to Abutment Curtain Wall concrete

7.6.1 Evaluation

The key driver of this issue is quality and cost, specifically ensuring that the abutment curtain wall is concrete is not damaged by vehicle impact loads. To reflect this, quality is weighted at 60% each, followed by cost at 20% and time and safety at 10% each.

The evaluation matrices for this option is shown in Table 7-19 and Table 7-20 and is discussed further in Section 7.6.2.

Table 7-19 - SB-001 Evaluation Matrix One

Weighting		No action	Score	Rigid pavement	Score	Protection angle	Score
Time	10%	0	0	-1	-0.1	0	0
Quality	60%	0	0	2	1.2	4	2.4
Cost	20%	0	0	2	0.4	-1	-0.2
Safety	10%	0	0	0	0	0	0
Sum		0		1.5		2.2	

Table 7-20 - SB-001 Evaluation Matrix Two

Weighting		High strength concrete	Score	Approach slab	Score
Time	10%	0	0	-3	-0.3
Quality	60%	1	0.6	4	2.4
Cost	20%	0	0	-3	-0.6
Safety	10%	0	0	0	0
Sum		0.6		1.5	

7.6.2 Discussion

As identified by Table 7-19 and Table 7-20, the evaluation indicated that including a protection angle on the sill beam is considered to be the most viable option for addressing this issue. The evaluation criteria and scores are discussed below, however the "no action" option is not discussed as the reasons for its scoring are considered self-explanatory.

7.6.2.1 Time

Modification of the bridge approach from a flexible pavement to a rigid pavement is estimated to result in an additional two days construction work to account for the time required for mixing of the additive and a rework allowance for if the activity does not go according to plan. This additional time has resulted in a time score of -1.

Installation of a steel protection angle to the sill beam will result in a minor increase in precast construction time but it will have no impact on site construction time. The focus of this report is on site work, so this option has received a time score of 0 as no site change will be realised. Similarly, use of high strength concrete has also received a time score of 0 as this is a change made in the precast process and will have no impact on site activities.

The approach slab option will need to be cast in-situ which is a labour intensive option. It is estimated that this activity will add five working days to the program based on three days to tie the cages and two days to form, pour and strip the slabs. This increase in working time has resulted in a time score of -3.

7.6.2.2 Quality

Pavement stabilisation is a specialised activity that requires experience to properly execute. Regional local government authorities, particularly those with a small workforce or rate base, may lack the skills required to carry out this exercise to the require quality standards with regards to binder application and mixing, compaction etc., however it is assumed that if this were the case then a competent contractor would be engaged to complete the task. Nonetheless, if properly executed, a rigid pavement would result in a more predictable pavement structure with a more controlled rate of failure than a flexible pavement. This change has resulted in a quality score of +2 because even though a quality risk is present, the benefit of the option (if performed competently) will result in a positive quality outcome.

A steel protection angle, as the name suggests, provides mechanical protection to the concrete against breakout. This is a commonly applied technique hence no quality issues are expected during the construction of the element and, when in operation, overall construction quality will be increased as the concrete is physically protected to reduce failure potential. These characteristics have resulted in a quality score of +4.

High strength concrete will result increased capacity of the unprotected face to resist breakout which is a positive impact, however specifying different concrete strengths within the same element increases the chance of error during the precast process. To reduce the risk of error, it is proposed to cast the entire sill beam out of high strength concrete. Despite this material change, concrete breakout is primarily due to tensile forces which is improved little by specifying a concrete of higher compressive strength. The result of this change has been assessed as having a quality score of +1.

Inclusion of a concrete approach slab will result in a reduction to the risk of breakout to the sill beam by providing a controlled and smooth surface transition to enter the bridge deck however, if left unprotected, the front of the approach slab will be prone to breakout instead. Figure 6-10 shows the approach slab arrangement and includes a 90 x 90 steel protection angle to protect the front of the slab, hence this change has received a quality score of +4 consistent with the protection angle option discussed earlier in this section.

7.6.2.3 Cost

Specifying a rigid approach pavement rather than a flexible pavement will result in a higher upfront cost due to the need to procure additional plant and material needed to undertake stabilisation. A cost estimate has been prepared on the assumption of stabilisation of two 150 mm subgrade layer for 50 m of bridge approach, with costs estimate per cubic metre of compacted material. The resultant change to upfront cost is shown in Table 7-21, however it is widely accepted that rigid pavement will result in lower ongoing maintenance cost and increased duration between rehabilitation when compared to a flexible pavement. For this reason, this option has received a cost score of +2 because, even though the upfront cost impact is negative, this option has an overall positive cost impact.

Table 7-21 - Upfront cost estimate for rigid pavement option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Stabilised pavement	285	1	1	285	$ 180	$ 51,300
Flexible pavement reduction	285	1	1	285	$ -150	$ -42,750
Site facility recovery	1	1	1	1	$ 100	$ 100
					TOTAL	$ 8,650

Installation of the protection angle will incur a material cost for the steel angle and protection studs, as well as the cost associated with an estimated 0.5 days' work for a single labourer during the precast process. Table 7-22 shows the change in cost associated with this option, a change which represents a 0.19% increase in project cost and is reflected in a cost score of -1.

Table 7-22 - Cost estimate for protection angle option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
90 x 90 x 10 EA	39.6	1	1	39.6	$ 59	$ 2,336
Shear studs	32	1	1	32	$ 10	$ 320
Precast time increase	1	0.5	6	3	$ 59	$ 177
					TOTAL	$ 2,833

Use of high strength concrete is not expected to result in any appreciable variation to working time or material cost as the quantity of material to be purchased and placed in not proposed to change. It has therefore been estimated that there will be no cost impact associated with this option which is reflected in a cost score of 0.

The construction of two approach slabs is a significant activity which will add an estimated five days to the construction program. An additional 37 cubic metres of concrete (inclusive of 10% wastage allowance) will be required as well as an estimated $6,000 of additional reinforcing steel. An allowance for formwork and falsework has been included, the value of which is minimal as only the back face will need to be formed with materials assumed to consist of 12mm form ply with 90 x 45 mm timber falsework. Table 7-23 shows the change in cost associated with this option, a change which represents a 2.58% increase in project cost and is reflected in a cost score of -3.

Table 7-23 - Cost estimate for approach slab option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	5	9.5	142.5	$ 59	$ 8,408
Concrete	37	1	1	37	$ 286	$ 10,571
Concrete pump work	1	1	6	6	$ 180	$ 1,080
Concrete pump travel	1	1	1	1	$ 300	$ 300
Steel	1	1	1	1	$ 6,000	$ 6,000
Formwork/falsework allowance	1	1	1	1	$ 200	$ 200
Site facility recovery	1	5	1	5	$ 100	$ 500
					TOTAL	$ 26,097

7.6.2.4 Safety

There is no appreciable change to safety through implementation of any of the identified options, hence all options have received a safety score of 0.

7.6.3 Recommendation

The recommended option to address this issue is include a protection angle to the front face corner of each sill beam. Although this option will result in minor increase to precast cost, the significant increase to overall quality is sufficient to mitigate this change. There is no foreseeable impacts on site construction time or safety as a result of implementation of this option.

7.7 SB-002: Roughness of curtain wall running surface

7.7.1 Evaluation

The key driver of this issue is safety, specifically ensuring that a suitably slip resistant surface is provided over the enter bridge concrete surface to reduce the risk of a traffic accident. To reflect this, safety is weighted at 70%, followed by cost, time and safety at 10% each.

The evaluation matrix for this option is shown in Table 7-24 and is discussed further in Section 7.7.2

Table 7-24 - SB-002 Evaluation matrix

Weighting		No action	Score	Rough finish	Score	Seal bridge deck	Score
Time	10%	0	0	0	0	-1	-0.1
Quality	10%	0	0	0	0	2	0.2
Cost	10%	0	0	0	0	-2	-0.2
Safety	70%	0	0	5	3.5	3	2.1
Sum		0		3.5		2.0	

7.7.2 Discussion

As identified by Table 7-24, the evaluation indicated that rough finishing the top of the curtain wall is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the "no action" option is not discussed as the reasons for its scoring are considered self-explanatory.

7.7.2.1 Time

Modifying the finishing requirements for the top surface of the sill beam will result in a negligible (unlikely to be more than 10 minutes) increase in construction time during the precast process with no increase in time for site works. As such, this option has scored 0 for time impacts.

Sealing of the bridge deck with a 10 mm chip seal is expected to add approximately 0.5 days to the site construction time. This time has been estimated under the assumption that the bridge is being constructed on-line and only the bridge deck will be sealed i.e. the seal on the bridge approaches is adequate and require no work. If the bridge is constructed off-line or the sealing activity happens in conjunction with another planned seal nearby then the time will likely be reduced. Nonetheless, this option has been evaluated on the premise that the top of the curtain wall and the bridge deck is the only area being sealed which has resulted in a time score of -1.

7.7.2.2 Quality

The running surface of the precast deck units was rough broom finished as were the cast in-situ deck closure strips with no resultant quality issues. Rough finishing is a common construction technique which is not expected to present any difficulties for a competent precast supplier and, as such, this option has received a quality score of 0 to represent no change to the quality environment. As an aside, this activity would also be recommended to be carried out if the bridge deck were to be sealed as the rough finish would promote adhesion of the seal to the concrete.

Sealing is a common activity that is routinely carried out by local government and other road authorities. Sealing of the bridge deck is not expected to result in any quality issues, rather it will improve the deck by providing an additional waterproof layer which may compensate for errors in finishing of the closure pours by sealing cracks/joints between the precast element and the in-situ pour. This change has resulted in a quality score of +2.

7.7.2.3 Cost

As discussed earlier in the evaluation, implementation of the rough finishing option is expected to result in a negligible increase in precast construction time. No additional materials are required to implement this option and no additional transport costs will be incurred. This extremely minor change has resulted in the cost impact being immeasurable and a resultant cost score of 0.

Sealing of the bridge deck has been assumed to be a standalone activity which is expected to take 0.5 days. The area to be sealed is 255 m^2 (deck area of 8.5 m x 30 m) which is small by industry standard and will result in a high square metre rate as the establishment and disestablishment cost will not be spread over a large sealing area. The cost estimate for this option is presented in Table 7-25 and has resulted in a 1.44% increase in project cost with a cost score of -2.

Table 7-25 - Cost estimate for deck seal option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Establish & disestablish	1	1	1	1	$ 10,000	$ 10,000
Primer	255	1	1	255	$ 10	$ 2,550
11 mm chip seal	255	1	1	255	$ 10	$ 2,550
Site facility recovery	1	0.5	1	0.5	$ 100	$ 50
					TOTAL	$ 15,150

7.7.2.4 Safety

Provision of a slip resistant surface is the primary safety concern that has led to the identification of this issue. Rough finishing the top of the curtain wall will provide a surface with a near-identical running surface to the bridge deck. When compared to smooth trowel finish, implementation of this option would result in a significant safety improvement with no reasonably foreseeable short or long term maintenance implication. The result of this change is a safety score of +5.

Sealing of the bridge deck including the curtain wall would also result in safety benefits by providing a slip resistant surface however, unlike the rough finishing option, a seal will require ongoing maintenance to address flushing, plucking of stone etc. It is common for road authorities to delay maintenance activities in order to achieve the maximum possible life out of each seal and minimise cost. If the seal is not adequately maintained then the safety benefits of this activity may be reduced – a reality which has resulted in a safety score of +3 for this option.

7.7.3 Recommendation

The recommended option to address this issue is to rough finish the top surface of the sill beam curtain wall concrete. This option will result in a significant increase in operational safety for this precast element with no foreseeable impacts on time, quality or cost

7.8 DMI-001 / DME-001: End diaphragm cover

7.8.1 Evaluation

The key driver of this issue is quality, specifically ensuring that sufficient concrete cover is achieved in the end diaphragm to adequately protect the reinforcement from corrosion. To reflect this, quality is weighted at 70%, followed by cost, time and safety at 10% each.

The evaluation matrix for this option is shown in Table 7-26 and is discussed further in Section 7.8.2

Table 7-26 - DMI001/DME001 Evaluation matrix

Weighting		No action	Score	Modify reo layout	Score	Corrosion resistant reo	Score	Increase diaphragm depth	Score
Time	10%	0	0	0	0	0	0	0	0
Quality	70%	0	0	3	2.1	4	2.8	5	3.5
Cost	10%	0	0	0	0	-1	-0.1	0	0
Safety	10%	0	0	0	0	0	0	0	0
Sum		0		2.1		2.7		3.5	

7.8.2 Discussion

As identified by Table 7-26, the evaluation indicated that increase the cover at the bottom surface of the end diaphragm is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the "no action" option is not discussed as the reasons for its scoring are considered self-explanatory.

7.8.2.1 Time

Modifying the reinforcement layout by raising the lower layer by 20 mm and/or specifying corrosion resistant reinforcement is not expected to result in any change to construction time as the quantity of steel to be tied has not changed. Both of these options have therefore received a time score of 0.

Increasing the depth of the end diaphragm to provide additional concrete and achieve design cover is estimated to increase the time taken to place and compact the concrete by approximately 15 minutes. When considered in the context of the project as well as the duration of the subject concrete pour (about five hours) the increase is considered negligible. The result of this assessment is a time score of 0 as the change is too small to be considered a notable negative impact.

7.8.2.2 Quality

All of the reinforcement for the trial bridge was placed and tied within the dimensional requirements of the B80 specification hence no quality issues are expected to result from the steel tying activity if the reinforcement layout is modified. This option proposes to raise the bottom layer of reinforcement by 20 mm to achieve design cover which is a positive quality result, however reducing the spacing between the layers has the unlikely potential to result in localised voids or segregation of concrete. When the balance of these changes is considered, the option has received a quality score of +3 as the overall change is positive.

Changing the reinforcement in the end diaphragm to be corrosion resistant by specifying galvanised steel or stainless steel would not modify the reo layout or steel tying activity in any way hence, as discussed above, no quality issues are expected. Use of the corrosion resistant material is expected to compensate for any lack of cover as the steel will not corrode and increase risk of bridge failure, however there is the potential that the use of different material in this single location may be overlooked during future construction. When this risk is considered, the result of this change is an assessed quality score of +4.

Increasing the depth of the end diaphragm by lowering the soffit by 20 mm to achieve design cover without modifying the reinforcement layout or materials is also not expected to result in any quality issues as no issue were observed with tying the identical steel layout during the trial bridge construction. Modification of the formwork would be a simple exercise requiring no additional technical skills and, given the cover would be 40 mm, the risk of shrinkage cracking will not measurably increase. After consideration of these points, the option has received a quality score of +5.

7.8.2.3 Cost

Modification of the reinforcement layout by raising the bottom layer of reinforcement is not expected to result in any increase is construction time nor any notable variation to material cost. It is therefore assessed that this option will realise no cost impact which has resulted in a cost score of 0.

Similarly, use of corrosion resistant materials will also not result in any increase in construction time. The required quantity of galvanised steel or stainless steel reo is estimated to cost $2,000 more than the same quantity of plain steel deformed bar, with this cost determined from recent industry rates. The result of this change is a cost score of -1.

Increasing the depth of the end diaphragm will result in an additional 0.94 m^3 of concrete being placed inclusive of 10% wastage allowance. It is estimated that this addition concrete will take approximately 15 minutes to place and compact with no additional finishing time required as the finishing area (in m^2) has not changed. The cost estimate for this option is presented in Table 7-27 and has resulted in a 0.03% increase in project cost, a change so minor that it has resulted in a cost score of 0.

Table 7-27 - Cost estimate for increase diaphragm depth option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	1	0.25	0.75	$ 59	$ 44
Concrete	0.94	1	1	0.94	$ 286	$ 267
Concrete pump work	1	1	0.25	0	$ 180	$ 45
					TOTAL	$ 357

7.8.2.4 Safety

There is no appreciable change to safety through implementation of any of the identified options, hence all options have received a safety score of 0.

7.8.3 Recommendation

The recommended option to address this issue is to increase the depth of the end diaphragm by lowering the soffit by 20 mm. This option will result in a significant increase in operational quality with a minor increase in cost and working time and not reduction in construction or operation safety.

7.9 DME-002: Scupper height

7.9.1 Evaluation

The key driver of this issue is safety, specifically ensuring that water drains effectively and efficiently from the bridge deck to avoid creating areas of standing water and reduce the risk of vehicular aquaplaning. To reflect this, safety is weighted at 70%, followed by cost, time and quality at 10% each.

The evaluation matrix for this option is shown in Table 7-28 and is discussed further in Section 7.9.2.

Table 7-28 - DME002 Evaluation matrix

Weighting		No action	Score	Lower scuppers	Score
Time	10%	0	0	0	0
Quality	10%	0	0	0	0
Cost	10%	0	0	0	0
Safety	70%	0	0	5	3.5
Sum		0		3.5	

7.9.2 Discussion

As identified by Table 7-28, the evaluation indicated that lowering the scupper inlet to allow water to fully drain from the deck is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the “no action” option is not discussed as the reasons for its scoring are considered self-explanatory.

7.9.2.1 Time

Lowering the scupper inlet level would result in no additional time in the precast process or in site works as the element to be installed would be identical, only it would be located 10 mm lower than present. Setting of the scuppers is a process that is entirely confined to the precast yard, hence this option would have no impact on site works and has received a time score of 0.

7.9.2.2 Quality

Lowering the scuppers would result in a change to the manufacturer of the deck units, a process that would be performed by a specialist and experienced precast contractor. There were no quality issues relating to the scuppers during construction of the trial bridge, hence, given the minor nature of the change that both options propose, no issues are expected in future constructions provided that a similar level of quality control is adopted. The result of this assessment is a quality score of 0.

7.9.2.3 Cost

Lowering the level of the scuppers is a no-cost change as it would not result in any change in construction time or material demand. As such, the cost score has been assessed to be 0.

7.9.2.4 Safety

Lowering the scupper inlet would result in an increase in operational safety by allowing water to drain freely from the deck thereby reducing the potential for aquaplaning. This benefit would be achieved by means of the positive hydraulic gradient between the deck level and the scupper inlet, however the level change would also make the scuppers less likely to be blocked and rendered ineffective by debris and plant growth as such debris including soil would also flow away freely. It is therefore concluded that this option would result in a significant safety benefit, a change that has resulted in a safety score of +5.

7.9.3 Recommendation

The recommended option to address this issue is to lower the level of the scuppers such that the inlet is below the finished deck level to allow efficient drainage from the deck. This option will result in a significant increase in operational safety with no foreseeable negative impacts to time, quality or cost.

7.10 DMI-002 / DME-003: Surface finish of precast deck flanges

7.10.1 Evaluation

The key driver of this issue is quality, specifically ensuring that the soffit formwork is properly sealed to prevent leakage of concrete and potential localised bony or understrength areas. This quality issues that may result from inadequate sealing were no present on the site, rather they are anticipated issues that could occur if the current rough finish of the deck flanges is maintained as-is in future construction. Given that the quality issue was not observed on site, quality has been designated a 50% weight, followed by time at 20% and quality and cost at 15% each.

The evaluation matrix for this option is shown in Table 7-29 and is discussed further in Section 0.

Table 7-29 - DME002/DMI003 Evaluation Matrix

Weighting		No action	Score	Cast in-situ soffit	Score	Smooth finish edge	Score	Recessed edge	Score
Time	20%	0	0	-5	-1	0	0	-1	-0.2
Quality	50%	0	0	-1	-0.5	4	2	4	2.0
Cost	15%	0	0	-4	-0.6	0	0	0	0
Safety	15%	0	0	-5	-0.75	0	0	0	0
Sum		0		-2.85		2.0		1.8	

7.10.2 Discussion

As identified by Table 7-29, the evaluation indicated that smooth finishing the last 50 mm of each precast flange is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the "no action" option is not discussed as the reasons for its scoring are considered self-explanatory.

7.10.2.1 Time

Removal of the precast deck module flanges to create a cast in-situ soffit will result in an estimated additional four weeks of site construction time. Individual soffits will need to be cast between each adjacent deck unit which, when expansion joints are considered, will result in nine discrete elements. The additional time will be required due to the need to build and install bespoke individual formwork panels and adequate bracing, pour and compact and additional 6.7 m^3 of concrete as well as the time taken for the concrete to cure prior to stripping of the formwork. This additional time has resulted in a time score of -5.

Modifying the precast flanges from the deck units to be smooth finished will increase precast construction time by an estimated nine hours as the smooth edge will need to be manually formed with a trowel or similar, however it will result in a minor reduction in site time due to more efficient sealing activities. The change in site time is expected to be in the order of 30 minutes hence it is deemed relatively inconsequential. As such, this option has received a time score of 0.

Modifying the precast flanges from the deck units to include a 9 mm recess would also result in an estimate nine additional man hours of work in the precast process, however, unlike the previous option, additional site construction time would also be required. The purpose of the recess is to provide a smooth and confined area in which to install and seal a strip of 9 mm compressed cement sheet. It is estimated that cutting the large sheets into appropriately sized strips prior to install would be a full day's work for a single person, with the time taken to seal the strips being generally equivalent to the time taken to seal the current bitumen impregnated tape arrangement. The result of this change is a time score of 0.

7.10.2.2 Quality

The degree of quality control and quality assurance achievable in a precast yard is typically greater than that which is achievable by site work by virtue of standardisation and repetition as well as the removal of site hazards such as working at heights which allows more efficient construction practices. Removal of the precast deck flanges in favour of construction of a full length cast in-situ soffit would likely result in a decrease in construction quality as each formwork panel would need to be individually installed and checked for positional tolerance. The formwork is envisaged to consist of F12 formply panels with 90 x 45 mm structural pine transverse and longitudinal bracing. A length of pine would be screw bolted along the web of each deck tee to provide vertical edge support, with acrow props staggered to pick up the longitudinal bracing ad provide ground support. The fixing location of the edge support elements into the deck flange would need to be carefully chosen to ensure avoidance of reinforcing steel and prestress wire, especially the latter as damage to strand would result in the deck unit beings structurally compromised. Consideration of these risks has resulted in a quality score of -5.

Modification of the precast flanges to have either a flat or recessed smooth finished edge would be a minor change to the precast process, however the flanges are mass concrete elements designed as formwork with structural contribution to the completed bridge. Both of these options would allow a more efficient seal between adjacent flanges by providing a consistent surface, thereby allowing the form material to sit plat against the concrete surface. As such, both of these options have received a quality score of +4.

7.10.2.3 Cost

As discussed previously, smooth finishing the edge of the flange would result in an addition nine hours of precast time (30 minutes per flange) which, at $59 per hour, would create a cost increase of $531. No additional materials would be required for this option hence the assessed cost increase is $531 or 0.05% of total project cost which has received a cost score of 0.

Similar to the smooth finish option, providing a recessed edge would also require an additional nine hours of precast construction time, however additional site construction time would also be required. The recess would be intended to provide a channel in which to install and seal a strip of 9 mm compressed cement sheet. This sheet is supplied in large sheets, commonly 1.8 x 1.2 mm which would need to be cut to size to fit. It is estimated that cutting the required number of strips would take a single worker one day, with the time to install and seal the sheets the same as what is currently required to install and seal the bitumen impregnated tape. Purchase of the sheet would incur an additional material cost, however this is compensated by not needing to purchase the bitumen tape. The cost estimate for this option is presented in Table 7-30 and has resulted in a 0.08% increase in project cost with a cost score of 0.

Table 7-30 - Cost estimate for recessed flange option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
15 mm comp cement sheet 1.8 x 1.2 m	3	1	1	3	$ 120	$ 360
Labour site	1	1	9.5	9.5	$ 59	$ 561
Labour precast	1	1	9	9	$ 59	$ 531
Bitumen tape reduction	1	1	1	1	$ -600	$ -600
					TOTAL	$ 852

Removal of the flanges from the precast deck unit would result in an estimated four weeks of site construction time as discussed earlier. Additional formwork and falsework material would be required, as well as two mobile scaffolds (hire) to provide a safe area for working at heights and an EWP to assist in lifting the formwork panels. An additional 6.7 m^3 of concrete would need to be supplied to account for the volume added by removal of the precast flanges and, although the supply quantity is balanced by an identical precast material cost reduction, placement and compaction of this concrete would result in an additional two hours of site work. The cost estimate for this option is presented in Table 7-31 and has resulted in a 4.24% increase in project cost and a cost score of -5.

Table 7-31 - Cost estimate for cast in-situ option

Item	Quantity	Days	Hours/day	Quantity	Rate	Total
Labourer	3	20	9.5	570	$ 59	$ 33,630
Mobile scaffold hire	2	20	1	40	$ 20	$ 800
EWP hire (lifting)	1	20	1	20	$ 250	$ 5,000
Engineer design/check	1	1	1	1	$ 2,500	$ 2,500
Formply F12 1200 x 1800	54	1	1	54	$ 50	$ 2,700
Trans brace 90 x 45 struct pine	99	1	1	99	$ 5	$ 495
Long brace 90 x 45 struct pine	180	1	1	180	$ 5	$ 900
Acrow prop 3.2-3.9m hire	108	5	1	540	$ 10	$ 5,400
Fixings allowance	1	1	1	1	$ 500	$ 500
Site concrete additional	6.7	1	1	6.7	$ 286	$ 1,911
Concrete pump work additional	1.0	1	2	2	$ 180	$ 360
Precast labour reduction	12	1	1	12	$ -200	$ -2,400
Precast concrete reduction	6.7	1	1	6.7	$ -286	$ -1,911
Precast mould reduction allowance	1	1	1	1	$ -5,000	$ -5,000
Bitumen tape reduction	1	1	1	1	$ -600	$ -600
Bitumen tape labour reduction	3	1	9.5	28.5	$ -59	$ - 1,682
Site facility recovery	1	19	1	19	$ 100	$ 1,900
					TOTAL	$ 44,504

7.10.2.4 Safety

Modification of the flanges to provide a smooth edge, whether flat or recessed, would not result in any change to the safety environment for the site work component of the bridge construction as the process of sealing would be more or less identical to the current practice. Both of these option have therefore resulted in a safety score of 0

Construction of a cast in-situ soffit would be a significant exercise involving a large amount of working at heights. This risk would be mitigated by the use of mobile scaffold, and EWP and fall arrest systems. Additional risk of abrasion injury, strain and fatigue would also result from the increases quantity of formwork carpentry and installation of formwork panels required for this option. It is noted that these risks are already present in other element of the bridge construction however, when compared to the risks associated with the current formwork sealing activity, the changes represent a significant reduction in site construction safety. The outcome of this assessment is a safety score of -5.

7.10.3 Recommendation

The recommended option to address this issue is smooth finish the edge of precast flanges to provide a consistent surface from which to seal the bitumen impregnated tape. This option will an increase to construction quality by creating a better seal between adjacent flanges with no appreciable change to site time, cost or safety.

Alternatively, if pre-cut strips of compressed cement sheeting can be sourced, the additional time associated with the recessed flange option can be avoided which would also make this option comparatively viable.

7.11 DS-002: Deck cross-fall finishing

7.11.1 Evaluation

The key drivers of this issue is are quality and safety, with both facets relating to ensuring that water drains freely from the bridge deck. Quality and safety have therefore been designated weightings of 40% each, followed by time and cost at 10% each.

The evaluation matrix for this option is shown in Table 7-32 and is discussed further in Section 0.

Table 7-32 - DS001 Evaluation matrix

Weighting		No action	Score	Flat central closure pour	Score	One way cross-fall	Score	Rounded profile	Score
Time	10%	0	0	2	0.2	2	0.2	1	0.1
Quality	40%	0	0	-5	-2	4	1.6	3	1.2
Cost	10%	0	0	0	0	0	0	0	0
Safety	40%	0	0	-5	-2	-5	-2	0	0
Sum		0		-3.8		-0.2		1.3	

7.11.2 Discussion

As identified by Table 7-32, the evaluation indicated that specifying a rounded finish on the central closure pour is the most appropriate option for addressing this issue. The evaluation criteria and scores are discussed below, however the "no action" option is not discussed as the reasons for its scoring are considered self-explanatory.

7.11.2.1 Time

Finishing of the central closure pour took approximately twice as long as finishing of the outer closure pours due the difficult in achieving the required apex. Removing the apex to allow a flat central closure pour would result in an estimated time reduction of 0.5 man hours per pour area. The same time difference would be realised with a one way cross fall option, as the central pour would be finished flat and cross-fall achieved by a level difference between adjacent precast modules. This difference may appear negligible in the context of the bridge construction, however finishing of the deck pour is a time sensitive activity as concrete has a limited working time. This reduction is therefore reflected in a time score of +2.

Retaining the cross-fall in the central closure pour but removing the defined apex in favour of a generally rounded finish would also result in a reduction in finishing time as the focus would be on achieving an overall adequate drainage profile rather than a straight line through the centre of the pour. This change would result in an estimated reduction in construction time by 0.25 man hours per pour area and a time score of +1.

7.11.2.2 Quality

Changing the grade of the central closure pour to be flat without modifying the reinforcement detail would lead to a 12 mm loss of cover (0.4 m steel projection from precast module at 3% grade). One of the key components of the CBS system is standardisation hence modification of the internal deck units to create a second class of deck with projected reinforcement designed specifically for the central closure pour would not be a feasible action. Removing the apex from the pour area would, however, remove the issue associated with finishing of same thereby allowing greater consistency and quality control. Nonetheless, this positive impact does not outweigh the negative impact of compromising cover to steel embedment's hence this option has received a quality score of -5.

Similar to the previous option, a one way deck cross-fall would allow a central stitch area and achieve greater consistency of finish. The difference between the two options is that overall grade would be achieved by level differences between adjacent deck modules in the same direction across the bridge. As the two deck modules involved in the central stitch will be on the same grade there will be no issues with loss of cover to concrete, hence this option has received a quality score of +4.

The third option differs from the two already presented in that the cross fall is retained albeit without a defined apex, rather the central closure stitch would be finished with a generally rounded profile. This change would still allow water to drain away from the in-situ pour area with a high level of finish consistency and no anticipated quality issues. The quality change score for this option has been assessed as +3.

7.11.2.3 Cost

As discussed earlier, all of the proposed options result in a minor increase to working time with no change to material quantities. The cost increase is therefore considered to be negligible and all options have received a time score of 0.

7.11.2.4 Safety

Changing the grade of the central closure pour to be flat would not result in any change in construction safety, however removing the cross-fall will encourage ponding of water in the pour area. This ponding will decrease operational safety by increasing the risk of aquaplaning and resultant vehicular accidents. As such this option has an assessed safety score of -5.

Modifying the deck levels to have a constant cross-fall across the full bridge width and allow a flat (relative to adjacent deck modules) central closure pour would also not result in any change to construction safety without causing ponding of water. Despite this comparatively positive outcome, a one way cross-fall would typically only be suitable for a bridge constructed on a horizontal curve whereas the CBS system does not cater for skew bridges. A bridge with constant cross-fall on a straight alignment would create difficulty for vehicles to stay within their lane which would increase the risk of vehicles leaving the road and having head on collisions. This increase risk has resulted in a safety score of -5.

A rounded central closure pour area would not change construction safety, nor would it have any impact on operational safety as the water would still be able to drain freely from the bridge deck. This has resulted in a safety score of 0.

7.11.3 Recommendation

The recommended option to address this issue is replace the defined apex in the central closure pour with a generally rounded profile. This option will result in an increase in construction quality and a minor reduction in cost with no appreciable adverse impacts on time or safety.

7.12 Conclusion

This chapter has evaluated the presented options for eleven of the identified issues from Chapter Six on time, quality, cost and safety criteria. The underlying issue for each of the issues was examined and weighting assigned commensurately. The result of the chapter is a recommended option for each of analysed issues as summarised below in Table 7-33.

Table 7-33 - Summary table of issues and recommended options

Issue code	Description	Recommended option
AFT001 PFT-001	Spacing of reinforcement in footings presents risk of falling and injury to worker	Modify reinforcement from N20 bars at 200 mm c/c to N16 bars at 130 mm c/c
AFT-002	Width of retaining wall results in visual inconsistency at front of Abutment B and compaction difficulties below sill beam at rear of Abutment B	Increase with of retaining wall to 1080 mm to has visually consistent finish and remove the need to compact material beneath the sill beam overhang
AFT-003	Wing wall is not fully supported on retaining wall which leaves the wing wall unstable and prone to damage during pavement construction	Extend retaining wall to support the full length of the wing wall
AFT-004	No drainage provision at Abutment A to drain water and relieve hydrostatic pressure	Specify drainage system on drawings
AFT-005 PFT-002	Specified hole for fixing dowels to rock is too large for easy procurement of suitable drilling plant	Change from N36 dowels in 100 mm holes to 24 dowels in 50 mm holes
SB-001	The top of the sill beam curtain wall may be subject to concrete damage and breakout due to high vehicle impact loads	Install 90 x 90 x 10 steel angle to protect the corner of the curtain wall exposed to traffic coming off the approaches

SB-002	The top surface of the sill beam curtain wall forms part of the running surface but is provided smooth finished which has poor tyre grip properties	Rough finish the top surface of the curtain wall
DMI-001 DME-001	Cover to steel at the underside of the end diaphragm is less than design	Increase cross beam thickness by 20 mm to achieve cover
DME-002	Scupper inlet is located higher than the finished deck level which will prevent full draining of water from the bridge deck	Lower scupper inlet to facilitate proper drainage
DMI-002 DME-003	Flanges are rough finished which created issues with sealing between adjacent units	Smooth finish final 50 mm of the precast deck flanges to allow consistent sealing surface
DS-001	Access to install bottom layer of longitudinal reinforcement is obstructed by top layer of precast projected transverse reinforcement	Not analysed – opportunity for further work by others
DS-002	Finishing of the central closure pour with two way cross-fall is difficult to maintain a straight defined apex	Specify generally rounded finish to remove the need for a defined apex
DS-003	Reinforcement layout in end diaphragm is congested which may result in inadequate concrete penetration	Not analysed – opportunity for further work by others
TB-001	The top rail of traffic barrier is 800 mm off deck height whereas the minimum height of a temporary edge protection system is 900 mm. It would assist in construction if the traffic barrier could also function as edge protection	Not analysed – opportunity for further work by others
TB-002:	The bolts projected from the deck kerb into the traffic barrier are	Not analysed – opportunity for further work by others

	different sizes, likewise the receiving holes in the traffic barrier base plate. These variations increase the chance of construction errors.	
B-001 SB-004	Half of the bearings on the sill beam are located within the stitch areas to the Abutment which results in extra site work that would be required if there was no clash	Not analysed – opportunity for further work by others
B-002	Bearing pin specified as independent to bearing plate caused issue achieving design penetration into head stock or sill beam void and elastomeric bearing pad	Not analysed – opportunity for further work by others
B-003	Access to install bearing grout pad formwork between the bearing and the Abutment curtain wall is limited	Not analysed – opportunity for further work by others
J-001	Difficult to ensure straight alignment of transverse expansion joints during pouring adjacent deck cross beams.	Not analysed – opportunity for further work by others
HB-001	Some stainless steel bolts fixing the hold down bracket sheared in the stainless steel ferrules.	Not analysed – opportunity for further work by others

8 Conclusion

8.1 Project conclusion

This project started by researching the existing modular bridge systems available to market that are typically considered during the planning of bridge replacement projects on local roads in NSW. Constructability was then examined and the key areas of time, quality and cost were identified as discussed, followed by safety in design. The literature generally agreed that optimisation of these elements typically results in an overall improvement in project outcomes. It was identified that a gap exists in the literature relating to the construction of the modular Country Bridge Solutions system, hence the project direction was born.

This project has documented the site works associated with the pilot construction of the Country Bridge Solutions system as implemented during replacement of the existing timber bridge over Bookookoorara Creek on Mount Lindesay Rd in Tenterfield Shire. The construction records were analysed and twenty areas were identified in which the design or methodology could be improved to facilitate better constructability or safety outcomes. These issues were generally spread amongst all facets of the construction, however it is noted that the abutment and pier footings were over-represented and that these are not standard elements and do not form part of the Country Bridge Solutions system itself.

The identified issues were explained and concept options were developed to assist in their resolution. A matrix was then developed to analyse the options on weighted time, quality, cost and safety criterion which were identified as key aspects of constructability and safety in design in the available literature. The concepts for eleven issues were examined and analysed using this matrix with one option being identified as the optimal solution for each issue (it is noted that this is an optional task in the project specification – refer to Appendix A: Project Specification).

Overall, this project has contributed to the engineering body of knowledge by documenting the construction of the pilot bridge for the benefit of future construction teams. The identified areas and concepts are presented to assist in the development of the Country Bridge Solutions system which is ultimately aimed at providing an efficient and effective bridge replacement option on low volume roads.

8.2 Future work

This project has identified twenty areas of the bridge design or construction methodology that may be refined to support greater constructability or improved safety outcomes and design options intended to address same. The options presented for eleven of the issues were evaluated to identify the optimum concept, however additional work is required to evaluate the additional issues and options. Once completed, the recommended option can be evaluated for inclusion in the Country Bridge Solutions system and may progress to detailed design.

A limitation of this project is that it examined a single bridge construction deliver by one construction team hence the results have the potential to be influenced by local practices. It is therefore recommended that the process is repeated for the next construction of the Country Bridge Solutions system both to see if the implementation of any of the recommended options had a positive impact on site works (if any were implemented) and any additional issues are raised. If further issues are raised, a comparative analysis of methodologies between the two constructions would also be valuable to determine why the issue was not present of recognised in this project.

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Appendix A: Project Specification

ENG4111/4112 Research Project
Project Specification

For:	Alexander Rosnell
Title:	Investigating design and construction issues for precast concrete bridge over Bookookoorara Creek
Major:	Civil Engineering
Supervisors:	Dr Weena Lokuge (USQ) Mr Peter Young (RMS)
Sponsorship:	NSW Roads and Maritime Services
Enrolment:	ENG4111 – EXT S1, 2016 ENG4112 – EXT S2, 2016
Project Aim:	To document the construction process of the pilot bridge under RMS Country Bridge Solutions and identify areas of design refinement from a constructability perspective.

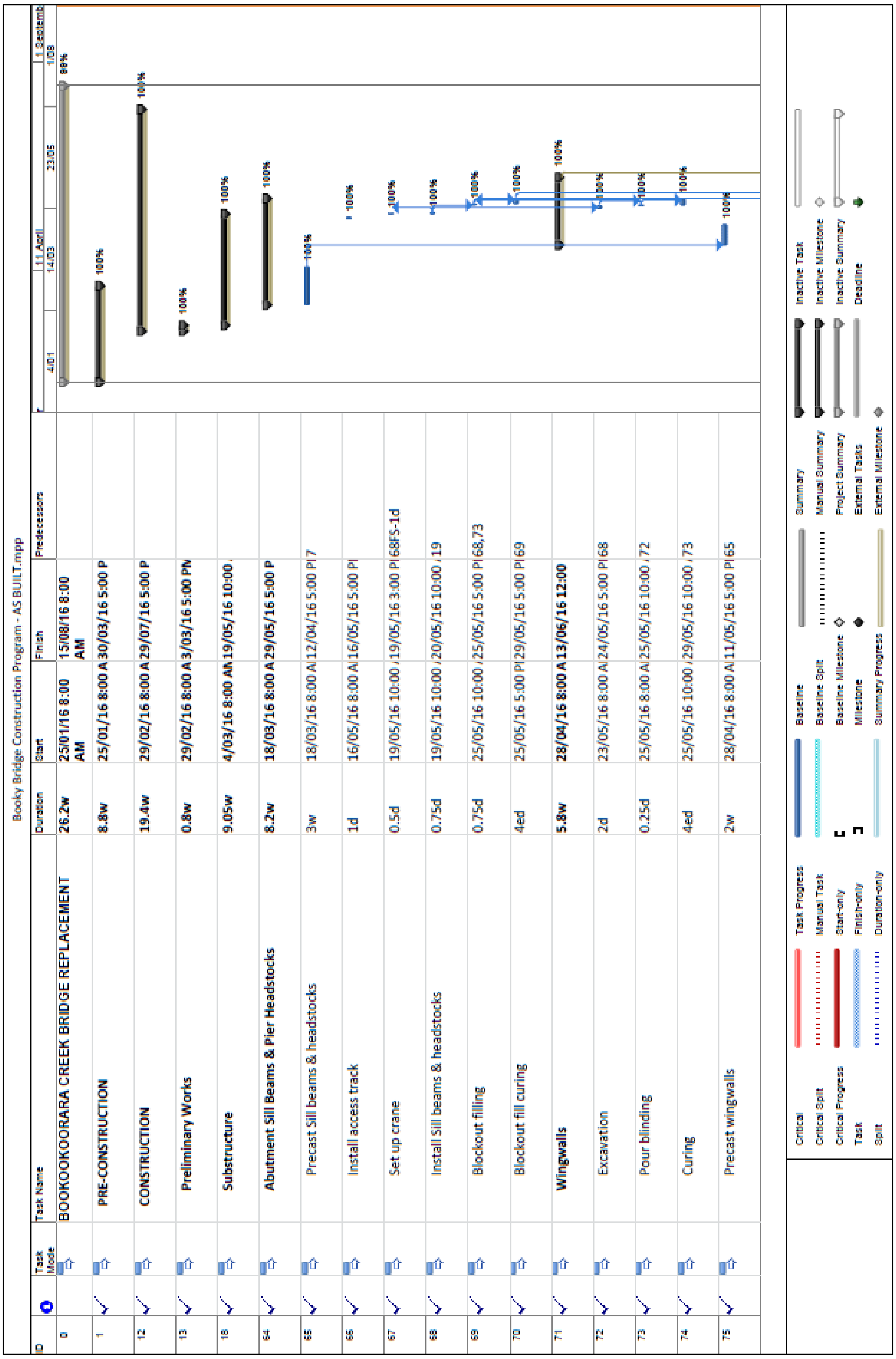
Programme: Issue B, 7TH SEPTEMBER 2016

1. Investigate and provide a brief evaluation of existing precast concrete bridge systems on the general market.
2. Investigate and discuss key aspects of constructability and safety in design
3. Procure resources and construct the pilot bridge over Bookookoorara Creek
    a. Keep a construction diary of key activities and progress
    b. Regularly attend the construction site to observe progress and identify troublesome, difficult to construct or unsafe elements of the design (if any exist)
4. Record variations to the design made during the construction process and explain the rationale behind the changes
5. Identify further areas of design refinement
6. Provide design concepts which may resolve the identified issues
7. Develop a matrix for assessment of proposed design changes

If time and resources permit:

8. Assess the proposals and provide detailed design on at least one of the issues
9. Make recommendations for further investigation of constructability on future bridges under the Country Bridge Solutions program which are to be constructed in different environments (e.g. driven piles with cap rather than cast in-situ footings)

Appendix B: Construction program

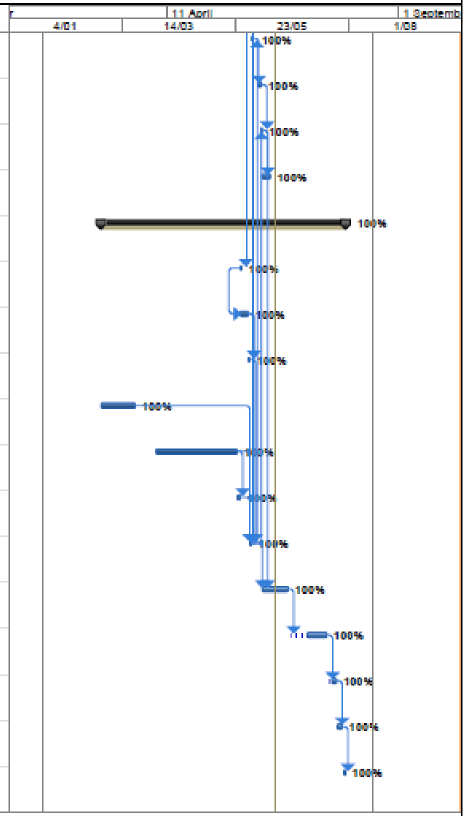


Booky Bridge Construction Program - AS BUILT.mpp

ID	Task Mode	Task Name	Duration	Start	Finish	Predecessors
0		BOOKOOKOORARA CREEK BRIDGE REPLACEMENT	26.2w	25/01/16 8:00 AM	15/08/16 8:00 AM	
1		PRE-CONSTRUCTION	8.8w	25/01/16 8:00 A	30/03/16 5:00 P	
12		CONSTRUCTION	19.4w	29/02/16 8:00 A	29/07/16 5:00 P	
13		Preliminary Works	0.8w	29/02/16 8:00 A	3/03/16 5:00 PN	
18		Substructure	9.05w	4/03/16 8:00 AN	19/05/16 10:00 .	
64		Abutment Sill Beams & Pier Headstocks	8.2w	18/03/16 8:00 A	29/05/16 5:00 P	
65		Precast Sill beams & headstocks	3w	18/03/16 8:00 A	12/04/16 5:00 PI	7
66		Install access track	1d	16/05/16 8:00 A	16/05/16 5:00 PI	
67		Set up crane	0.5d	19/05/16 10:00 /	19/05/16 3:00 PI	68FS-1d
68		Install Sill beams & headstocks	0.75d	19/05/16 10:00 /	20/05/16 10:00 /	19
69		Blockout filling	0.75d	25/05/16 10:00 /	25/05/16 5:00 PI	68,73
70		Blockout fill curing	4ed	25/05/16 5:00 PI	29/05/16 5:00 PI	69
71		Wingwalls	5.8w	28/04/16 8:00 A	13/06/16 12:00	
72		Excavation	2d	23/05/16 8:00 A	24/05/16 5:00 PI	68
73		Pour blinding	0.25d	25/05/16 8:00 A	25/05/16 10:00 /	72
74		Curing	4ed	25/05/16 10:00 /	29/05/16 10:00 /	73
75		Precast wingwalls	2w	28/04/16 8:00 A	11/05/16 5:00 PI	65

Booky Bridge Construction Program - AS BUILT.mpp

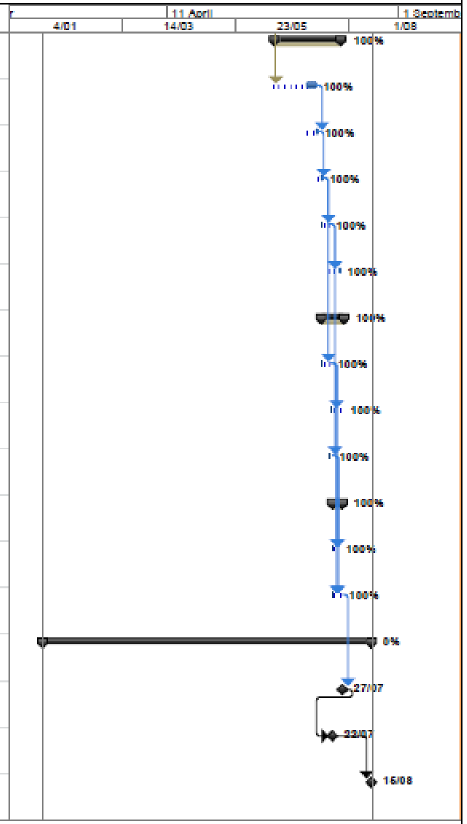
ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
76	✓		Install wingwalls	1d	2/06/16 8:00 AM	2/06/16 5:00 PM	87
77	✓		Reinforcement	2d	6/06/16 8:00 AM	7/06/16 5:00 PM	76
78	✓		Blockout filling	0.5d	8/06/16 8:00 AM	8/06/16 12:00 PM	87,77
79	✓		Blockout fill curing	5ed	8/06/16 12:00 PM	13/06/16 12:00 PM	78
80	✓		**Deck Modules**	**19.2w**	**1/03/16 8:00 AM**	**29/07/16 5:00 P**	
81	✓		Survey set out	1d	26/05/16 8:00 A	26/05/16 5:00 PM	69
82	✓		Bearing set out and packers	3d	26/05/16 8:00 A	30/05/16 5:00 PM	81SS
83	✓		Hold down brackets	1d	31/05/16 8:00 A	31/05/16 5:00 PM	82
84	✓		Traffic barrier railing construction	3w	1/03/16 8:00 AM	21/03/16 5:00 PM	
85	✓		Precast Deck modules	5.8w	4/04/16 8:00 AM	23/05/16 5:00 PM	8
86	✓		Install traffic barrier railing off site	2d	24/05/16 8:00 A	25/05/16 5:00 PM	85
87	✓		Install Deck modules	1d	1/06/16 8:00 AM	1/06/16 5:00 PM	70,82,84,86,85,83
88	✓		Bearing adjustment	10d	8/06/16 1:00 PM	24/06/16 12:00 PM	87,78
89	✓		Grouting	8d	24/06/16 1:00 PM	18/07/16 12:00 PM	88
90	✓		Bearing grout curing	6ed	18/07/16 12:00 PM	24/07/16 12:00 PM	89
91	✓		Bearing epoxy injection rework	3d	25/07/16 8:00 A	27/07/16 5:00 PM	90
92	✓		Bearing load transfer	2d	28/07/16 8:00 A	29/07/16 5:00 PM	91



Critical | Task Progress | Baseline | Summary | Inactive Task
Critical Split | Manual Task | Baseline Split | Manual Summary | Inactive Milestone
Critical Progress | Start-only | Baseline Milestone | Project Summary | Inactive Summary
Task | Finish-only | Milestone | External Tasks | Deadline
Split | Duration-only | Summary Progress | External Milestone

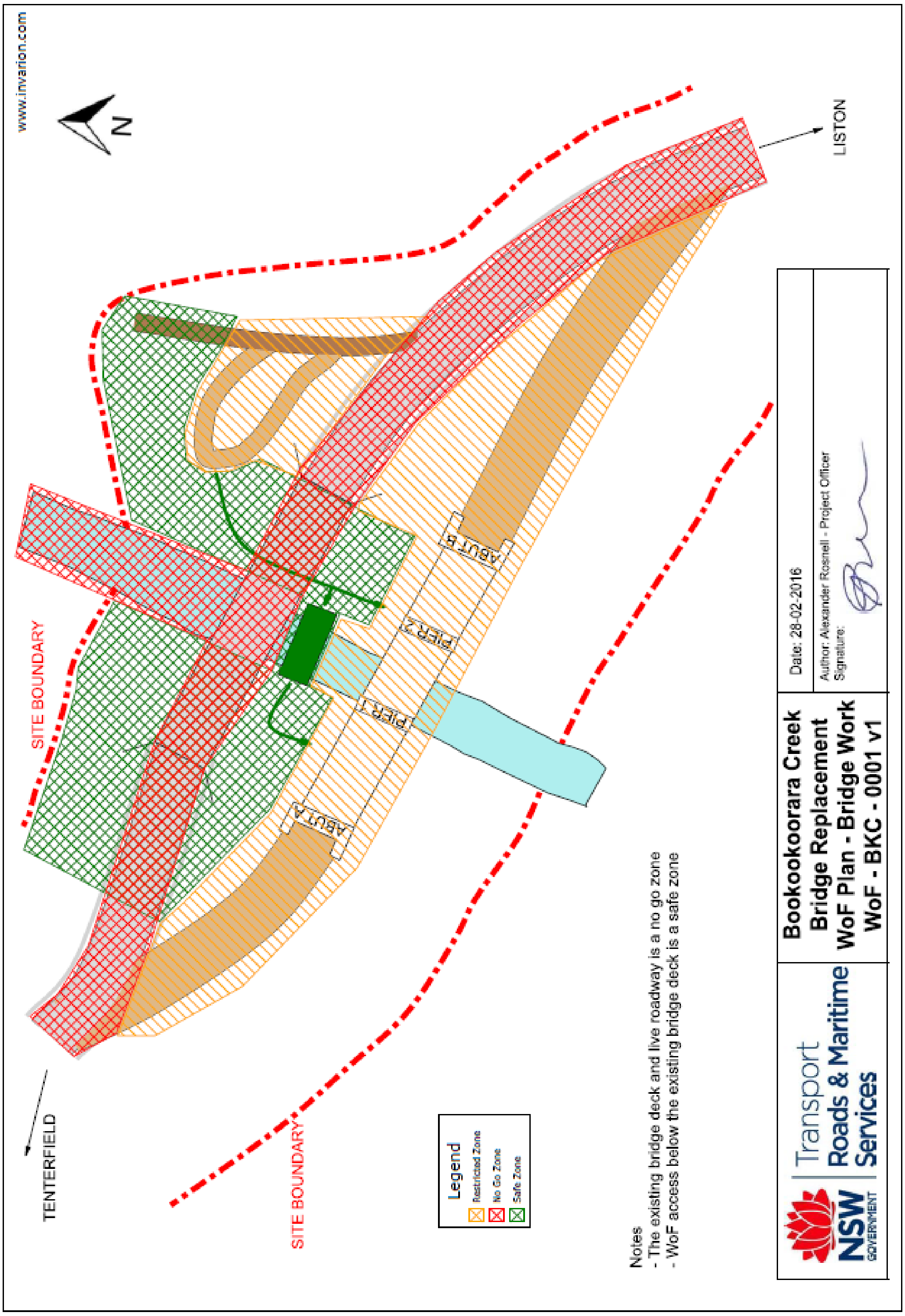
Booky Bridge Construction Program - AS BUILT.mpp

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
93	✓		**Abutment Stitch & Deck Closure Slabs**	**5.8w**	**16/06/16 8:00 A**	**26/07/16 5:00 P**	
94	✓		Reinforcement	18d	16/06/16 8:00 A	11/07/16 5:00 P	71
95	✓		Soffit & edge formwork	5d	6/07/16 8:00 AM	12/07/16 5:00 P	94
96	✓		Concrete	4d	12/07/16 8:00 A	15/07/16 5:00 P	95
97	✓		Cure	5ed	14/07/16 5:00 P	19/07/16 5:00 P	96
98	✓		Strip & bag	5d	20/07/16 8:00 A	26/07/16 5:00 P	97
99	✓		**Barriers & Deck Joints**	**2w**	**15/07/16 8:00 A**	**28/07/16 5:00 P**	
100	✓		Traffic barrier railing adjustment grouting	4d	15/07/16 8:00 A	20/07/16 5:00 P	96
101	✓		Traffic barrier railing grout curing	2d	21/07/16 8:00 A	28/07/16 5:00 P	100
102	✓		Transverse deck joints	2d	19/07/16 5:00 P	21/07/16 5:00 P	97
103	✓		**Completion tasks**	**0.8w**	**22/07/16 8:00 A**	**27/07/16 5:00 P**	
104	✓		Environmental control removal	1d	22/07/16 8:00 A	25/07/16 5:00 P	100,102
105	✓		Site disestablishment	2d	22/07/16 8:00 A	27/07/16 5:00 P	100,102
106			**POST-CONSTRUCTION**	**26.2w**	**25/01/16 8:00 A**	**15/08/16 8:00 A**	
107			WAE drawings	0d	25/01/16 8:00 A	27/07/16 5:00 P	105
108	✓		Close-out meeting	0d	22/07/16 8:00 A	22/07/16 8:00 A	107
109	✓		Project review	0d	15/08/16 8:00 A	15/08/16 8:00 A	108

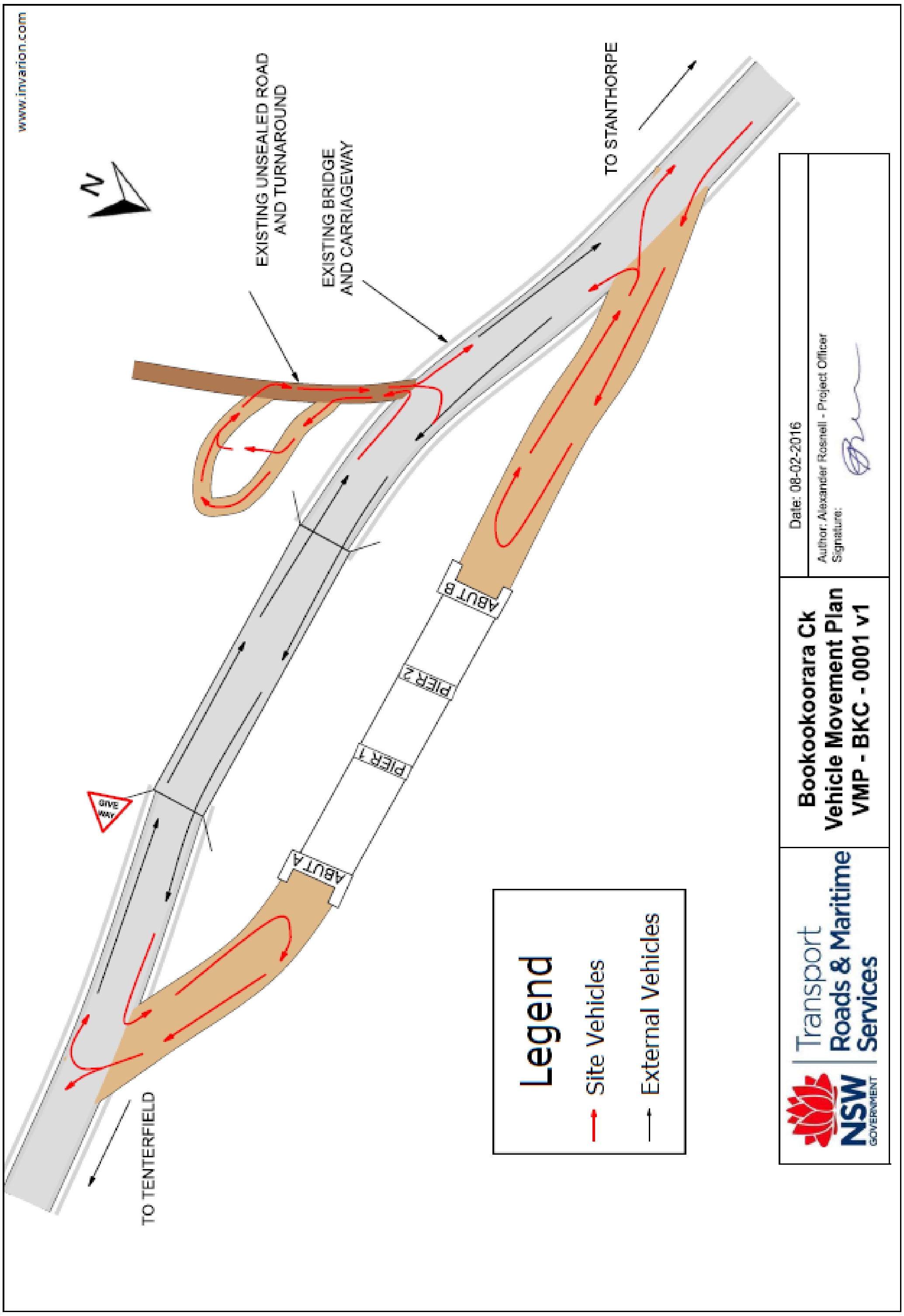


Critical
Critical Split
Critical Progress
Task
Split
Task Progress
Manual Task
Start-only
Finish-only
Duration-only
Baseline
Baseline Split
Baseline Milestone
Milestone
Summary Progress
Summary
Manual Summary
Project Summary
External Tasks
External Milestone
Inactive Task
Inactive Milestone
Inactive Summary
Deadline

Appendix C: Workers on Foot plan



Appendix D: Vehicle Movement Plan



Appendix E: Erosion and Sediment Control Plan