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FINAL REPORT

Project Title: Long-term performance of FRP replacement components and structures (Year 3 - 2017/18)

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SUMMARY

A limited number of existing complete fibre reinforced polymer (FRP) structures and structures with replacement FRP components have been identified on the TMR network. One of the objectives of installing these components is to gain practical understanding of the performance of the FRP material in bridge applications. Consistent with this objective, this project involved a performance review of FRP components and structures in the TMR network. In addition, TMR is interested in assessing the condition of its stockpiled FRP girders (manufactured by Wagners Composite Fibre Technologies (WCFT) and Loc Composites and stored at several Districts) to determine whether it is appropriate to utilise them. This project included a visual inspection of TMR FRP bridges and replacement components and an assessment of the performance of the stockpiled FRP girders through visual inspection and serviceability load testing.

This report presents the outcomes of the inspection of TMR's FRP components and bridges based on a defect rating system proposed for assessing the severity and extent of defects. This report also recommends maintenance and monitoring actions for these components. Some future directions for further research and investigation into the application of FRP products on the TMR bridge network are also proposed.

The stockpiled girders have been stored in relatively benign conditions with some ultra-violet (UV) exposure and moisture. The WCFT girders showed a low level of deterioration/degradation, with the most common damage due to physical handling and transportation. Serviceability load testing on selected Wagners girders indicated that there were no significant changes in the stiffness.

All replacement FRP girders installed on existing timber bridges were also inspected. Given that these FRP girders are covered by the bridge deck, they were found to be in generally fair condition, with a low level of material degradation identified. The most common types of defects were minor loss of material, fibre exposure and localised damage due to physical impacts, moisture ingress and direct sunlight. A good level of interaction of the FRP girders with the surrounding components was observed.

Some structural defects and material degradation were observed on the complete FRP structures, e.g. cracking on the external faces of the FRP components on Sandy Creek Bridge due to UV light coupled with moisture impact, and differential deflections of adjacent FRP units on Taromeo Creek Bridge.

The recommendations arising from this investigation are as follows.

1. The stockpiled WCFT girders showed signs of deterioration. An RFP should be issued to WCFT seeking upgrade of all stockpiled girders to stage 3 performance, and an improved surface coating to better protect structural integrity. Assuming that a cost-effective proposal is forthcoming, these girders should be upgraded and issued for use on the TMR network.
2. The nature of the brown staining on the LOC420 girders should be investigated. If it is found that the staining has phenolic resin origins, then the LOC400 and LOC420 girders should be subject to minor repair by TMR staff based on advice from the University of Southern

Queensland (USQ), and the stockpiled girders issued for use in the TMR network.

3. Further to (1) and (2), the girders should be subject to detailed inspection and defect mapping following installation, and as part of the regular TMR inspection cycle.
4. Defects in girders that had been previously installed in timber bridges were mapped as part of this project. Based on the current TMR strategy with respect to fibre composites, it is proposed that these defects not be repaired, but that they be maintained consistent with (3) above so that fibre composite durability issues can be better understood in the future.
5. Heifer Creek #7 Bridge had the most significant fibre composite defects. This bridge should be inspected annually with defects mapped in detail, and the inspection report should be reviewed by TMR E&T. Particular care should be taken to monitor changes in delaminated flange and de-bonded angles. Any changes in these defects should trigger a detailed structural review.
6. Further to (5), the reason for the sag in the girders should be investigated and the girders carefully monitored.
7. A Level 3 investigation into the differential movement apparent in Taromeo Creek Bridge should be undertaken.
8. Further to (2), the nature of the brown staining on the Oakey Creek Bridge should be investigated if phenolic leaching in the stockpiled girders cannot be demonstrated.
9. There may be merit in reviewing TMR technical guidelines and specifications (including the ϕ factor) if the TMR strategy in relation to fibre composite materials changes to one of active promotion. Such a review should be accompanied by systematic industry engagement.

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1 INTRODUCTION

1.1 Background

The use of innovative fibre reinforced polymer (FRP) materials in bridge construction and refurbishment is increasing rapidly. The development of knowledge in the application of these materials is also increasing through not only research and development (universities) but also the increasing number of commercial solutions being made available by private companies.

Queensland Department of Transport and Main Roads (TMR) has several FRP systems currently in service including ‘complete’ FRP bridge superstructures and FRP structural components used as replacements for hardwood timber girders in timber bridges. Other jurisdictions have also replaced components such as corbels.

Guidelines and specifications for the use of FRP products are already established. However, while the application of these materials from a design perspective is understood, there is a lack of knowledge regarding their long-term performance. In addition, condition rating of these structures is not well covered in current practice and there is limited information available on TMR’s past research projects related to the monitoring and testing of FRP structures and components.

The purpose of this project is to develop a better understanding of the long-term performance of in-service FRP replacement components and complete FRP bridge superstructures. Specifically, the project aims to:

- establish a knowledge base for the performance of existing components/bridges of this type under TMR management
- identify issues related to the interaction between replacement FRP components and existing timber components on timber bridges
- progress towards a reliable approach to assessing the durability and long-term performance of this type of component/bridge.

A literature review was carried out as part of Year 1 of the project. The review of the international literature showed that most monitoring programs worldwide are based on visual inspections coupled with conventional load testing using traditional instrumentation such as strain gauges, deflection sensors and accelerometers. The durability and performance of the FRP structures are assessed based on comparisons of field measurements and observations at different times during the life of the structure. While the general conclusion was that the FRP structures have been performing as expected, and limited levels of material deterioration have been observed, due to the fact that the FRP structures have only been in use for infrastructure applications for a short period of time (first composite bridge constructed in 1997), further monitoring is required to validate long-term performance.

The investigation of TMR’s current bridge stock undertaken during Year 2 of the project led to the identification of a limited number of existing FRP structures and structures with replacement FRP components on its network. While load testing of in-service bridges is a good approach to assess the long-term performance of the structures, it is out of scope of this project because:

- there is no data available at the initial state and during the life of the structure which would allow the change in the behaviour of the structure to be determined
- If load testing can be conducted to establish the current state of the structure, then it should be repeated periodically to measure the changes over time; however, this exercise is costly and could not deliver the objectives of this project in a short time frame.

In addition, TMR has expressed its interests in the utilisation of its stockpiled FRP girders. As a result, the project direction was varied, leading to the following tasks in Year 3:

- visual inspection of selected FRP bridges and replacement components in service
- condition assessment of stockpiled FRP girders
- performance load testing of stockpiled FRP girders, with the number to be tested depending on the available budget.

This report presents the findings of the inspection/testing program undertaken in Year 3 and provides recommendations for the utilisation of the unused stockpiled FRP girders and the updating of the TMR guidelines on condition rating of FRP components and structures.

1.2 Scope/Methodology

1.2.1 Scope

The scope of this project included the following:

- Establish a knowledge base for the performance of existing components/bridges under TMR jurisdiction.
- Identify issues related to the interaction between a replacement FRP component and existing timber components on timber bridges.
- Propose revisions of the guidelines for condition rating of FRP components and structures if appropriate.
- Provide a basis for the more reliable assessment of the durability and long-term performance of FRP girders.
- Provide the basis for TMR to make informed decisions regarding the utilisation of currently-stockpiled FRP girders.

1.2.2 Inspection Methodology

Close visual inspection was carried out on the FRP replacement components and other bridge components in interaction with them. The inspection involved:

- close visual inspection, seeking to identify localised defects and damage such as local crushing caused by impact, local cracks due to inappropriate storage and lifting, fibre blooming, degradation of cut surfaces and damage due to vandalism
- checking for delamination by tapping the girder with a rubber hammer
- checking for cracks in the FRP materials and any signs of UV or moisture deterioration
- identifying issues related to the interaction between a replacement FRP component and existing timber components on timber bridges.

Particular attention was paid to defects related to moisture effects. According to Karbhari (2009), the influence of moisture on adhesives is believed to play a critical role in the debonding failure of FRP/adhesive systems. The plasticisation effect of moisture enhances the fracture toughness of adhesives due to greater plastic deformation and enhanced crack-tip blunting mechanisms. However, the moisture may penetrate along the fibre-matrix interphase and cause deleterious effects to the fibre-matrix bond, resulting in loss of integrity at that level. In addition, moisture and chemical solutions have also been shown to cause deterioration within fibres. In the case of glass fibres, degradation is initiated by the extraction of ions from the fibre by the water. These ions combine with water to form bases (alkaline solutions), which etch and pit the fibre surface, resulting in flaws that significantly degrade strength. This can result in premature fracture and failure of fibres and laminates.

Inspection tools include:

Camera	Hammer	Flash light
Measuring tape	Spirit level	Callipers
Cracking card	Ladder	Magnified glass
Markers	Binoculars	

1.3 Outlines

The report includes eight sections and an Appendix. Section 1 provides the background information on this project, with the remaining sections detailing the results of this investigation as follows:

- Section 2 provides an overview of WCFT and LOC girders as pre-engineered fibre composite products. Discussion is provided on issues related to the influence of design and manufacture philosophy on structural concepts and durability of these girders, and the significance of typical defects. A defect rating system adopted from the UK Highways Agency is proposed for rating FRP components within this project. Durability and long-term performance investigation techniques are then described.
- Section 3 discusses the findings from the visual inspections of stockpiled FRP girders.
- Section 4 provides a summary of the results of serviceability testing of selected Wagners girders and compares the findings with the past test results to identify changes in the performance and stiffness of these girders.
- Section 5 discusses the findings of the inspection of replacement components on existing timber bridges. Defect mapping is provided for each component, with the severity and extent of defects and the possible root causes discussed.
- Section 6 discusses findings from the inspections of complete FRP bridges, including defect mapping.
- Section 7 summarises the discussions of the overall inspection/testing program. A discussion on the capacity and failure modes of the FRP girders based on the past test results is also provided.
- Section 8 provides recommendations and conclusions based on a number of assumptions regarding TMR's future strategy in relation to FRP materials. Particularly, recommendations are provided on the utilisation of the stockpiled FRP girders and continuing monitoring of FRP components and complete FRP bridges on the network, as well as proposed future review and investigations.

TMR standard drawings of FRP replacement components for existing timber bridges are reproduced in Appendix A.

2 DURABILITY ASSESSMENT OF PRE-ENGINEERED FRP MEMBERS

The FRP girders used for the replacement of timber girders on TMR timber bridges are pre-engineered products. Under this arrangement:

1. TMR specified functional requirements including the design basis
2. suppliers developed the design, manufactured and construction philosophies, then defined and delivered the product to technical and QA requirements.

The result of this arrangement is that TMR has less visibility on technical details compared with conventional delivery.

Fibre composite (FC) components such as girders are delivered using this approach because materials must first be selected and engineered before components can be engineered, and material selection and engineering is highly dependent on the manufacturer's production capability. This delivery approach facilitates optimisation across multiple processes, with the trade-off being a reduction in technical control by TMR.

By comparison, the function, design, manufacture and construction philosophies (including QA) of conventional civil infrastructure components (concrete and steel) are specified by TMR, with suppliers and contractors delivering in accordance with specified requirements.

The aim of TMR's 'Bridging the Gap' project was to develop FC bridge girders as a replacement/substitute for degrading girders in existing timber bridges in Queensland. The purpose of this project was to prototype and trial market-ready products which met the functional requirements specified by TMR. Two commercial suppliers, namely Wagners Composite Fibre Technologies (WCFT) and Loc Composites (LOC), were selected by TMR to supply production prototypes for evaluation. The development of improved production processes and systems could lead to the availability in the Australian market of commercially-viable FC bridge girders through the use of advanced manufacturing techniques. A total of 91 composite girders which passed the performance criteria (stiffness and strength) provided by the TMR were manufactured and proof-loaded.

This section discusses the principles applied to the development of the design of TMR's FRP girders. It provides a basis for understanding how the FRP components function. In light of this understanding, the performance of these girders can be assessed based on the factors that may affect the strength and durability of the girders. This is discussed in subsequent sections of this report.

2.1 Influence of Design and Manufacture Philosophy on Durability

The cost per unit stiffness and cost per unit strength of FC components is substantially greater than conventional infrastructure materials such as steel, reinforced or prestressed concrete. Considerably greater attention to detail regarding the targeted use of materials is therefore required if FC elements are to become commercially competitive. This is a key driver for the selection of the pre-engineered delivery process rather than the conventional delivery process for FC products. The manufacturing capability of the supplier is a key consideration for both manufacturing and design choices, for example:

1. WCFT manufacturing is focused on the production of pultruded elements, then combining these elements during the product assembly process, primarily through the use of adhesives
2. LOC manufacturing is focused on the production of cast elements (albeit continuous casting), then combining these elements during the product assembly process, primarily through the

use of adhesives, but ensuring that elements can resist all significant applied actions via fibre dominated mechanisms.

This subtle difference in philosophy means that glue-line integrity is critical to overall structural integrity for the WCFT girders, whereas glue-line integrity is normally not critical for LOC girders because all glue lines are reinforced by fibres which transfer both primary and secondary loads. During the 'Bridging the Gap' project, the WCFT concept was developed such that the Stage 3 girders incorporated additional laminates in critical areas to ensure the fibre dominated behaviour; this illustrates the importance of the product tailoring as part of the project.

A key advantage of using FC is their durability under a wide range of conditions. However, this durability still relies on the fibres (glass, etc.) being protected by adequate layers of resin. Deterioration in the surface layers of resin will still lead (eventually) to a loss of structural integrity. UV light, abrasion and water ingress all contribute to the slow deterioration of surface resin layers. Fire is also a concern for durability. LOC utilise phenolic resin in their continuous casting process, and this material (particularly combined with mineral filler) is inherently resistant to fire (it will char but not burn), whereas the vinyl ester and epoxy resins used by both suppliers are susceptible to fire and will burn even with heavy mineral filler loading. Whilst additives can be added to the production resin to inhibit this risk, or fire-resistant coatings can be applied to finished products to address this problem, both approaches add cost to the final product.

FC are inherently strong but flexible (low stiffness compared with conventional infrastructure materials). This attribute, combined with their (relatively) high cost, means that there are opportunities to consider the incorporation of conventional materials with FC (such combinations are typically referred to as hybrid members). Two examples utilised by LOC are:

1. treated ply used as a 'core' material in prototype LOC girders prior to the availability of phenolic sandwich board material
2. steel (encased in FC elements) used as a cost-effective method to control deflection in LOC girders up to and beyond the serviceability limit state.

While these examples provide significant value in terms of cost-effective material use, they potentially introduce additional durability concerns since, for both examples, the conventional materials are susceptible to environmental deterioration at a faster rate than their FC hosts, and the secondary effects of deterioration (e.g. volume increase) can affect the overall integrity of the hybrid members. The alternative to this approach is typically to increase the amount of glass fibre in the member; however, this is not a cost-effective way of meeting member stiffness criteria.

In recognition of the surface deterioration mechanisms of FC materials in the environment, both WCFT and LOC products typically are protected by a resin later applied once major manufacturing activities are complete. This coating ranges from 'paint' to poured particulate filled resin (PFR) render, with the choice of resin based on compatibility with the substrate resin finish, and potentially fire mitigation requirements.

Careful functional consideration was required by both suppliers in their product development to accommodate fitting and fastener requirements associated with integration of these FC components into structural systems. Compromises are typically required that include consideration of:

- construction tolerances
- tools and skills readily available of site
- variability of structure types can configurations
- durability of site adjustments, particularly fastener holes.

Fitting and fastener activities have the potential to increase the likelihood of moisture penetration into load-carrying fibres. While this is a relatively minor issue for glass fibres, its impact on any incorporated steel of timber elements is potentially much more damaging (see earlier discussion on hybrid members).

2.2 WCFT Girders

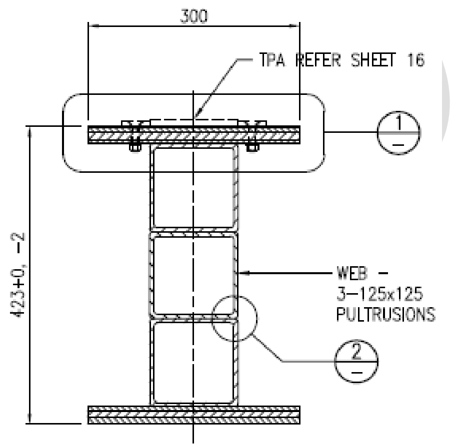
Three types of girders were differentiated based on the cross-section configuration, namely S1, S2 and S3 (Figure 2.1 to Figure 2.3). Refer to Appendix A.1 for the girders' detailed design drawings. The main components on the cross-section of the Wagner's girders include:

- three pultruded square hollow sections stacked on top of one another, acting as the member web
- pultrusion flange plates
- composite angles or flat plates used to enhance the shear capacity of the girder at either end of the girder
- HPR26 (Epoxy) adhesive used to glue the components together.

The difference in the design between the three girder types is that the flange of S1 girders comprises three thin pultrusion plates glued together by adhesive, while girders S2 and S3 use a single thick pultrusion plate. In addition, for S1 and S3 girders, pultruded composite angle reinforcement was placed at the end of sections of all flange-web connections for 2.1 m length at the end of the girders, while only flat plates were used on the web of the S2 girders on the same girder segments. The S3 girders were further improved by Wagner, in which three layers of 750 gsm TRIAXIAL were added to both sides of the web in between angles to improve the shear strength of the girder.

While the term 'pultrusion' is used with respect to Wagners elements, the square hollow section elements are actually 'pull windings', meaning that ±45° fibres are included in element manufacture. This means that shear behaviour transverse to the element is fibre dominated; however, these fibres are discontinuous at glue lines. Consequently, the integrity of the girders depends on the strength of various glue lines used to form the cross-section, unless additional laminates are added to reinforce the glue lines.

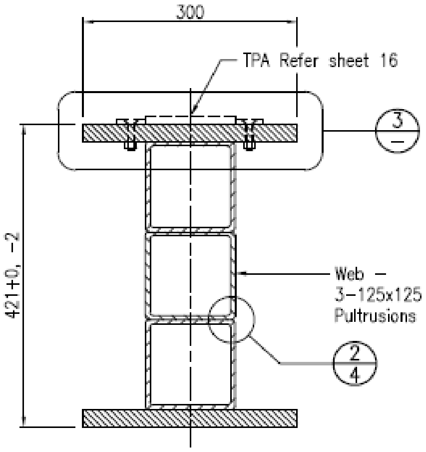
Figure 2.1: WCFT FRP girder type S1



Source: TMR standard drawing No. 2285.

Stockpiled girder type S1

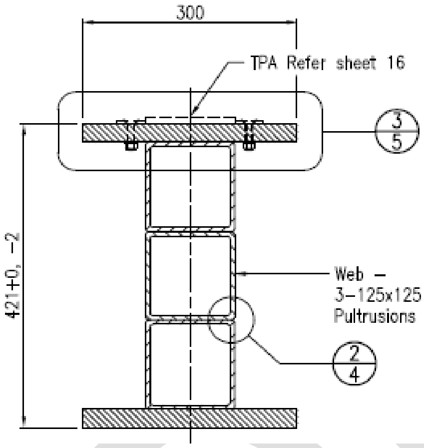
Figure 2.2: WCFT FRP girder type S2



Source: TMR standard drawing No. 2285.

Stockpiled girder type S2

Figure 2.3: WCFT FRP girder type S3



Source: TMR standard drawing No. 2285.

Stockpiled girder type S3

2.3 LOC Girders

Two types of LOC girders were identified based on the cross-section configuration and material type, namely LOC400 (Figure 3.15) and LOC420 (Figure 3.16). Refer to Appendix A.2 for the girders' detailed design drawings. The typical girder cross-section was constructed using a number of LOC panels, hybrid reinforcement modules and PFR. While a paint coating was applied to the external surfaces of the LOC400 girders, a thick coating of resin was applied to the LOC420 girders. This resin was believed to contain fire retardant to protect the girder against UV and premature failure in the event of fire.

The LOC panels were produced using a sandwich construction consisting of two layers of triaxial glass in each skin with a phenolic PFR core. The triaxial glass layers comprise glass fibres distributed in the 0° and ±45° directions. The panels were used to form the webs of the girders and the deck panels. They carry the majority of the shear in the girders and contribute to bending capacity.

The hybrid reinforcement modules forming the top and bottom flanges consist of:

- square hollow pultruded FC members

- a deformed high-tensile steel reinforcement bar
- PFR that fills the voids within the hybrid module.

The main role of the steel reinforcement bar is to provide stiffness, particularly in the working load range. The FC member provides a corrosion-protective shell for the steel element; it also provides strength and stiffness. The filled resin system binds the steel and FC members together into a single structural unit, simultaneously providing an additional thick layer of corrosion protection to the steel.

As discussed in Shaw & van Erp (2009), the LOC girder design is robust with respect to ultimate loading as follows:

- When the hybrid unit is loaded past its serviceability level, the steel reinforcement bar will yield at a strain of approximately 0.25%. As the steel yields, its load-carrying capacity plateaus while the composite tube will continue to carry increasing load until it reaches its failure strain at approximately 1.6–1.8%. This failure strain is significantly below the ultimate failure strain of the steel reinforcement bar. Consequently, both the steel and composite tube continue to contribute fully to the load-carrying capacity of the hybrid element until the composite tube breaks, at which point there will still be residual capacity in the steel provided glass rupture has not compromised member integrity.
- Due to the yielding of the steel, the hybrid member exhibits a ductile behaviour which is a highly desired attribute in civil engineering structures. In addition, a significant degree of redundancy is available within the new structural element. It is extremely unlikely that all materials in the hybrid member will fail at the same time. If one of the materials accidentally fails, there other materials that offer alternative load-carrying capacity.
- Resin-dominated failure modes (typically brittle and defect sensitive) are not permitted.
- The tensile and bending contributions of PFR and EPS (Expanded Polystyrene)-PFR are ignored.
- Glued joints are limited to the lap shear mode of failure. No tensile glued joints are permitted other than secondary tension in joints.

2.4 Significance of Typical Defects in FRP Girders

The primary load-carrying elements of FRP girders are glass fibres and, in the case of LOC girders, steel reinforcing bars. Clearly, defects that impact on the integrity of these elements have major structural significance. These elements are joined with glue lines, and they are typically configured to transfer shear across the glue line between adjacent elements. Typically, large surface areas are involved with the joining of these components, and theoretically there will be significant excess capacity to transfer shear requirements. However, glue lines can behave in a relatively brittle manner such that a defect in the glue line can initiate progressive failure with much lower shear stress than would be indicated theoretically. Glue-line defects may not be externally visible. Glue-line defects are less significant where they are appropriately reinforced by fibres (for example the LOC confining glass triaxial wrap).

Further to this, and for the same reason, both longitudinal and vertical shear actions should ideally be carried by fibres. To the extent that shear actions are carried by resin alone, or resin with fibre filler that is not aligned with shear action, any defects have the potential to be more significant than they would be if shear was designed to be carried by fibres.

Evidence of deterioration of conventional materials embedded in hybrid products may provide evidence of a significant structural defect because:

1. Conventional materials (timber or steel) will deteriorate more rapidly than FC materials. As a result, the load carrying capacity of the element will be compromised earlier than would be the case for an all FC element.
2. Deterioration of conventional materials is often associated with volumetric swelling, and this can reduce the integrity of the whole structural element. This is generally the more significant concern.

Surface defects that impact the weathering resistance of the exterior surface of FC members have the potential to impair durability and reduce life. The cost-benefit trade-off between the cost of repair and the value of the repair is unclear at this stage and warrants further consideration. In addition, part of the purpose of the 'Bridging the Gap' project was to gain experience with FC materials. Consequently, recommendations from this project should consider the:

1. desirability to repair defects
2. variability of methodologies to repair defects
3. value of understanding outcomes for the member if no repair is undertaken.

2.5 Defect Rating of FRP Components

In order to provide a defect mapping for individual FRP components to better identify the nature and future propagation of the defects, it is recommended that a defect extent and severity rating system, as used by the UK Highway Agency (2007), be adopted to rate the FRP components. Defect mapping provides more detailed documentation of defects than the standard condition state rating of members normally used for bridge inspection in Australia. Defect mapping allows the condition of a bridge element to be recorded in terms of the severity of damage/defect and the spatial extent of the damage/defect. The following definitions were adopted to describe the extent and severity parameters:

- extent: the area, length or number (as appropriate) of the bridge element affected by the defect/damage.
- severity: the degree to which the defect/damage affects the function of the element or other elements on the bridge.

Both extent and severity are parameters that are used to inform decisions about maintenance planning and management. The use of separate codes for each parameter eliminates any obscurity in the distinction between, for example, a single but severe defect and extensive but superficial deterioration. Codes that may be used to describe the extent and severity levels are shown in Table 2.1 and Table 2.2 respectively.

Permissible combinations of severity and extent are shown in Table 2.3. This shows that some severity/extent combinations are not permissible, e.g. 2A, 3A, 4A and 5A. These combinations are not permitted because it is not possible to have a severity condition greater than 1 when the extent description is 'no significant defect'.

Table 2.1: Defect Extent codes

Code	Description
A	No significant defect
B	Slight, not more than 5% of surface area/length/number
C	Moderate, 5%–20% of surface area/length/number
D	Wide: 20%–50% of surface area/length/number

Code	Description
E	Extensive, more than 50% of surface area/length/number

Table 2.2: Generic Severity description

Code	Description
1	As new condition or defect has no significant effect on the element (visually or functionally)
2	Early signs of deterioration, minor defect/damage, no reduction in functionality of element
3	Moderate defect/damage, some loss of functionality could be expected
4	Severe defect/damage, significant loss of functionality and/or element is close to failure/collapse
5	Extensive, more than 50% of surface area/length/number

Table 2.3: Permissible combinations of severity and extent

Extent	Severity				
	1	2	3	4	5
A	1A				
B		2B	3B	4B	5B
C		2C	3C	4C	5C
D		2D	3D	4D	5D
E		2E	3E	4E	5E

Note: shaded boxes represent non-permissible Severity/Extent combinations.

In light of this rating system, the detailed severity descriptions of FRP components are proposed in Table 2.4. It should be noted that, while this Table is used for rating FRP components on existing bridges, the defects of stockpiled components can also be rated using the second and third rows.

When an element has more than one type of defect/damage, the following guidelines should be used to assess its condition:

- When the severity of one defect is adjudged to be at least one severity category higher than any other defect on the element, then the severity for the element is defined based on this dominant defect. Other defects do not reduce the functionality of the element beyond that caused by the dominant defect. The extent code in this case should correspond to the area affected by the dominant defect alone.
- Where the cumulative effect of several defects is adjudged to be the same as, or worse than, the effect of the dominant defect then the severity code should be reported based on the cumulative effect of all the defects on the element. Where no dominant defect is evident, the severity should be based on the cumulative effect of the defects the inspector feels are relevant. The extent code in this case should correspond to the area affected by all defects considered in assessing the severity.

Table 2.4: Severity descriptions for FRP components

Severity	1	2	3	4	5
.1	▪ No sign of loss of protective coating including flaking, peeling or spalling.	▪ Minor loss of protective coating; minor water stains and discoloration with areas not exceeding 1000 mm x 50 mm (or equivalent).	▪ Moderate loss of protective coating; moderate water stains and discoloration of an area between 1000 mm x 50 mm (or equivalent) and 1000 mm x 100 mm (or equivalent); moderate deformation of the coating of resin (LOC girder).	▪ Major signs of coating damage; loss of protective coating in an area in excess of 1000 mm x 100 mm (or equivalent).	▪ Disintegrated through damage.
.2	▪ No loss of section, delamination, cracking, chipping, impact damage, fire damage distortion, twisting, bulging.	▪ Minor delamination of composites (WCFT girder) ▪ Minor longitudinal cracking less than 600 mm long, 1 mm wide and no greater than 15 mm deep; minor deformation of the coating of resin (LOC girder). ▪ Minor crushing and damage of composite due to impact.	▪ Delamination of composites; moderate cracks in the glue line; minor impact damage up to a depth of 1 mm and an area not exceeding 50 mm x 50 mm or equivalent in any one location with damage restricted to the first layer of fibreglass (WCFT girder). ▪ Fire damage may also be visible but with no apparent distortion of the girder. ▪ Transverse cracks less than 50 mm long, 1 mm wide, and 5 mm deep; longitudinal cracks between 600 mm and 1000 mm long, 1 to 2 mm wide and not exceeding 15 mm deep; moderate deformation of the coating of resin (LOC girder).	▪ Significant impact damage or delamination, cracking or splitting exceeding 1 mm deep and/or exceeding an area of 50 mm x 50 mm or equivalent; tear of fibre at bolt locations; excessive deflection under load, twisting, evidence of fire damage causing cracks or distortion, bulging in the sides of pultrusions; moderate to severe cracks on the glue line (WCFT girder). ▪ Transverse cracks between 50 and 100 mm long, 1 to 2 mm wide, and 5 mm deep; longitudinal cracking exceed 1000 mm in length, are greater than 2 mm wide and exceed 15 mm in depth; severe deformation of the coating of resin (LOC girder).	▪ Collapsed or collapsing.
.3	▪ No visible signs of loose crushing or splitting in the packer/girder seating.	▪ Minor splitting in the packer/girder seating.	▪ Minor misalignment or moderate splitting or minor crushing of packer/girder seating.	▪ Significant misalignment or splitting or moderate to severe crushing of packer/girder seating.	▪ Girder separated into multiple elements.
.4	▪ No signs of rusting or damage to fixing.	▪ Non-structural bolts loose, minor corrosion of fixings.	▪ Non-structural bolts missing, moderate corrosion of fixings.	▪ Structural fixings missing.	▪ Failure of element due to missed/failed fixings.

2.6 Durability and Long-term Performance Investigation Techniques

In-service investigation techniques were required for this project to investigate the performance of TMR FRP components and structures including close visual inspection and serviceability load testing. The scope of 'Bridging the Gap' project included the development of:

1. design criteria and functional specifications for FC members
2. laboratory testing to validate structural performance of girders against (1)
3. development of recommended field fitting techniques (drilling for fasteners, etc.).

Activities excluded from the 'Bridging the Gap' project included no post installation inspection or defect mapping and no in-service performance testing (post installation). Consequently, there was no established baseline for either of these aspects available to the project.

Consequently, this (durability and long-term performance project) was designed to reference aspects that were baselined as part of 'Bridging the Gap' and provide a current baseline for post-installation defect mapping. This involved detailed visual inspection using the method outlined in Section 1.2.2, and selective benchmarking of service load structural behaviour of some girders against similar tests undertaken during the 'Bridging the Gap' project.

Additional investigative techniques excluded from this investigation were as follows:

- Non-destructive inspection techniques such as acoustic and ultrasonic assessment, thermal imaging, and radar/microwave NDT.
- Load testing under known and in-service loads to investigate the changes in the stiffness and strengths of the structural components over time, using traditional instrumentation techniques such as strain gauges, deflection sensors and accelerometers. The results of the load testing can provide information on the development of damage, the degradation of materials, and changes in the stiffness/strength of components and the whole structure.
- Durability testing to investigate the effects of UV, moisture and environmental factors on the materials. The following test duration and testing condition are recommended to produce meaningful results (Karbhari 2009):
  - Testing over extended (18+ months) time periods. Tests conducted over short time periods (less than 18 months) can yield misleading results due to the effects of post-cure and slow interphase and fibre level degradation. This can lead to an erroneous level of comfort in some cases.
  - Testing under combined conditions (stress, moisture, solution, temperature, and/or other regimes) at both the materials and structural levels is critical.

The rationale for the exclusion of these techniques from this investigation included:

1. limited funding
2. the lack of baseline data for comparison
3. insufficient evidence to target more sophisticated investigations.

More in-depth investigation techniques may be justified to better understand the root causes and the extent of material degradation in the future to better understand long-term trend performance of these components/structures.

3 INSPECTION OF STOCKPILED FRP GIRDERS

3.1 General

TMR currently stocks some FRP girders at a number of Districts. These girders were manufactured by WCFT and LOC. A small number of girders have been used on existing bridges as discussed in Section 5. TMR is looking at assessing the current condition of the remaining girders, in order to inform the Districts regarding their utilisation.

A total of 53 FRP girders, 9.7 metre long and fabricated by WCFT, are stockpiled at the University of Southern Queensland (USQ) in Toowoomba. Twenty-two LOC girders were distributed to three TMR Districts, including North Coast at Maroochydore (six girders), South Coast at Nerang (eight girders), and Darling Downs at Charlton (eight girders).

Visual inspections were undertaken at the stockpiles, including:

- close visual inspection to check for localised defects and damage such as local crushing caused by impact, local cracks due to inappropriate storage and lifting, and degradation of coating surfaces
- checking for delamination by tapping the girder with a rubber hammer
- checking for any signs of degradation of the FRP materials due to environmental impacts such as ultraviolet or moisture.

3.2 WCFT Girders

The WCFT girders stockpiled at the USQ Toowoomba campus were inspected on 19/09/2017. The girders had been stacked in two layers (Figure 3.1) since circa 2012. Due to limited space and lifting capacity, only 12 girders were transported to a nearby yard for detailed inspection, including five S1, five S2 and two S3-type girders (Table 3.1). Anecdotally, these girders showed typical deterioration and damage, and representing the common condition of the remaining girders. All girders were previously marked with unique identification numbers, including the girder number and the production lot.

Figure 3.1: WCFT FRP girder stockpile at USQ

Wagner FRP Stockpiled – image 1

Wagner FRP Stockpiled – image 2

Table 3.1: Identification of the WCFT FRP girders inspected at USQ

Type S1	Type S2	Type S3
Girder 1–3	Girder 2–1	Girder 3–3
Girder 1–6	Girder 2–2	Girder 3–4
Girder 1–15	Girder 2–3	–
Girder 1–16	Girder 2–4	–
Girder 1–17	Girder 2–5	–

3.2.1 *Girder Type S1*

The following main defects were found on the Type S1 girders:

- Chipping: located at the edge of the flanges (Figure 3.2 and Figure 3.3), due to the failure of the paint coating which resulted in further degradation of the external layer of the bond between flange plates. The length of each damaged area was approximately 100–200 mm. There were typically one to two areas along an edge.
- Splitting/delamination of flange layers: minor delamination of flange layers found mostly close to the ends of affected girder (Figure 3.4 and Figure 3.5).
- Failure of paint coating: blistering on the surfaces of girders where they were exposed to direct sunlight (Figure 3.6 and Figure 3.7).
- Fibre exposure: this defect is common to all types of girders. It occurs on the top surface of the top flange of the girders located on the top layer of the stockpile, possibly due to direct exposure to sunlight. The paint coating had completely failed such that some fibres are unprotected (Figure 3.8).
- Water ponding and debris accumulation were observed on top surface of the bottom flange causing discoloration and peel of the coating (Figure 3.3 and Figure 3.10).

Figure 3.2: Chipping noted at top flange of Girder 1-15

Figure 3.3: Typical chipping noted at bottom flange of Girder 1-3

Figure 3.4: Typical delamination noted at bottom flange of Girder 1-6

Figure 3.5: Delamination noted at bottom flange of Girder 1-17

Figure 3.6: Coating defect, holes noted due to air bubbles on painting, Girder 1-6

Figure 3.7: Coating deterioration at edge of flange, Girder 1-3

Figure 3.8: Fabrication defect with fibre exposed noted at top and bottom of Girder 1-3

3.2.2 Girder Type S2

For FRP girders type S2 the most common defect was related to the failure of the coating due to exposure to direct sunlight or where there is water ponding and debris accumulation. Paint blistering and/or peeling were observed at these areas (Figure 3.9 and Figure 3.10) Most of the blistering was found on Girders 2–1. Other Type S2 girders had minor blistering and loss of paint coating. Exposure of the fibres was observed on Girders 2–3 and Girders 2–6 (Figure 3.11), possibly due to physical impact during handling and storage.

Figure 3.9: Typical blistering and peeling of coating

Figure 3.10: Typical peeling of coating noted at the edge of flange

Figure 3.11: Typical peeling of coating at the top of girder G2-6 with fibre exposure

3.2.3 Girder Type S3

Similar to girder Types S1 and S2, the most common damage observed on the Type S3 girders was coating damage with fibre exposed, and coating blistering (Figure 3.12 and Figure 3.13). It is worth noting that the finished surfaces on the web between the strengthened angles of these girders were very coarse and uneven (Figure 3.14).

Figure 3.12: Coating and peeling with fibre exposed noted at top of girder

Figure 3.13: Blistering noted at edge of flanges

Figure 3.14: Typical defective finishing surfaces noted at the end of girder type S3

3.2.4 Summary of Defect Ratings of WCFT FRP Stockpiled Girders

In general, the stockpiled girders had a fair defect rating (2B–3B – see Section 2.5) with mostly surface damage and coating deterioration. In particular, the following defects were observed:

- minor delamination at the flange edges (S1 girders)
- minor chipping at isolated locations
- failure of paint coating where water was ponding on the top face of the bottom flange
- fibre exposure due to wear of the coating and physical impact on the top surface.

These defects were localised and did not extend throughout the girders, nor were they type-specific. Due to the nature of these defects, it would be prudent to undertake minor repair and re-coating of the girders before use on existing bridges.

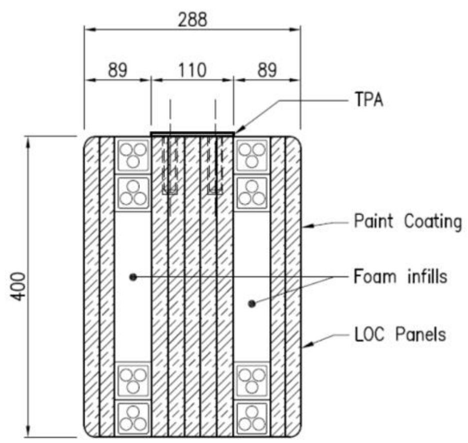
The remaining girders stored at the original location had similar types and extent of deterioration. The top face of the top flange of the girders on the top layer had deteriorated the most with the scattered failure of the paint coating which resulted in the exposure of fibres in small areas. The underside and other surfaces of the girders lower down in the stockpile appeared to be intact and in good condition.

Refer to Section 8.2.1 for recommendations regarding the utilisation of these girders.

3.3 LOC Girders

The FRP girders stockpiled at the RoadTek storage yards in Nerang and Maroochydore were inspected on 01/11/2017, while the girders at Charlton were inspected on 28/11/2017. Two types of girders were identified based on the cross-section configuration and material: LOC400 (Figure 3.15) and LOC420 (Figure 3.16). Currently, eight FRP LOC400 girders are stockpiled at Nerang, six FRP LOC420 girders are at Maroochydore, and eight FRP LOC420 girders are at Charlton.

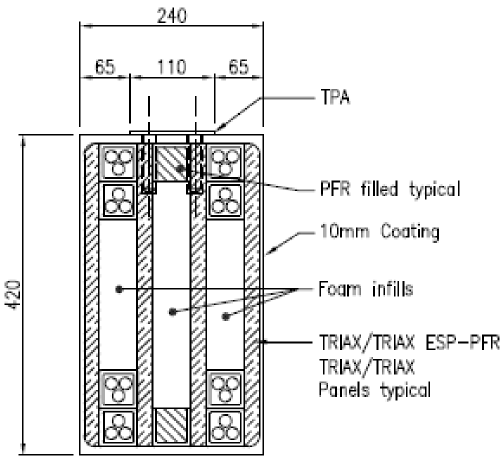
Figure 3.15: LOC FRP girder type LOC400



Source: TMR standard drawing No. 2280

Girders type LOC400 at Nerang

Figure 3.16: LOC FRP girder type LOC420



Source: TMR standard drawing No. 2280

Girders type LOC420 at Maroochydore

3.3.1 LOC400 Girders at Nerang

The eight LOC400 girders at Nerang were stacked in two layers with ground contact (Figure 3.17). Due to the limited space between the girders, only visible areas were inspected. The girders were hand-marked with numbers from 2–11, without number 1 and 3. Girders 2, 6, 7, 8 and 9 had a thicker paint coating layer compared with the other girders.

In general, the girders had a fair defect rating (2B – see Section 2.5). The most pronounced deterioration observed was a loss of the coating at isolated areas along the top edge of the girders (Figure 3.18). There was a minor tear of the FRP material at the edge of girder 6 due to physical impact (Figure 3.19). There were no signs of material degradation.

Figure 3.17: LOC FRP girders stockpiled at Nerang

LOC FRP stockpile at Nerang

Figure 3.18: Typical loss of coating at girder top edges

Figure 3.19: Minor tear of the FRP material due to physical impact

3.3.2 LOC420 Girders at Maroochydore

The six LOC420 girders at Maroochydore were located next to each other on packers off the ground (Figure 3.20). Due to the limited space between the girders, only visible areas were inspected. The girders were hand-marked with numbers 1, 5, 9, 11, 17 and 20.

Figure 3.20: LOC FRP girder stockpiled at Maroochydore RoadTek

LOC FRP Stockpiled – image 1

LOC FRP Stockpiled – image 2

The following main defects were identified in the LOC420 girders at Maroochydore:

- Transverse cracking at the top surface of girders 5, 9 and 11 on one-third to a half-length of each girder. The crack widths ranged from 0.15 mm to 0.5 mm (Figure 3.21 and Figure 3.22).
- Pitting on the top surface of girders 17 due to missing coating material (Figure 3.23), likely due to environmental effects.
- Surface deformation (in ripple form) on all side faces of all girders (Figure 3.24). Given that the LOC420 girders have a very thick coating of resin (10 mm), as shown in Figure 3.16, this issue might have originated during the fabrication process.
- Rust-like stains at the underside of the girders (Figure 3.25). It is not known if water had penetrated the material and corroded the internal steel. Based on the TMR standard drawings, these girders were wrapped with three sheets of E-glass woven roving fabrics (AR117 450 gsm), following which a 10 mm thick coating was applied. As a result, steel rust

occurring due to water penetration was highly unlikely due to the wraps covering the cross-section of the girder. In addition, every three N16 steel bars were surrounded by PFR and encapsulated by 50 x 50 pultrusion. An alternative explanation may be that some phenolic constituents may have leached from the girders. This resin is yellow to brown in colour. Pale phenolic resin colours immediately after production during storage or processing.

A rating of 2D–2E (see Section 2.5) was given to these girders on the basis that the stains were of phenolic origin and the coating irregularity was a manufacturing issue. The former in particular requires substantiation.

Figure 3.21: Cracking on top of girder 11 at mid-span

Figure 3.22: Cracking on top of girder 9 at mid-span

Figure 3.23: Pitting defects on the top surface of girder 17

Figure 3.24: Typical deformation of the coating of resin

Figure 3.25: Typical rust-type stains at the underside of the girders

3.3.3 LOC420 Girders at Charlton, Toowoomba

The FRP LOC420 girders at Charlton were located next to each other on timber logs (Figure 3.26). Due to the limited space between the girders, only visible areas were inspected. There was no numbering on these girders.

Figure 3.26: LOC FRP girder stockpiled at Maroochydore RoadTek

LOC FRP Stockpiled – image 1

LOC FRP Stockpiled – image 2

Similar to the girders of the same type (LOC420) stored at Maroochydore, the girders stored at Charlton had similar defects but were less severe:

- cracking at the top surface of four girders with a crack width of up to 0.45 mm (Figure 3.27 and Figure 3.28).
- surface irregularities (in ripple from) at various locations on the side faces off all girders (Figure 3.29); some fibre was exposed on these areas
- minor damage as shown in Figure 3.30.

These girders were rated 2D–2E (see Section 2.5) assuming that the coating irregularity was a manufacturing issue.

Figure 3.27: Cracking on top of girder 6 at mid-span

Figure 3.28: Cracking on top of girder 8 at mid-span

Figure 3.29: Deformation of coating at various locations

Figure 3.30: Minor damage on coating

3.3.4 Summary of Condition of Stockpiled LOC Girders

It is evident from the observations that the defects in the LOC girders were type-specific. While the LOC400 girders stored at Nerang could be used following minor repairs of the paint coating, the LOC420 girders should not be used without further investigation. Refer to Section 8.2.2 and Section 8.2.3 for the recommendation regarding the utilisation of these girders.

4 SERVICEABILITY LOAD TESTING OF STOCKPILED GIRDERS

4.1 General

Three stockpiled WCFT girders representing three production stages (different cross-section) configurations were selected for the serviceability testing. These tests replicated the performance tests conducted at the USQ test facility as part of the 'Bridging the Gap' project in 2010–2012. The setup of the tests was the same as that used in the earlier testing (refer to Section 4.2.1 and Section 4.3.1) for comparison purposes.

Bending and shear serviceability tests were performed on each selected girder. The purpose of the bending testing was to verify the girders' flexural performance and bending stiffness, while the purpose of the shear testing was to evaluate the shear performance of the girders and the reliability of the glue lines.

The following data was derived from the performance tests:

- load-deflection curves
- the bending stiffness, EI, based on the load-deflection curves
- the behaviour of the test girders during testing
- observation of possible damage due to the test loads.

These results were compared with the past test results, which are available from the 'Bridging the Gap' project, including:

- testing of stage 1 girders manufactured by WCFT – 2010 (20 girders) (Aravinthan, Kahandawa & Manalo 2012a)
- testing of stage 2 girders manufactured by WCFT – 2011 (20 girders) (Kahandawa, Manalo & Aravinthan 2011)
- testing of stage 3 girders manufactured by WCFT – 2012 (20 girders) (Aravinthan, Kahandawa & Manalo 2012b).

In addition, the test results were compared with the required serviceability criteria for bending and shear for replacement components on existing timber bridges specified by TMR.

The assessment of the stockpiled girders was based on the acceptance testing criteria specified in TMR's MRTS59-2014 *Manufacture of FRP Composite Girders* (Queensland Department of Transport and Main Roads 2017) as follows:

1. Deflection versus applied load plot shall be linear in the serviceability limit state region as shown in the load deflection curve in *Chapter 16: Design of Fibre Reinforced Polymer (FRP) Composite Girders*, of the department's *Design Criteria for Bridges and Other Structures* (TMR 2018).
2. Girders with deflections higher than specified shall be rejected. Girders with deflections lower than the specified value may be accepted subject to ascertaining that the reason for the difference does not compromise the performance of the girder.
3. Girders shall show no sign of distress during testing.

Additionally, for FRP girders to be used as replacement for timber girders on timber bridges, the target bending stiffness, EI, is 2.96 x10^{13} Nmm^2 (TMR Standard drawing SD2285).

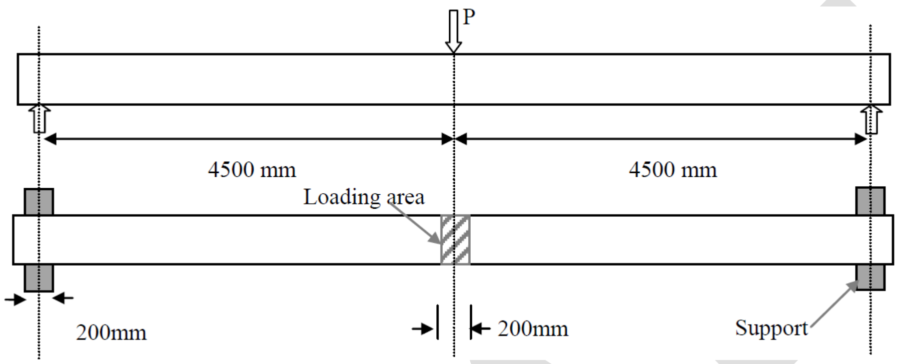
Based on the findings from the inspection of stockpiled girders at USQ carried out in September 2017 (Section 3.2), the three girders with the worst condition (G1-3, G2-3 and G3-3) were selected for testing; coincidently they were all girder number 3.

4.2 Serviceability Bending Testing

4.2.1 Test Setup

The serviceability bending test was conducted using a 3-point static bending test set-up shown in Figure 4.1. The girder was simply-supported at its ends. A maximum load of 165 kN was applied at the mid-span of the girder.

Figure 4.1: Bending test set-up



Source: Aravinthan et al. (2012b).

Each end of the girder was placed on a steel plate sitting on top of a steel I-beam. The centre-to-centre distance between the supports was 9000 mm. The applied load was transferred to the girder through a steel plate and recorded during the tests via a load cell which was built in to the jack. A sampling rate of 10 Hz was used to record measurements.

The deflections of the test girder were measured at five locations, including the mid-span section, two quarter-span sections and two sections close to the supports. Due to the availability of instrumentation, different types of displacement sensors were used, including two LVDTs, two string pots and one laser. A string pot was installed at the mid-span.

Two load cycles were applied to girders G1-3 and G2-3 while three load cycles were applied to girder G3-3. Figure 4.2 shows a typical setup for a bending test.

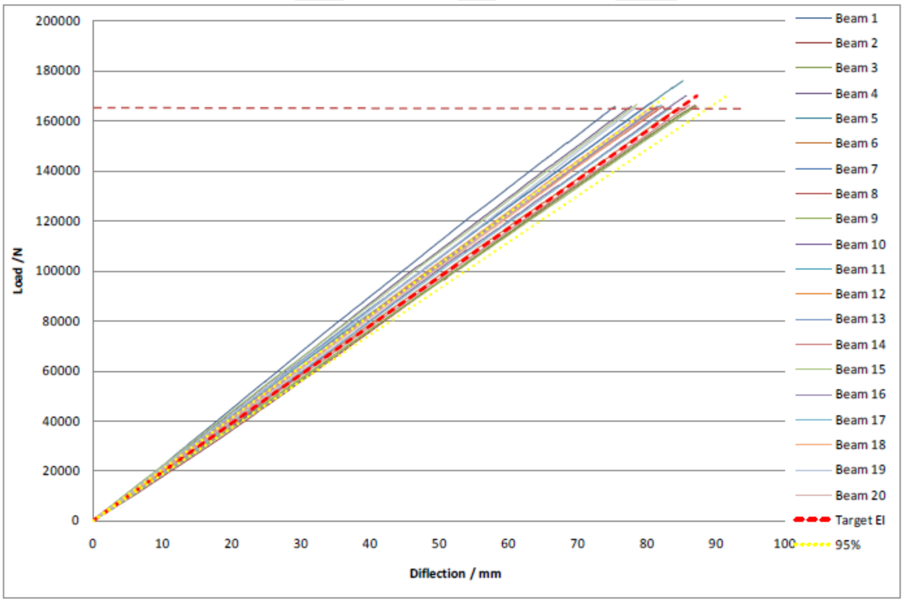
No new defects or defect propagation was observed during the bending tests.

Figure 4.2: Typical setup for bending tests

4.2.2 Load-deflection Curves

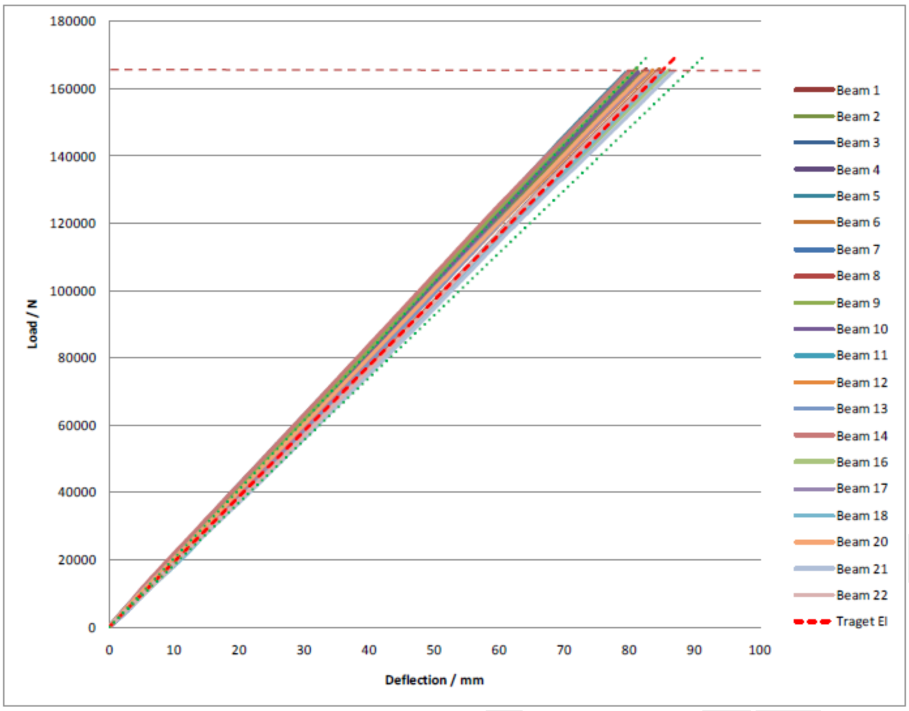
Serviceability bending testing was conducted on 20 girders of each production batch in 2010–2012 (Figure 4.11 – Figure 4.13). It was reported that all the girders behaved elastically up to the applied serviceability load of at least 165 kN. No failure in the FC materials or debonding failure was observed during testing. Summaries of the maximum applied loads and deflections are presented in Table 4.1 –Table 4.3 for stage 1 to stage 3 girders, respectively. Maximum deflections and corresponding maximum loads were also recorded.

Figure 4.3: Load-deflection curves – Stage 1 girders, 2010 bending tests



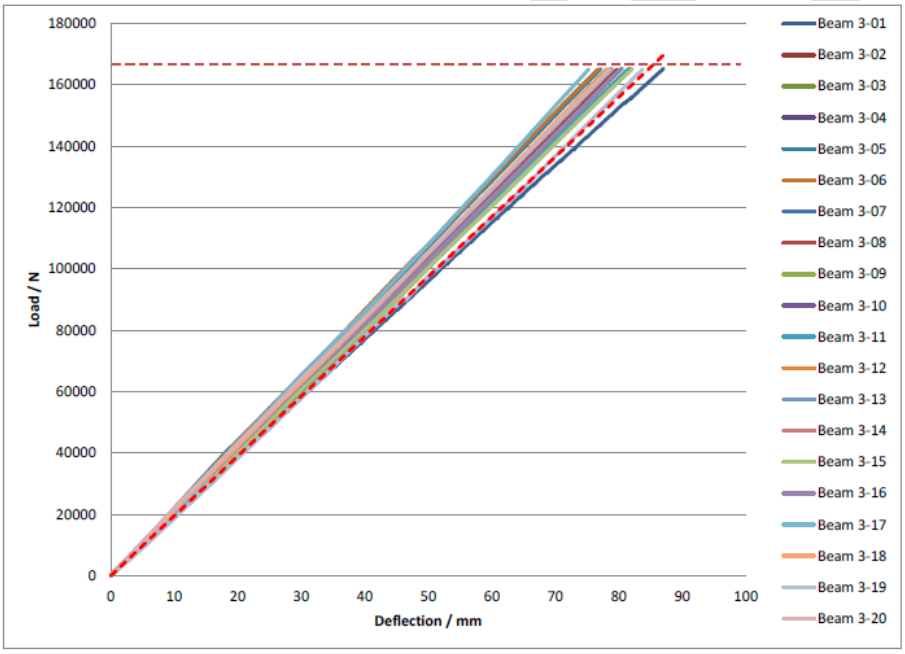
Source: Aravinthan et al. (2012a).

Figure 4.4: Load-deflection curves – Stage 2 girders, 2011 bending tests



Source: Kahandawa et al. (2011).

Figure 4.5: Load-deflection curves – Stage 3 girders, 2012 bending tests



Source: Aravinthan et al. (2012b).

Table 4.1: Maximum deflections – Stage 1 girders, 2010 bending tests

Bend Test				
WCFT Beam Test Summary				
beam #	max load kN	deflection mm	EI	Variation from target EI, %
WCFT01	165.74	75.42	3.3378E+13	12.76
WCFT02	166.05	87.04	2.8975E+13	-2.11
WCFT03	165.52	86.82	2.8955E+13	-2.18
WCFT04	165.60	77.82	3.2318E+13	9.18
WCFT05	176.15	85.26	3.1377E+13	6.00
WCFT06	165.33	84.54	2.9700E+13	0.34
WCFT07*	167.55	80.78	3.1500E+13	6.42
WCFT08	165.47	82.41	3.0494E+13	3.02
WCFT09	165.49	87.18	2.8831E+13	-2.60
WCFT 10	174.71	88.10	3.0117E+13	1.75
WCFT 11	165.69	81.69	3.0803E+13	4.06
WCFT 12	165.58	80.91	3.1079E+13	5.00
WCFT 13	166.24	82.04	3.0775E+13	3.97
WCFT 14	166.27	86.09	2.933E+13	-0.91
WCFT 15	166.60	78.65	3.2172E+13	8.69
WCFT 16	165.55	81.42	3.0881E+13	4.33
WCFT 17	165.13	83.45	3.0052E+13	1.53
WCFT 18	165.19	81.74	3.0692E+13	3.69
WCFT 19	165.13	78.37	3.2000E+13	8.11
WCFT 20	165.83	80.75	3.1190E+13	5.37
		average EI	**3.0731E+13**	
		target EI	**2.9600E+13**	
		diff	**3.82%**	
		Median	**3.0789E+13**	
		Variance	**1.4916E+24**	
		Z	**-1.96**	
		95%	**2.8395E+13**	

Source: Aravinthan et al. (2012a).

Table 4.2: Maximum deflections – Stage 2 girders, 2011 bending tests

Bend Test				
WCFT Beam Test Summary (Stage 2)				
beam #	max load kN	defln mm	EI	Variation from Target EI, %
WCFT01	165.9418694	82.4523	3.05661E+13	3.26
WCFT02	165.1711989	82.2718	3.04909E+13	3.01
WCFT03	165.0611032	84.1129	2.98036E+13	0.69
WCFT04	165.4739623	81.2129	3.0945E+13	4.54
WCFT05	165.0060553	79.5162	3.1516E+13	6.47
WCFT06	165.8317736	84.5942	2.97724E+13	0.58
WCFT07	165.0060553	82.6448	3.03229E+13	2.44
WCFT08	165.4739623	81.0203	3.10186E+13	4.79
WCFT09	166.1895849	81.2249	3.10743E+13	4.98
WCFT10	165.0060553	81.4054	3.07846E+13	4.00
WCFT11	165.0611032	82.8253	3.02669E+13	2.25
WCFT12	165.5290102	83.4631	3.01208E+13	1.76
WCFT13	165.143675	82.8374	3.02776E+13	2.29
WCFT14	165.3088187	80.0336	3.13697E+13	5.98
WCFT15				
WCFT16	165.5565342	86.0863	2.92078E+13	-1.33
WCFT17	165.1161511	82.8253	3.0277E+13	2.29
WCFT18	165.2262468	85.232	2.94417E+13	-0.53
WCFT19				
WCFT20	165.2812947	82.7411	3.03381E+13	2.49
WCFT21	165.1987229	86.7482	2.89223E+13	-2.29
WCFT22	165.0886271	84.3295	2.9732E+13	0.45
		average EI	**3.03124E+13**	
		target EI	**2.96E+13**	
		diff	**2.41%**	

Source: Kahandawa et al. (2011).

Table 4.3: Maximum deflections – Stage 3 girders, 2012 bending tests

Beam #	Max load kN	Defln mm	EI	Variation from target EI, %
3-01	165.32	87.07	2.88E+13	-2.58%
3-02	165.03	79.77	3.14E+13	6.15%
3-03	165.11	78.08	3.21E+13	8.50%
3-04	165.31	81.69	3.07E+13	3.83%
3-05	165.23	77.16	3.25E+13	9.87%
3-06	165.06	76.72	3.27E+13	10.39%
3-07	165.28	80.56	3.12E+13	5.27%
3-08	165.14	78.69	3.19E+13	7.68%
3-09	165.14	81.98	3.06E+13	3.36%
3-10	165.23	78.78	3.19E+13	7.61%
3-11	165.03	81.50	3.08E+13	3.90%
3-12	165.03	78.97	3.17E+13	7.22%
3-13	165.31	81.17	3.09E+13	4.50%
3-14	165.23	80.35	3.12E+13	5.51%
3-15	165.17	82.13	3.05E+13	3.19%
3-16	165.03	85.36	2.94E+13	-0.80%
3-17	165.06	75.30	3.33E+13	12.47%
3-18	165.09	78.34	3.20E+13	8.13%
3-19	165.01	83.78	2.99E+13	1.06%
3-20	165.06	78.72	3.18E+13	7.58%
		average EI	**3.13E+13**	
		target EI	**2.96E+13**	
		diff	**5.64%**	
		Std.Dev	**1.11E+12**	
		COV	3.53%	

Source: Aravinthan et al. (2012b).

The load-deflection curves based on the bending tests conducted in 2017 on the selected girders described in Section 4.1 are plotted in Figure 4.6 to Figure 4.8. It can be seen that the load-deflection coincides with a relatively straight line, indicating that all the girders behaved in a linear elastic manner. The deflection of girders G1-3, G2-3 and G3-3 at a load of 165 kN were 82.24 mm, 83.91 mm and 81.65 mm, respectively. These deflections were within the deflection ranges derived from the past tests of the corresponding girders of the same production stage.

It should be noted that the ultimate bending test conducted in 2012 reported an ultimate load of 400 kN and a deflection at failure of 213 mm (refer to Figure 7.2)

Figure 4.6: Load-deflection curves – girder G1-3, 2017 bending test

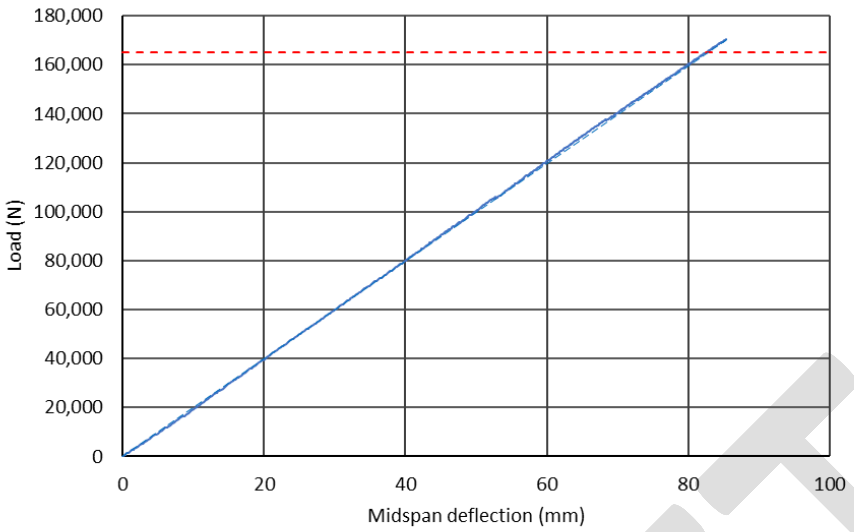


Figure 4.7: Load-deflection curves – girder G2-3, 2017 bending test

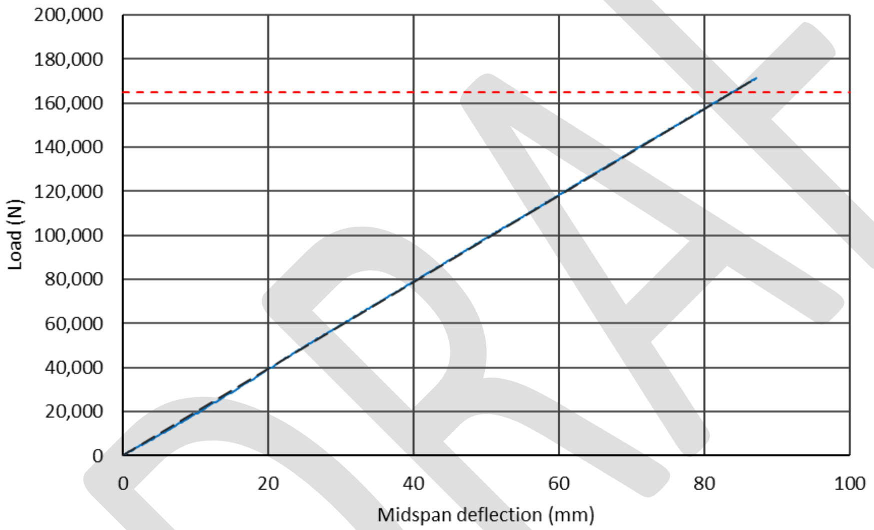
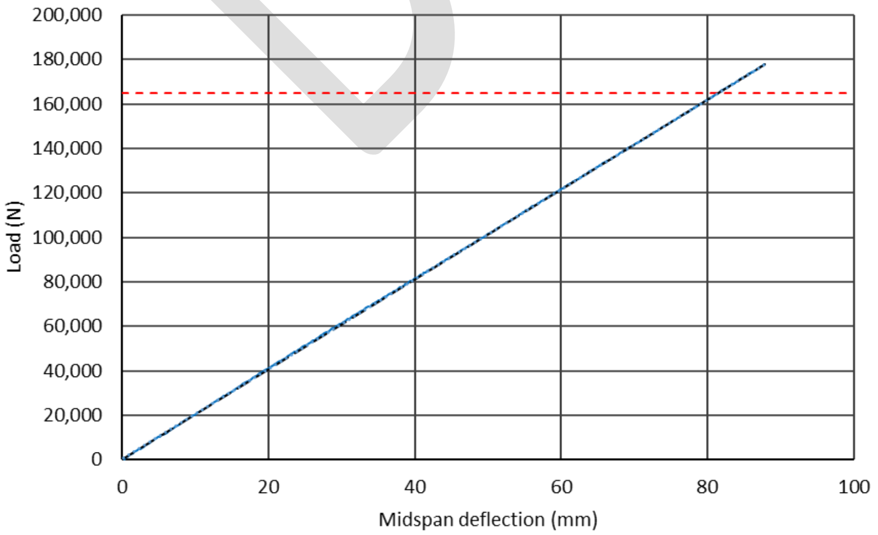


Figure 4.8: Load-deflection curves – girder G3-3, 2017 bending test



4.2.3 Bending Stiffness

The *EI* value was calculated based on the simply supported girder test shown in Figure 4.1 using the relationship shown in Equation 1.

$$EI = \frac{L^3}{48}\left[\frac{\Delta P}{\Delta v}\right] \qquad (1)$$

where

EI = bending stiffness of the girder (Nmm^2)

$\frac{\Delta P}{\Delta v}$ = slope of the load-deflection curve

L = span length (mm)

Table 4.4 compares the bending stiffness obtained in the current testing program with the test data obtained in the earlier testing. It shows that the variation in the bending stiffness after five to seven years was not statistically significant (a maximum 4.4% reduction in girder G3-3), noting that these girders had been stored in a stockpile where no load other than self-weight had been applied. In addition, the marginal changes might have included measurement errors.

Table 4.4: Changes in bending stiffness of test girders

	Max load (kN)	Deflection (mm)	Bending stiffness, EI (Nmm^2)
Girder G1-3			
2010 test	165.52	86.82	2.8955E+13
2017 test	165.04	82.24	3.0481E+13
Changes in stiffness (%)			5.3
Girder G2-3			
2011 test	165.06	84.11	2.9804E+13
2017 test	165.16	83.91	2.9893E+13
Changes in stiffness (%)			0.3
Girder G3-3			
2012 test	165.11	78.08	3.2100E+13
2017 test	165.04	81.56	3.0734E+13
Changes in stiffness (%)			–4.4

For FRP girders to be used as timber girder replacement on timber bridges, the target bending stiffness, EI, was 2.96 x 10^{13} Nmm^2 (TMR Standard drawing SD2285). Table 4.5 shows that all girders tested in 2017 satisfied the TMR stiffness requirements.

Table 4.5: Tested vs required performance criteria

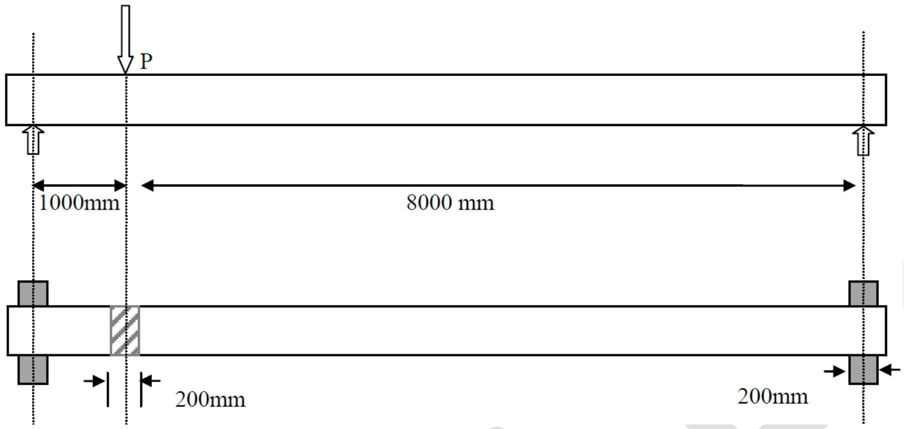
Criteria	Required	Girder 1-3	Girder 2-3	Girder 3-3
Bending stiffness, EI, Nmm^2	$2.96\text{x}10^3$	$3.05\text{x}10^{13}$	$2.99\text{x}10^{13}$	$3.07\text{x}10^{13}$
Deflection at M = 330 kNm, mm	–	72.52	72.76	72.85

4.3 Serviceability Shear Test

4.3.1 Test Setup

The test loading was applied at a distance of 1.0 m from one of the supports as illustrated in Figure 4.9 and Figure 4.10. A string pot was used to measure the deflections under the loading point. The girder was loaded to 175 kN and then unloaded. Three load cycles were applied to girders G1-3 and G2-3 while two load cycles were applied to girder G3-3.

Figure 4.9: Shear test setup



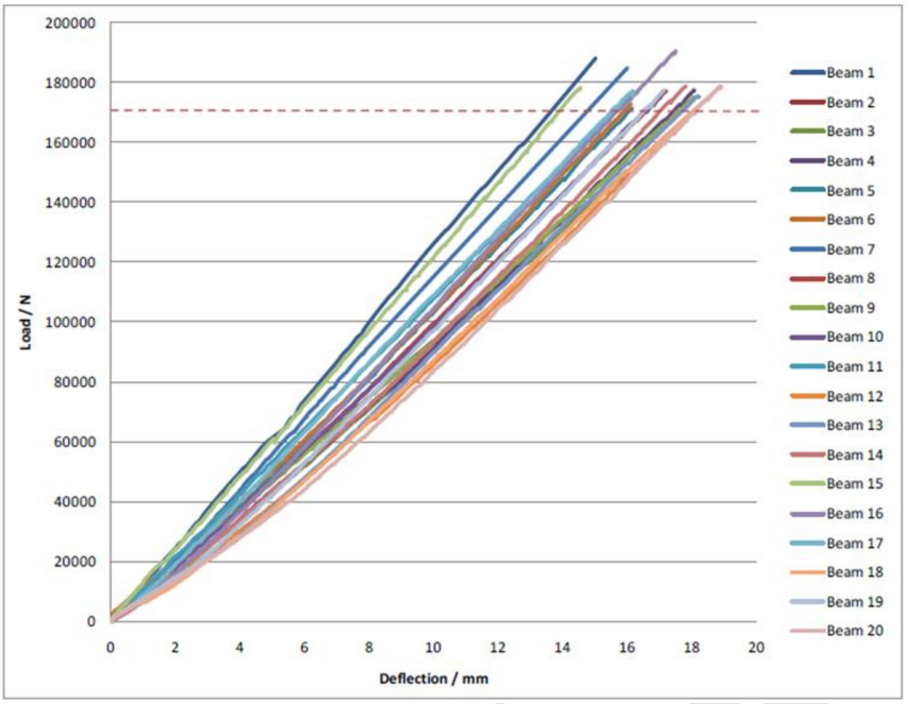
Source: Aravinthan et al. (2012b).

Figure 4.10: Typical setup for shear testing

4.3.2 Load-deflection Curves

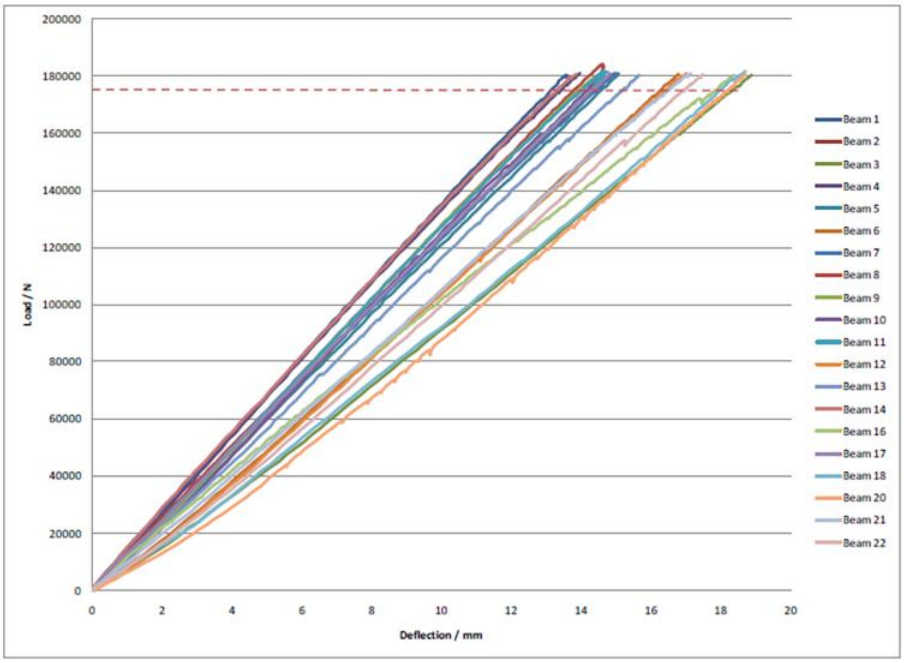
Serviceability shear tests had been conducted on 20 girders of each production batch in 2010–12 and the results are shown in Figure 4.11 to Figure 4.13. It was reported that all girders behaved similarly under the applied load of approximately 175 kN. No failure in the FC materials or debonding was observed during the testing. A summary of the maximum applied loads and deflections are presented in Table 4.6 to Table 4.8 for stage 1 to stage 3 girders, respectively.

Figure 4.11: Load-deflection curves – Stage 1 girders, 2010 shear tests



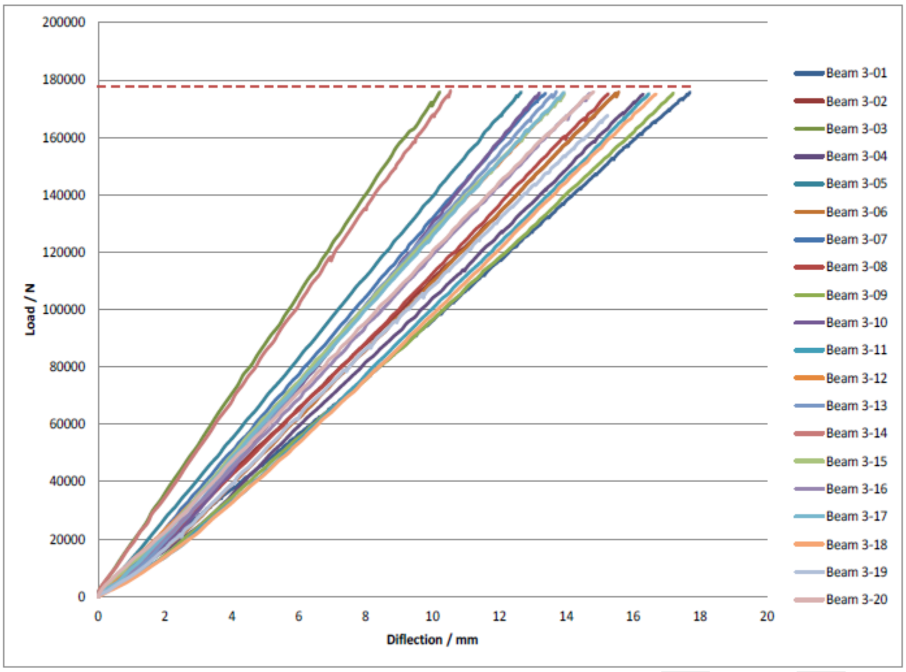
Source: Aravinthan et al. (2012a).

Figure 4.12: Load-deflection curves – Stage 2 girders, 2011 shear tests



Source: Kahandawa et al. (2011).

Figure 4.13: Load-deflection curves – Stage 3 girders, 2012 shear tests



Source: Aravinthan et al. (2012b).

Table 4.6: Maximum deflections – Stage 1 girders, 2010 shear tests

Shear Test		
CFT Beam Test Summary		
beam #	max load kN	deflection mm
WCFT01	174.71	14.33
WCFT02	175.82	19.67
WCFT03	178.62	19.09
WCFT04	177.57	18.05
WCFT05	176.96	16.69
WCFT06	177.21	16.49
WCFT07*	179.84	15.51
WCFT08	177.37	17.25
WCFT09	179.70	18.54
WCFT 10	175.29	17.06
WCFT 11	179.29	16.49
WCFT 12	177.34	18.71
WCFT 13	175.62	18.27
WCFT 14	178.82	17.82
WCFT 15	178.23	14.54
WCFT 16	190.53	17.48
WCFT 17	177.01	16.18
WCFT 18	175.60	18.60
WCFT 19	178.43	17.21
WCFT 20	178.84	18.90
Average	**178.14**	**17.34**

Source: Aravinthan et al. (2012a).

Table 4.7: Maximum deflections – Stage 2 girders, 2011 shear tests

Shear Test		
WCFT Beam Test Summary (Stage 2)		
beam #	max load kN	defln mm
WCFT01	180.14	13.60
WCFT02	180.89	14.92
WCFT03	180.53	18.87
WCFT04	180.97	13.95
WCFT05	180.91	15.07
WCFT06	180.39	16.79
WCFT07	180.03	15.01
WCFT08	180.28	14.50
WCFT09	180.12	14.43
WCFT10	180.42	14.64
WCFT11	181.63	14.75
WCFT12	180.97	16.99
WCFT13	180.36	15.64
WCFT14	180.72	13.85
WCFT15		
WCFT16	180.36	18.34
WCFT17	180.97	14.83
WCFT18	181.52	18.69
WCFT19		
WCFT20	180.78	18.68
WCFT21	180.94	17.14
WCFT22	180.75	17.48
	180.67	**15.91**

Source: Kahandawa et al. (2011).

Table 4.8: Maximum deflections – Stage 3 girders, 2012 shear tests

Shear Test		
WCFT Beam Test Summary		
beam #	max load kN	defln mm
3-01	175.78	17.70
3-02	175.36	15.47
3-03	175.82	10.22
3-04	175.02	16.31
3-05	175.82	12.66
3-06	175.88	15.57
3-07	175.38	13.38
3-08	175.19	15.26
3-09	175.44	17.21
3-10	175.52	13.20
3-11	175.22	16.47
3-12	175.24	13.91
3-13	176.02	13.71
3-14	176.29	10.54
3-15	175.13	13.95
3-16	175.38	14.70
3-17	175.74	13.93
3-18	175.05	16.69
3-19	167.48	15.22
3-20	175.82	14.83
	175.13	**14.55**

Source: Aravinthan et al. (2012b).

The load-deflection curves, based on the shear tests conducted in 2017 on the same girders described in Section 4.1, are shown in Figure 4.14 to Figure 4.16. The deflection data was recorded under the load, i.e. 1.0 m from the support. It can be seen that, in all cases, the load-deflection exhibits a non-linear manner at the low load levels but returns to linear at a higher load level. This might be due to the effects of the seating.

Figure 4.14: Load-deflection curves – girder G1-3, 2017 shear test

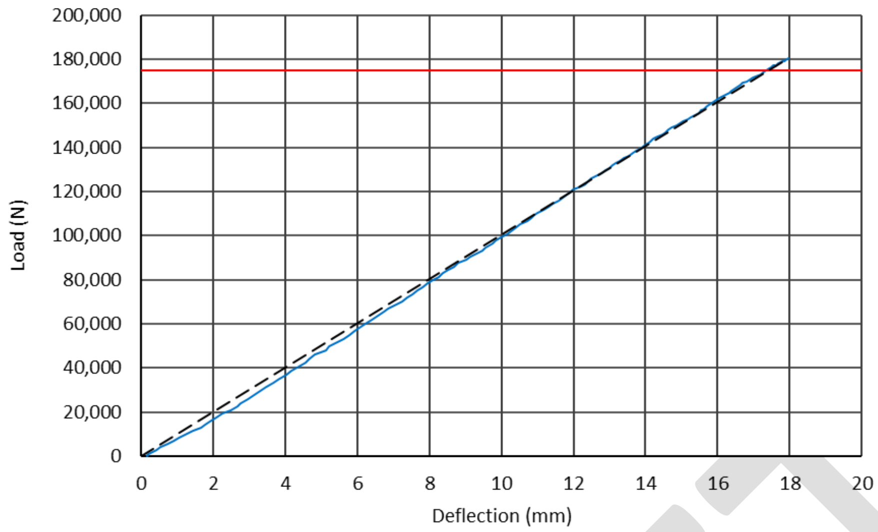


Figure 4.15: Load-deflection curves – girder G2-3, 2017 shear test

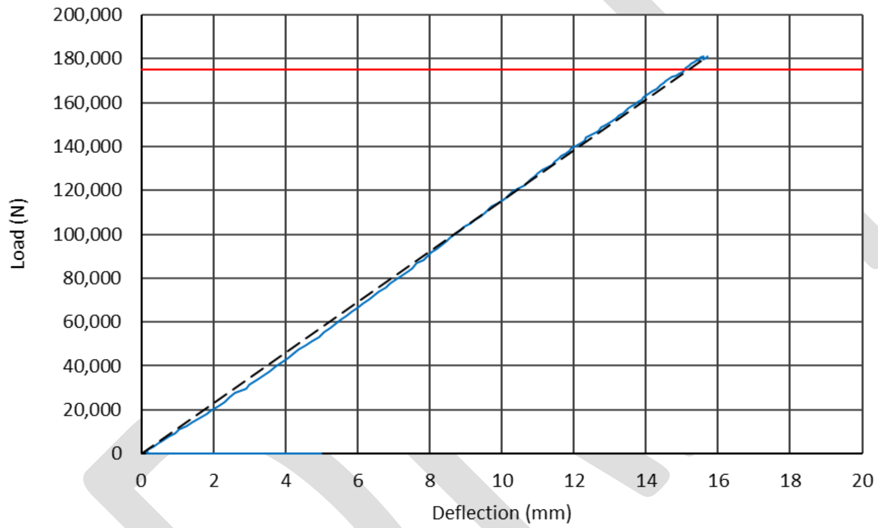
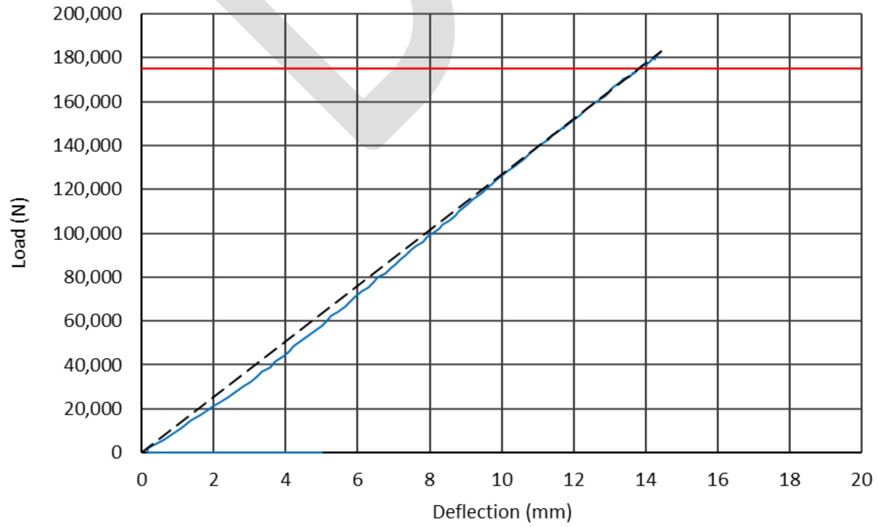


Figure 4.16: Load-deflection curves – girder G3-3, 2017 shear test



4.3.3 *Maximum Deflections*

Table 4.9 lists the current test results in comparison with the past results. While the deflections of girders G1-3 and G2-3 reduced under approximately the same loads, the deflection of girder G3-3 increased. This discrepancy might be due to isolated measurement errors associated with individual tests such as the test setup and how the load was applied.

Table 4.9: Changes in the stiffness of test girders

	Applied load (kN)	Deflection (mm)
Batch 1		
2010 test, average	178.14	15.91
2010 test, girder G1-3	178.62	19.09
2017 test, girder G1-3	178.27	17.68
Girder G2-3		
2011 test, average	180.67	15.91
2011 test, girder G2-3	180.53	18.87
2017 test, girder G2-3	180.85	15.71
Girder G3-3		
2012 test, average	175.13	14.55
2012 test, girder G3-3	175.82	10.22
2017 test, girder G3-3	175.08	13.81

5 INSPECTION OF FRP REPLACEMENT GIRDERS

5.1 Bridge List

Table 5.1 lists the existing timber bridges with FRP replacement girders on the TMR network. The FRP components on these bridges were inspected as part of this project.

Table 5.1: Bridges with FRP girders inspected

Bridge ID	Bridge name	Location of FRP girder	Manufacturer	Girder type	Inspection date
963	Oakey Creek Bridge	Span 1, Girder 2	LOC	LOC420	20/12/2017
		Span 2, Girder 2	LOC	LOC420	
		Span 3, Girder 2	LOC	LOC420	
1020	Obi Creek No 1 Bridge	Span 2, Girder 1	LOC	LOC420	20/12/2017
210	Coulson Creek Bridge	Span 2, Girder 4	LOC	LOC400	09/11/2017
181	Todd Bridge	Span 2, Girder 1	LOC	LOC400	09/11/2017
		Span 3, Girder 1	LOC	LOC400	
		Span 5, Girder 1	Wagner	S3	
229	Heifer Creek No 7 Bridge	Span 1, Girder 1	Wagner	S1*	09/11/2017
		Span 1, Girder 5	Wagner	S1*	
219	Horse Trough Bridge	Span 1, Girder 5	Wagner	Non-standard 9 cell girders	09/11/2017
		Span 2, Girder 4	Wagner		

Notes: * These two girders were installed on the same span by the district (not part of the 'Bridging the Gap' production). Refer to the notes in Section 5.2.2.

5.2 Inspection Findings

5.2.1 Coulson Creek Bridge

Coulson Creek Bridge is located at 2141 Lake Moogerah Road on South Coast Hinterland region. Main components of the bridge comprise:

- three-span timber superstructure
- four timber girders on the cross-section supported on corbels
- original timber plank deck with asphalt wearing surface
- timber headstocks on three timber piles.

One LOC400 girder was installed in 2014 to replace the external timber girder on span 2 (Figure 5.1). Annual Level 2 inspections carried out on the bridge since 2014 indicate that the FRP girder remains in good condition with a condition rating of condition state (CS) 2.

According to the latest L2 inspection report (June 2017), the structure was in poor condition with a condition rating of CS4 due to a number of severe defects such as excessive section loss on abutment piles, timber girders, and corbels, rotted timber deck planks, and excessive snipes on timber girders.

Figure 5.1: Coulson Creek Bridge – replacement FRP girder on span 2

(a) view of the FRP girder from below

(b) view of the FRP girder from side

The following observations were made for the FRP girder:

- water stains due to leaking and water ingress from the decayed timber deck causing discoloration throughout the length of the girder – more pronounced effects were found on the external face and bottom of the girder (Figure 5.2)
- moderate corrosion on the bolt nuts (Figure 5.3)
- minor local crushing of composite surface due to physical impact at the outside face of the girder (Figure 5.4)
- a split corbel supporting pier 2 end of the FRP girder at the location of the connection bolt (Figure 5.5).

Figure 5.2: Typical water stain along FRP girder

Figure 5.3: Bolt corrosion under FRP girder

Figure 5.4: View of minor surface deformation due to impact at the RHS end of FRP girder

Figure 5.5: Split noted at corbel under end of FRP girder

In general, the FRP girder had minor defects (a 2D rating) except for the discoloration on the surface of the girder due to water ingress. Without a more intrusive test, there was no other evidence of material degradation or distress of the composite materials at the interfaces with other bridge components. The FRP girder showed no sign of excessive deformation or differential displacement. This indicates that the load-sharing mechanism is in place, and the replacement girder is performing as expected.

It should be noted that Thioflex 600 or an approved equivalent sealant is required (in SD 2280) to apply around the bolts' washer to prevent water ingress; however, it was not the case on this bridge, causing the corrosion of the bolts.

5.2.2 Heifer Creek No. 7 Bridge

Heifer Creek No. 7 Bridge is located at Heifer Creek Eighth Crossing on the Gatton – Clifton Rd in the Darling Downs region. The main components of the bridge comprise:

- three-span timber superstructure
- five timber girders on the cross-section supported on timber corbels
- a timber plank deck was replaced by plywood deck
- two wall-type concrete piers on spread footing
- each abutment has a timber headstock supported on five timber piles.

Two Type S1 WCFT girders (S1-G1 and S1-G5) were installed in 2011 to replace the external timber girders on span 1 (Figure 5.6). Annual Level 2 inspections carried out on the bridge since 2014 indicate that the FRP girder remains in CS2, and both girders have a sagging profile of 15-30 mm at mid-span.

According to the latest L2 inspection report (September 2016), the structure was in fair condition with a condition rating of CS2. For the FRP girders, the sag remains unchanged; all bolting is tight and there is no cracking in the girders; the packers are working properly.

Figure 5.6: Heifer Creek No 7 Bridge

(a) General view of bridge from RHS

(b) View of FRP girders on Span 1

The following observations were made for the FRP girders:

- Both girders sagged at the middle section, as indicated in the previous inspection reports.
- Delamination of the composite material at the bottom flange layer of S1-G1 at several locations (Figure 5.7).
- Minor chipping of the coating at the soffit of girder S1-G1 (Figure 5.8).
- Tear of a fibre strip at almost every bolt position; this is likely due to the installation of the bolts holding down the deck (Figure 5.9).
- Minor corrosion on all bolt nuts (Figure 5.9).
- Multiple areas of water stains or discoloration at the underside of the girders (Figure 5.8 and Figure 5.9).
- Bulging of the rubber pads at supports with corroded steel bearing plates (Figure 5.10). It is noted that the abutment end of the FRP girders was supported on two discrete points while the other end was supported continuously on corbels.
- Loss of bond between the angle and the flange of the girder (Figure 5.11).
- Wasp nests on several locations on the web of the girders (Figure 5.12).

It should be noted that the delamination and loss of bond are considered significant defects for the FRP girders. Particular care should be taken to monitor any propagation of these defects.

Figure 5.7: Delamination of flange plate at several locations on S1-G1

Figure 5.8: Minor chipping of the coating at the soffit on LHS on end 2 of S1-G1

Figure 5.9: Strip of fibre removed at the soffit of girder, close to a bolt connection

Figure 5.10: Bulging of rubber pads at bearings

Figure 5.11: Loss of bond between the angle and flange of the girder

Figure 5.12: Wasp netting on girder's web

In general, the FRP girders had moderate defects, with a 3B defect rating. The observed damage was due to the installation process. No evidence of further degradation of materials was noted. The magnitude of sagging remained unchanged since 2014. There was good interaction with other bridge components (no signs of distress/material degradation at the interface with the plywood deck, corbels and rubber pads).

It should be noted that two WCFT girders were installed on this bridge on a single span, in contrary to SD 2285 which was issued after a trial installation of the first WCFT girder on Todd Bridge. These two girders were not proof-tested, and the District took the initiative to install them. The girders were not part of the 'Bridging the Gap' production project.

5.2.3 Todd Bridge

Logan River Todd Bridge is located at O'Briens Crossing on the Boonah-Rathdowney Rd in the South Coast region. The main components of the bridge comprise:

- seven-span timber superstructure
- five timber girders on the cross-section supported on timber corbels
- the previous deck (corrugated steel) was replaced by plywood deck circa 2017
- piers comprise timber piles and headstocks.

One LOC400 girder was installed in late 2014 on span S2, and another of the same type on span S3 in 2017. In early 2015, one Type S3 WCFT girder was installed on span S5. All these FRP girders were installed to replace the external timber girder G1 on the corresponding span (Figure 5.13).

Annual Level 2 inspections carried out on the bridge since 2016 indicate that the FRP girders on spans S2 and S5 remain in CS2 while the LOC girder on span S3 is in CS1. According to the latest L2 inspection report (August 2017), the structure was in fair condition, with a condition rating of CS2. There was no damage to the FRP girders.

Figure 5.13: Todd Bridge – view of FRP replacement girders

The following observations were made for the LOC girders:

- minor corrosion at the bolts holding down the deck (Figure 5.14)
- minor scratches on the surface at end 1 of girder S2-G1 due to impact, likely during the re-decking process (Figure 5.15)
- water stains along the FRP girders on both inside and outside faces (Figure 5.16)
- minor corrosion of the steel packer plates under the girder supports on the corbels (Figure 5.17).

Comparing to the LOC400 stockpiled girders (Section 3.3.1), the girders on this bridge have less surface deterioration; this is likely due to lesser exposure to direct sunlight and moisture.

Figure 5.14: Corrosion of bolts at S2-G1

Figure 5.15: Minor deformation at S2-G1

Figure 5.16: Typical water stains on FRP girder

Figure 5.17: Corroded steel plate between FRP girder (S3-G1) and corbel (P2-COR1)

The WCFT type S3 girder (S5-G1) generally had a minor defect (Figure 5.18), with minor peeling off of the coating at the soffit of the girder.

Figure 5.18: WCFT FRP girder on span S5

In summary, all FRP girders on this bridge had a defect rating of 2B. Whilst there were water stains on the surface of the girders, there was no evidence of material degradation. There was a good level of interaction with other bridge components, with no signs of distress or material degradation at the interface with the plywood deck, corbels and steel packers.

5.2.4 Horse Trough Bridge

Horse Trough Bridge is located on Gatton-Clifton Road near Toowoomba. Main components of the bridge comprise:

- two-span timber superstructure
- five timber girders on the cross-section supported on timber corbels
- original deck was replaced by plywood deck
- piers comprise timber headstocks on four timber piles.

Two non-standard nine-cell WCFT girders were installed in 2015 to replace an interior timber girder on span S1 (S1-G2) and an external timber girders on span 2 (S2-G1) (Figure 5.19). Level 2 inspections carried out on the bridge in 2014 and 2015 indicated that the FRP girder remained in CS2. The structure was rated CS4 in the November 2014 L2 inspection report but was in CS2 in March 2015 after extensive rehabilitation work. There was no damage to the FRP girder.

Figure 5.19: RHS view of Horse Trough Bridge

(a) View of FRP girder on Span 1 (S1-G2)

(b) View of FRP girder on Span 2 (S2-G1)

The following observations were made for the FRP girders:

- A small burnt surface area (100 x 100 mm) due to fire damage at a location 2.5 m from the abutment on the external face of girder S2-G1, with fibres exposed and fire marks around the hole (Figure 5.20).
- A number of bolt holes were left open on the sides and at the bottom of both girders (Figure 5.21). Several holes were not well rounded with blooming fibres.
- A strip of fibre was removed at some unused bolt holes, possibly due to the hole drilling process (Figure 5.22).
- Water stains on the outer surfaces of the girders (Figure 5.23).
- Minor corrosion of steel packers located at the abutments between the FRP girder and the headstock (Figure 5.24).

Figure 5.20: View of a burning area noted at right hand side of FRP girder S1G2

Figure 5.21: Unused bolt holes on FRP girders

(a) View of a unused hole on the side face of the girder

(b) View of a unused hole at the bottom of the girder

Figure 5.22: View of fibre tear off noted at soffit of girder

Figure 5.23: View of typical water stain at the FRP girder

Figure 5.24: View of minor corrosion of steel packers

In general, all FRP girders on this bridge had moderate defects (a rating of 3B), with the small fire-damaged area and the pre-drilled holes on the sides and bottom of the girders. Whilst water stains were observed on the surface of the girders, there was no evidence of material degradation. There was a good level of interaction with other bridge components, and a good contact between FRP girder, timber packers and corbels.

5.2.5 Obi Creek No 1 Bridge

Obi Creek No 1 Bridge is located on Obi Rd in the North Coast region. Main components of the bridge comprise:

- three-span timber superstructure
- five timber girders on the cross-section supported on timber corbels
- original deck was replaced by corrugated steel deck
- piers comprise of timber headstocks on four timber piles.

One LOC420 girder was installed in 2016 to replace an external timber girder on span 2 (S2-G1) (Figure 5.25). The Level 2 inspection carried out on the bridge in August 2016 indicated that the structure was in CS3 and the FRP girder was in CS1.

Figure 5.25: View of FRP girder on Obi Ck No1 Bridge (external girder)

The following observations were made as a result of the current inspection:

- loss of composite and exposed fibres at the soffit of the FRP girder at several bolts holding down the deck (Figure 5.26), likely due to the installation process
- water stains and discoloration at various locations at the side face and at the soffit of the girder (Figure 5.27)
- minor bolt corrosion at all bolt locations (Figure 5.28)
- minor corrosion of the steel trough deck around the contact areas with the FRP girder (Figure 5.29)

In general, all FRP girders on this bridge had a defect rating of 3B, with the major damage comprising small areas of exposed fibres around the bolts and water stains on the girder's surface. There was no evidence of material degradation. There was a good level of interaction with other bridge components and good contact between the FRP girder, timber packers and corbels (Figure 5.30).

Figure 5.26: Loss of composite and exposed fibres at bolt location

Figure 5.27: Water stains along FRP girder

Figure 5.28: Typical minor corrosion of bolts

Figure 5.29: Corrosion of steel trough in contact with the FRP girder

Figure 5.30: Good contact with packers and corbel

5.2.6 Oakey Creek Bridge

Oakey Creek Bridge is located on Eumundi-Kenilworth Rd in the North Coast region. Main components of the bridge comprise:

- four-span timber superstructure
- five timber girders on the cross-section supported on timber corbels
- original deck was replaced by corrugated steel deck
- piers and abutments comprise of timber headstocks on four timber piles.

Three LOC420 girders was installed in 2016 to replace the timber girder (G2) on three spans S1–S3 (Figure 5.31). The structure was in CS4 in the September 2016 L2 inspection report but was in CS2 in February 2017 after extensive rehabilitation work. Double timber girders were installed as the edge girders. No information about the FRP girder was reported.

Figure 5.31: FRP girders on Oakey Creek Bridge

(a) FRP girder on span S1 (b) FRP girder on span S3

The following observations were made for the FRP girders:

- rust-like water stains and discoloration at multiple areas at the soffit of the FRP girders (Figure 5.32); the surface of the girders was very damp, with fresh water marks
- evidence that the girders were submerged during a recent flood with wave shape stains and debris noted at both sides of the FRP girders (Figure 5.33)
- loss of composite and exposed fibres due to impact at several locations at the soffit of the girders (Figure 5.34).

In general, all the FRP girders on this bridge had a defect rating of 3C. Rust-like water stains at the soffit of the girders require further monitoring and investigation. There was a good level of interaction with other bridge components is present and good contact between the FRP girder, timber packers and corbels.

Figure 5.32: Rust-like water stains at the soffit of S1-G2

Figure 5.33: View of typical wave shape water stains noted at both faces of FRP girders

Figure 5.34: Loss of composite and exposed fibres on S3-G2

5.3 Defect Rating of the FRP Component

In general, the FRP girders on existing timber bridges had minor to moderate defects according to the criteria proposed in Section 2.5. This rating was made based on the severity and extent of the defects. Table 5.2 summarises the findings for each girder and its defect rating. An overall condition state of each bridge based on the latest L2 inspection report is also included in this Table for reference.

Table 5.2: Defect rating of FRP girders installed on TMR timber bridges

Bridge name/ID	Bridge overall condition state	Girder location	Girder type	Girder defect rating	Typical defects/deterioration
Coulson Creek Bridge BIS 210	CS2	S2-G4	LOC400	2D	▪ Water stains due to leaking and water ingress causing discoloration throughout the length of the girder. ▪ Moderate corrosion of the bolts holding down the deck. ▪ Minor local crushing of composite due to impact. ▪ A corbel supporting pier 2 end of the FRP girder was split up.
Heifer Creek No. 7 Bridge BIS 229	CS2	S1-G1 & S1-G5	WCFT S1	3B	▪ Both girders were sagged at the middle section. ▪ Delamination of the composite material. ▪ Minor chipping of the coating. ▪ Year of a fibre strips at bolt locations. ▪ Loss of bond was found between the angle and the flange of the girder. ▪ Water stains or discoloration at the underside of the girders. ▪ Minor corrosion on all bolt nuts. ▪ Bulging of the rubber pads at supports with corroded steel bearing plates.
Todd Bridge BIS 181	CS2	S2-G1	LOC400	2B	▪ Minor bolt corrosion. ▪ Minor water stains along the girder. ▪ Minor damage due to impact.
		S3-G1	LOC400	2B	▪ Minor water stain along the girder.
		S5-G1	WCFT S3	2B	▪ Minor peeling of the coating at the soffit of girder.

Bridge name/ID	Bridge overall condition state	Girder location	Girder type	Girder defect rating	Typical defects/deterioration
Horse Trough Bridge BIS 219	CS2	S1-G2	WCFT Non-standard	3B	▪ various unused holes at soffit and on side faces. ▪ Minor water stains along the girder. ▪ tear of a fibre strips at bolt locations.
		S2-G1	WCFT Non-standard	3B	▪ Minor fire damage area. ▪ Various unused holes at soffit and on side faces. ▪ Minor water stains along the girder. ▪ Tear of a fibre strips at bolt locations.
Obi Creek No 1 Bridge BIS 1020	CS3	S1-G1	LOC420	3B	▪ Loss of composite and exposed fibres at the soffit of the girder at several bolt locations. ▪ Water stains and discoloration at various locations at the side face and soffit of the girder. ▪ Minor bolt corrosion. ▪ Minor corrosion of the steel trough deck around the contact areas with the FRP girder.
Oakey Creek Bridge BIS 963	CS2	S1-G2, S2-G2 S3-G2	LOC420	3C	▪ Rust-like water stains and discoloration at multiple areas at the soffit of the FRP girders. Possible water penetration into the material. ▪ Water stains and debris at both sides of the FRP girders. ▪ Loss of composite and exposed fibres due to impact at several locations at the soffit of the girders.

6 INSPECTION OF COMPLETE FRP BRIDGES

6.1 Bridge List

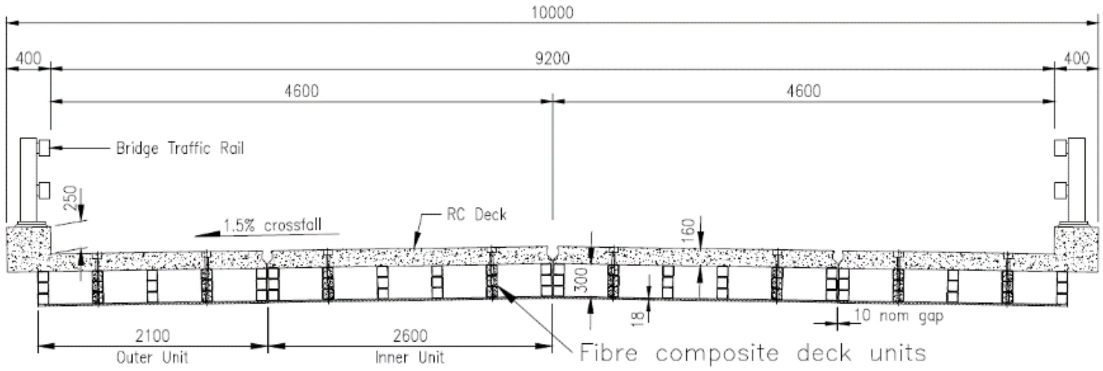
Two complete FRP bridges are on the TMR network, including the Taromeo Creek Bridge (road bridge), and Sandy Creek Bridge (pedestrian bridge). These bridges were inspected on 20/12/2017.

6.2 Inspection Findings

6.2.1 Taromeo Creek Bridge

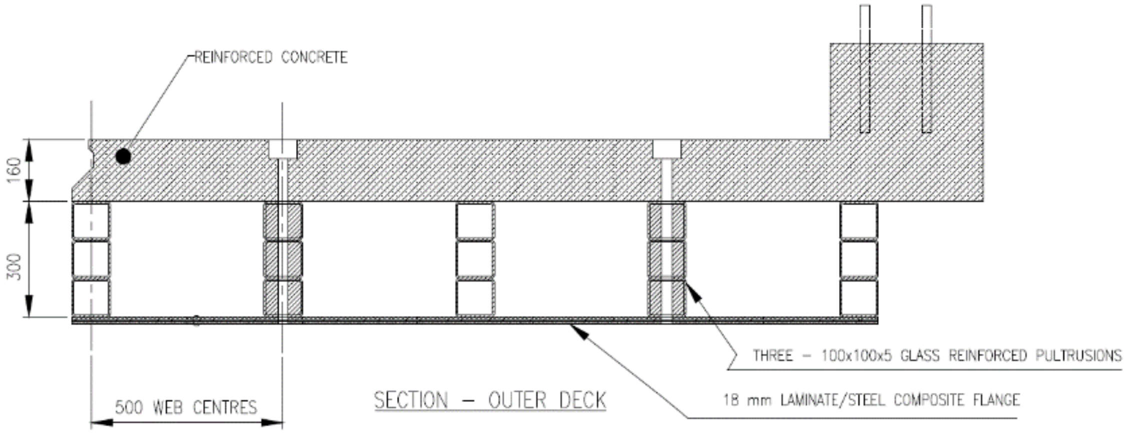
The Taromeo Creek Bridge is located on the D'Aguilar Highway in Blackbutt. It consists of one 10 m span and one 12 m span. It was the first FRP road bridge superstructure built in Queensland in 2005 (by WCFT). The FRP superstructure replaced an existing timber bridge. The design of this superstructure was jointly developed by WCFT and TMR. The cross-section of the deck comprises a standard reinforced concrete deck placed on glass pultruded webs and a FC/steel tensile flange (Figure 6.1 and Figure 6.2).

Figure 6.1: Taromeo Creek bridge – cross-section



Source: McCormick (2006).

Figure 6.2: Taromeo Creek bridge – cross-section of one external unit



Source: McCormick (2006).

A number of unusual construction details were found in the structure, including:

- Different coatings appear to have been used at the soffit of different units, resulting in different surface finishes (Figure 6.3).

- The gaps between units were filled with high-strength grout. Five transverse pads were glued to the underside, bridging the gap between units 3 and 4 on span 1 and between units 1 and 2 on span 2 (Figure 6.4).
- A smaller FRP girder was installed on the left-hand side of the structure, possibly to prevent logs from hitting the FRP units during flooding (Figure 6.5). This is different from the original design shown in Figure 6.1. This girder was hung on the concrete slab underneath the kerb and is not supported on the pier.
- A sheet was installed at the pier end of the right-hand side unit on span 2 (Figure 6.6), possibly for level adjustment. The remaining units sit directly on concrete on the pier and abutments.

The latest L2 inspection report (in 2013) rated the bridge in CS2. Unfortunately, details of the condition of the FRP components were not available for comparison.

Figure 6.3: Different finished surfaces of units

Figure 6.4: Transverse pads at soffit of bridge

Figure 6.5: Single FRP girder under the kerb

Figure 6.6: FRP sheet used as a packer on the headstock

The following main defects were found on the FRP deck units.

- Differential deflections at mid-span between units 2 and 3 at the underside of the units (Figure 6.7). These differential deflections were opposite in direction between span 1 (the deflection in unit 2 was about 16 mm lower than unit 3) and span 2 (the deflection in unit 3 was 25 mm lower than unit 2). The epoxy grout had completely failed at these locations.
- Exposed fibre and localised damage at the soffit of unit 2 and 3 at various areas (Figure 6.8)
- Damage to the fibre on a small area due to physical impact on the external FRP girder under the kerb unit (Figure 6.9).
- Deterioration of the coating at various locations around the mid-span at the soffit of unit 4 on span 1 (Figure 6.10).
- Water stains at the abutment headstocks (Figure 6.11) There was a significant water leak on the left-hand side of the pier headstock (Figure 6.12).

Figure 6.7: Differential profile at soffit of units 2 and 3 at mid-span

Figure 6.8: Fibre tear and exposed at underside of unit 3 on span 2

Figure 6.9: Damage on external FRP girder on Span 2

Figure 6.10: Defects on the coating on the soffit of unit 1

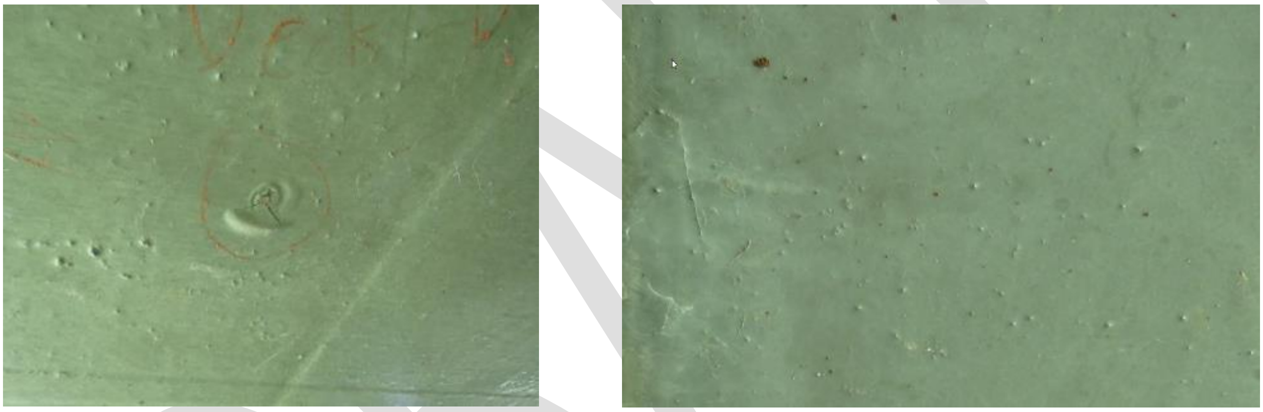


Figure 6.11: Typical water stains on abutment headstocks

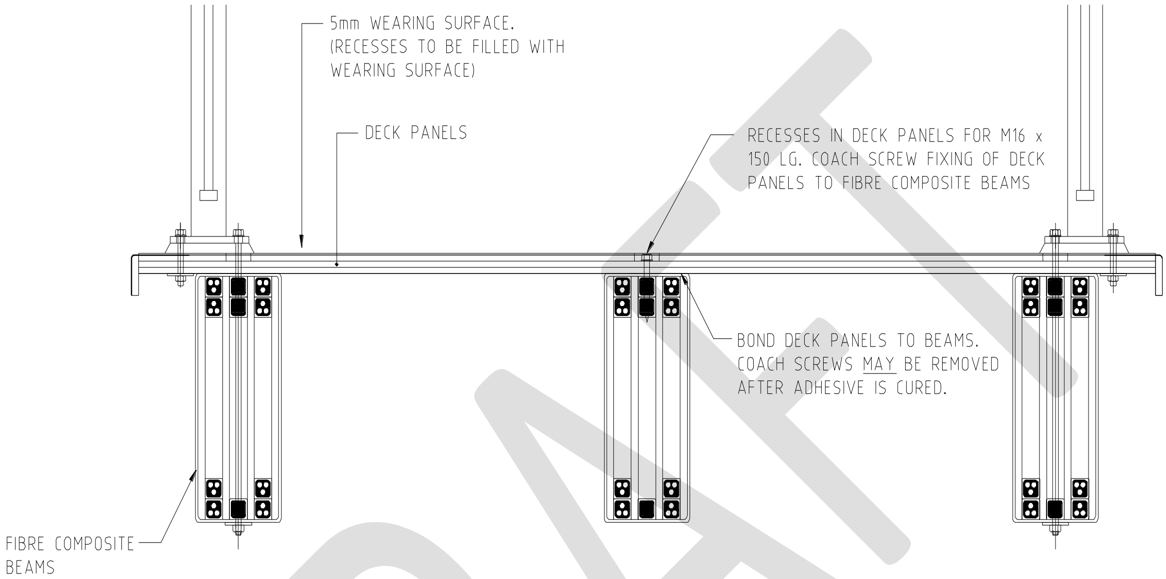
Figure 6.12: Water leak on pier headstock

6.2.2 *Sandy Creek Pedestrian Bridge*

The first entire FRP superstructure built in Queensland was Sandy Creek bridge (2003) – a 2.5 m wide, 14 m single-span pedestrian bridge. It comprises a 10 mm thick polymer concrete wearing surface on a deck formed from two layers of 25 mm thick LOC panels supported on three FC girders (Figure 6.13). The girder cross-section was constructed from a number of LOC panels (manufactured by LOC Composites Pty Ltd), hybrid reinforcement modules, PFR and GWR (glass fibre woven roving) laminates (Figure 6.14).

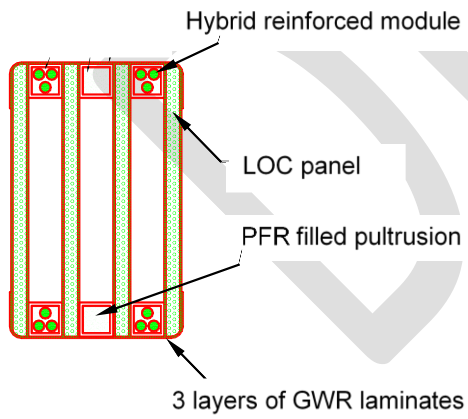
The LOC panels were manufactured to form a sandwich construction consisting of two layers of triaxial glass in each skin and the PFR – an epoxy resin. The triaxial glass layers comprise glass fibres distributed in the 0° and ±45° directions. The panels were used to form the webs of the girders and the deck panels. They carry the majority of the shear in the girders and bending moments in the deck.

Figure 6.13: Sandy Creek pedestrian bridge –cross-section



Source: Shaw and Van Erp (2009).

Figure 6.14: Sandy Creek pedestrian bridge – girder cross-section



Source: Shaw & Van Erp (2009).

Figure 6.15: Left-hand side view of Sandy Pedestrian Bridge

Figure 6.16: View from approach 1

The latest L2 inspection report (in 2016) rated the bridge in CS1. Unfortunately, details of the condition of the FRP components were not available for comparison.

The following main defects were found on the FRP component and in its surround components:

- Multiple vertical minor cracks (up to 0.15 mm wide) on the external side faces of girder 1 and girder 3, typically located at the areas affected by moisture due to water leak from the deck (Figure 6.17) Noting that there was no cracking at the internal side faces of the girders, even at the areas with water stains (Figure 6.18), this cracking may indicate material degradation due to UV since the external faces are exposed to direct sunlight, coupled with moisture effects.
- Moisture could have penetrated the material through the cracks.
- Various spalling areas with exposed fibre along the lower edges of girder 3 (Figure 6.19); due to wear of the coating as a result of a form of material degradation. There was a loss of material and discoloration around these locations.
- Water stains along all of the girders due to water leaking from the bridge deck, causing discoloration (Figure 6.20 and Figure 6.21).
- Excessive epoxy adhesive at the underside of the girders (Figure 6.22).

Figure 6.17: Vertical cracks at various areas on external side face of G1 and G3

On girder G1

On girder G3

Figure 6.18: No cracking observed at water leak areas on internal faces of girders

Figure 6.19: Typical spall with exposed fibre along lower edge of girder G3

Figure 6.20: Water stains at external face of girder G1

Figure 6.21: Water stains at an internal face of girders

Figure 6.22: Excessive epoxy adhesive at soffit of all girders

Apart from these defects, all connection details such as steel plates, angles and bolts were in good condition (Figure 6.23).

Figure 6.23: Connection details

6.3 Defect Rating of the FRP Bridges

Table 6.1 summarises the findings for each girder and its defect rating based on the criteria proposed in Section 2.5.

Table 6.1: Defect rating of components on FRP bridges

Bridge name/ID	Girder location	Girder type	Defect rating	Typical damage
Tarومeo Creek Bridge (BIS 34679)	S1D1	Hollow deck	3C	▪ Differential deflections between units 2 and 3 at mid-span (16 mm). ▪ Exposed fibre and localised damages at various areas at the soffit of units 2 and 3. ▪ Deterioration of the coating at various locations around the midspan at the soffit of unit 4.
	S2D1	Hollow deck	3C	▪ Differential deflections between units 2 and 3 at mid-span (16 mm). ▪ Exposed fibre and localised damages at various areas at the soffit of units 2 and 3. ▪ Deterioration of the coating at various locations around the midspan at the soffit of unit 4. ▪ Damaged fibre on a small area due to physical impact.
Sandy Creek Pedestrian Bridge (BIS 43374)	S1D1	FRP girder	1A	▪ No defects noted
	S1G1	LOC	2D	▪ Multiple vertical minor cracks (up to 0.15 mm wide) along the external side face of girder. Possible water penetration into material. ▪ Water stains along the girder. ▪ Excessive epoxy adhesive at the underside of the girders.
	S1G2	LOC	2C	▪ Water stains along the component. ▪ Excessive epoxy adhesive at the underside of the girders.
	S1G3	LOC	2D	▪ Multiple vertical minor cracks (up to 0.15 mm wide) along the external side face of girder. Possible water penetration into material. ▪ Water stains along the girder. ▪ Excessive epoxy adhesive at the underside of the girders. ▪ Minor spalling with exposed fibre and material discoloration at various locations along the lower edges of girder.

7 DISCUSSION

7.1 Performance of Existing FRP Components/Structures

The performance of the FRP components can be evaluated using the element defect rating system as presented in Section 2.5. The rating can be converted into the element condition scores (ECS) as shown in Table 7.1. The ECS ranged between 1.0 and 5.0, which is equivalent to five bands (Very Good, Good, Fair, Poor and Very Poor). With the exception of the first band (1.0), the remaining four bands can be related to four condition states (CS1–CS4) used in the current TMR condition rating system.

Table 7.1: Element condition scores

<table>
<tr><th rowspan="2">Extent</th><th colspan="5">Severity</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td>A</td><td>1.0</td><td></td><td></td><td></td><td></td></tr>
<tr><td>B</td><td></td><td>2.0</td><td>3.0</td><td>4.0</td><td rowspan="4">5.0</td></tr>
<tr><td>C</td><td></td><td>2.1</td><td>3.1</td><td>4.1</td></tr>
<tr><td>D</td><td></td><td>2.3</td><td>3.3</td><td>4.3</td></tr>
<tr><td>E</td><td></td><td>2.7</td><td>3.7</td><td>4.7</td></tr>
</table>

Note: shaded boxes represent non-permissible Severity/Extent combinations.
Source: Atkins (2007).

The scoring in this Table reflects the view that the extent of damage is less critical than the severity of damage. This system allows the asset owner to monitor change in the condition of individual component over time.

As observed from the site inspection for all inspected structures, in general the FRP replacement girders on existing timber bridges were in good to fair condition with defect ratings of 2B–3C (Table 5.2), or ECS = 2.0–3.1 (see Table 7.1). The most common deterioration issues include:

- water stains and discoloration of material due to moisture ingress, minor loss of material, fibre exposure and damage due to physical impacts
- peeling off of coating or wear
- tear of fibre strips on LOC girders due to installation process
- rust-like stains (on Oaky Creek Bridge) with potential water penetration into the material
- sagging of girders (on Heifer Creek No. 7 Bridge).

The main root causes of these deterioration issues are related to physical and environmental impacts, including:

- physical damage during installation
- water leakage
- moisture and direct sunlight.

The following general comments can be made with respect to the performance of the FRP replacement components on each bridge:

- Coulson Creek Bridge – the FRP girder showed no sign of excessive deformation or differential displacement. This indicates that the load-sharing mechanism is in place, and the replacement girder is performing as expected.

- Heifer Creek No. 7 Bridge – there were moderate defects in the FRP girders. The damage observed was likely related to the installation process. There was no evidence of further degradation of the materials. No long-term effects on stiffness were observed (as the sagging had remained unchanged since 2014). A good level of interaction with other bridge components was evident as there were no signs of distress/material degradation at the interface with the plywood deck, corbels and rubber pads.
- Todd Bridge – there were minor defects in all the FRP girders. Whilst there were water stains on the surface of the girders, there was no evidence of material degradation. There was a good level of interaction with other bridge components, with no signs of distress or material degradation at the interface with the plywood deck, corbels and steel packers.
- Horse Trough Bridge – there were moderate defects in all the FRP girders, mainly the presence of the small fire damage areas and pre-drilled holes left on the sides and bottom of the girders. Water stains were observed on the surface of the girders; however, there was no evidence of material degradation. There was a good level of interaction with other bridge components, with good contact between FRP girder, timber packers and corbels.
- Obi Creek No. 1 Bridge – there were moderate defects in all the FRP girders, with the major damage comprising small areas of exposed fibres around the bolts and water stains on the girder's surface. There was no evidence of material degradation. There was a good level of interaction with other bridge components, with a good contact between the FRP girder, timber packers and corbels (Figure 5.30).
- Oaky Creek Bridge– there were moderate defects in all the FRP girders. Rust-like water stains at the soffit of the girders require further monitoring and investigation. There was a good level of interaction with other bridge components, with a good contact between the FRP girder, timber packers and corbels.

There were performance issues with the complete FRP bridges, including:

- Taromeo Creek Bridge – a level of surface degradation, with differential deflections of adjacent FRP units, exposed fibre and localised damage, and deterioration of the coating due to moisture
- Sandy Creek Bridge – material degradation and cracking of the coating due to UV impacts coupled with moisture ingress and spalling of material with exposed fibre; moisture could have penetrated the material through the cracks.

7.2 Interaction of Replacement FRP Components

In general, there was a good level of interaction of the replacement FRP girders with other bridge components, with a good contact between the FRP girder, timber packers and corbels. Most of the FRP girders showed no signs of excessive deformation or differential displacement (except the sagging of the FRP girders on Heifer Creek No. 7). This indicates that the load-sharing mechanism is in place, and the replacement girder is performing as expected. The most common interaction issues include:

- water leaking from bridge deck (most common)
- moderate corrosion of bolts holding down the decks
- tear of a fibre strips at bolt locations due to installation process or due to the movement of the bolts
- split corbel (only on Coulson Creek Bridge)
- bulging of the rubber pads at supports with corroded steel bearing plates (only on Heifer Creek Bridge)
- minor corrosion of the steel trough deck around the contact areas with the FRP girder (only on Obi Creek Bridge).

The FRP superstructures of the Taromeo and Sandy Creek bridges are also interacting well with the concrete substructures.

7.3 Capacity and Failure Modes of FRP Girders

7.3.1 Ultimate Load Testing of WCFT Girders

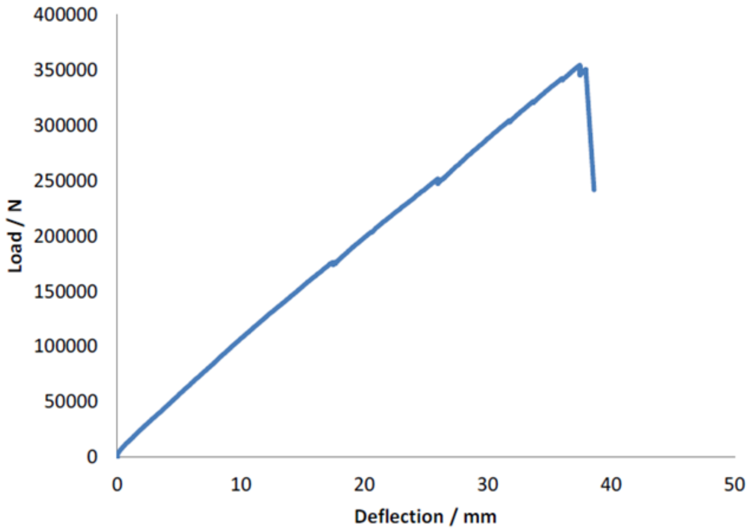
Ultimate shear test

The results of the shear testing of the WCFT girders were reported in Aravinthan et al. (2012a), Kahandawa et al. (2011), and Aravinthan et al. (2012b). The S1 girder loaded in shear to 350 kN was then used for the ultimate bending test. Girder S2 failed under a load of 432 kN, with the failure mode shown in Figure 7.1. The two S3 girders failed at 350 kN and 430 kN, respectively, with the failure mode and load deflection curve of one girder shown in Figure 7.2. It should be noted that the failure of the S3 girders occurred within the glue line between the two pultruded sections.

Figure 7.1: Failure of WCFT S2 girder in ultimate shear test

Kahandawa et al. (2011).

Figure 7.2: Load-deflection curve of WCFT S3 girder in ultimate shear test



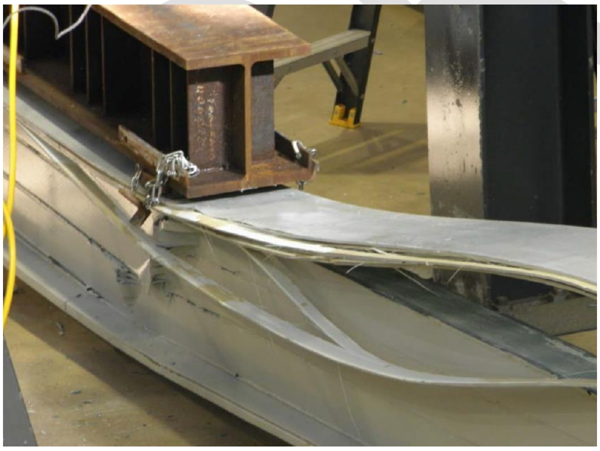
Source: Aravinthan et al. (2012b).

Ultimate bending test

Four-point bending testing was also conducted. Both the S1 and S2 girders failed at 400 kN, while the S3 girder was loaded to 340 kN and then used in the ultimate shear test. The failure modes in bending of S1 and S2 girders shown in Figure 7.3. These failures were brittle, due to the failure of the glue lines between the flange plates (on the S1 girder) and between the web cells (on the S2 girder).

The load-deflection curve for the S2 girder is shown in Figure 7.4. It can be seen that the girder behaved linear-elastic up to failure at a deflection at mid-span of 213 mm.

Figure 7.3: Failure of WCFT girders under 4-point bending test

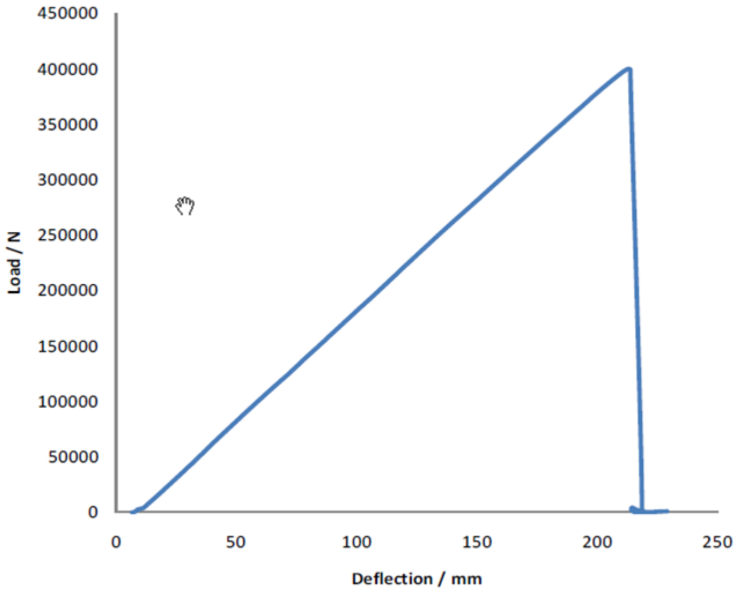


Failure of S1 girder in bending

Failure of S2 girder in bending

Source: Aravinthan et al. (2012a), Kahandawa et al. (2011).

Figure 7.4: Load-deflection curve of S2 girder under 4-point bending test



Source: Aravinthan et al. (2012a).

7.3.2 Ultimate Load Testing of LOC Girders

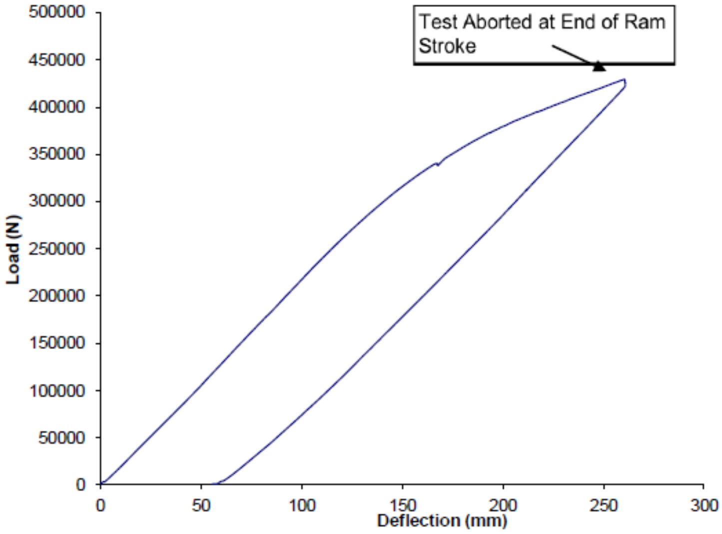
Ultimate shear tests

As reported by Aravinthan et al. (2012a) and Aravinthan et al. (2012c), two LOC girders were tested in shear with a maximum applied load of 350 kN. This load resulted in a shear force of 310 kN on the girder. In both tests, the girders behaved linear-elastic up to the maximum applied load, without any signs of failure.

Ultimate bending test

It was reported by Aravinthan et al. (2012a) that, with a span of 9000 mm in the 4-point bending tests, the two girders tested behaved linear elastic up to 340 kN which is equivalent to a 660 kNm bending moment before the steel started to yield. The test was abandoned at the end of the stroke of the hydraulic cylinder at a load of 429 kN and a maximum deflection of 261 mm. This load is equivalent to an 836 kNm bending moment which is 26% higher than the ultimate bending moment of 660 kNm required by TMR. The tested girder exhibited a ductile behaviour post yield.

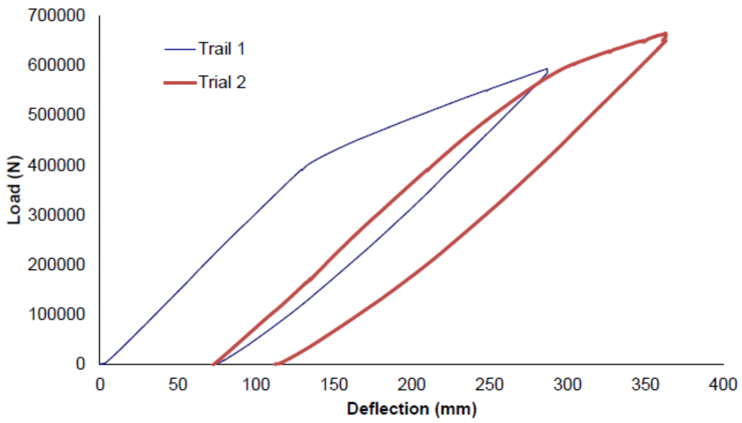
Figure 7.5: Load-deflection curve of LOC girder under 4-point bending test, 9000 mm span



Source: Aravinthan et al. (2012a).

Another bending test was conducted on the stage 2 LOC girder with a shortened span of 8000 mm (Aravinthan et al. 2012c). The girder failed at an ultimate load of 664.7 kN, which is equivalent to a bending moment of 1128.8 kNm and a deflection of 388.7 mm at mid-span. The load-deflection curves for two load cycles are plotted in Figure 7.6.

Figure 7.6: Load-deflection curves of LOC girder under 4-point bending test, 8000 mm span



Source: Aravinthan et al. (2012c).

7.3.3 Observations

Comparing the earlier serviceability tests with those described in Section 4, there has been no significant structural deterioration (at working load) of the WCFT girders (no stockpiled LOC girders were tested). Past test results showed that both suppliers have members that provide operating capacity at about the same ratio with respect to the onset of non-linear behaviour.

Based on the ultimate load tests reported in Section 7.3.1 and Section 7.3.2, the ultimate load behaviour of the girders from each supplier is very different:

- All the WCFT girders were taken to rupture at a small margin above target for key actions (bending and shear).

- Only one LOC girder was tested to failure in bending by shortening the span. There was a large margin above the target (M = 1128.8 kNm compared to M_target = 660 kNm). No girders were taken in rupture in shear.
- Non-linear behaviour was evident for the LOC girders, and all appeared to exhibit substantial ductility.
- All of the WCFT girders exhibited linear behaviour to rupture.
- The failure of the WCFT girders was analogous to timber failure while the failure of the LOC girders was analogous to steel failure.

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Recommendations Based on TMR FRP Strategy

TMR is well positioned to understand the characteristics of FRP structures in the road network, including durability, as a result of previous investments in the technology. The conclusions and recommendations from this project are influenced by TMR's future strategy in relation to FRP materials which could broadly be either:

1. TMR actively pursues the growth of FRP application on its network with significant investment in the future.
2. TMR retains the current level of investment in FRP and continues to increase its understanding of the material without significant further investment.
3. TMR does nothing, i.e. no further pursuit of the FRP technology.

The following recommendations are based on a TMR strategy consistent with (2) above. Based on this strategy and the report findings, it is recommended that TMR:

- utilise the past investment and increase understanding of FRP materials by having FRP components in place to be used as case studies/test samples
- improve the skill levels of TMR staff in the use and durability of FRP
- provide FRP girders to be used as a substitute for timber girders with upgrades required in some cases (specific details are provided in Section 8.2)
- continue monitoring previous investments for at least the next 5 to 10 years (specific details are provided in Sections 8.3 and 8.4); the basis for this monitoring should be consistent with Section 2.6 unless evidence to the contrary is forthcoming
- position TMR to be the best-informed Australian road agency with respect to FRP, given its current advantage resulting from previous investments.

These recommendations should be revised if the assumed TMR-FRP engagement strategy changes.

8.2 Recommendations Regarding the Utilisation of Stockpiled FRP Girders

8.2.1 Stockpiled Girders at USQ

In general, the stockpiled girders at USQ have minor to moderate defects, mainly related to surface damage and coating deterioration, including:

- minor delamination at the flange edges
- minor chipping at isolated locations
- failure of paint coating where water is ponding on the top face of the bottom flange
- fibre exposure due to wear of the coating and physical impact on the top surface.

These defects are localised and do not extend throughout the girders. Due to the nature of these defects, it is recommended that the girders be subject to some minor repair and re-coating before they are used on existing timber bridges.

The results of the serviceability load tests of selected girders (Section 4) indicate the following:

- The load-deflection curves coincide with a relatively straight line, indicating that all the tested girders behaved in a linear elastic manner in the working load range.

- The maximum deflections under the test loads were within the deflection ranges derived from the past tests of the corresponding girders of the same production stage.
- The changes in the bending stiffness of the girders were not statistically significant after 5 to 7 years (a maximum 4.4% reduction in girder G3-3), noting that these girders have been stored in a stockpile where no load other than the self-weight of the girders has been applied.
- All girders tested in 2017 satisfied the TMR stiffness requirements for FRP girders to be used as timber girder replacement on timber bridges (meeting the target bending stiffness, EI, is 2.96×10^{13} Nmm^2).

Based on the current condition and performance test results, the following recommendations are made regarding the utilisation of these girders:

- The girders' structural performance meets the original design requirements, without noticeable material degradation or changes in stiffness. Therefore, they are structurally fit for the original purpose.
- These girders have been stored in an open environment with exposure to direct sunlight and moisture for an extended period of time. It was noted that the bottom layers of the girders had less degradation than the top layer; therefore, it is likely that the main root cause of the degradation is direct sunlight. It is expected that, once installed, the degradation due to UV impact will slow down due to the fact that they will be covered by the bridge deck (as evident in the conditions of the girders of the same type installed in the network – refer to Section 5).
- Minor repair and re-coating is recommended before installation. This work should be carried out by the manufacturer or specialists in accordance with the manufacturer's specifications to avoid further damage.
- Due to the concern related to fire damage if the girders are used in areas prone to bushfires and vandalism, a fire-resistance coating, as recommended by the manufacturer, should be applied.

WCFT girders have undergone a development in the design philosophy, with three production stages (S1–S3) representing the outcome of these development stages and S3 girders being the cumulation of the process. These girders provide the best solution to the issues related to the glue line within the high shear area as discussed in Section 2.2. It is recommended that TMR seek an RFP from WCFT for the following tasks:

- upgrade all girders to S3 girder learnings (shear strengthening for the glue line in the high shear areas)
- recoat all girders using a more advanced resin coating (possibly) to minimise further deterioration, with an additional option for fire retardant.

Based on the RFP, TMR would be in a good position to make an informed decision on whether there is a business case to install the stockpiled WCFT girders on the network.

8.2.2 Stockpiled LOC400 Girders at Nerang

As indicated in Section 3.3.1, in general, these girders have minor defects, with the most pronounced deterioration being the wear of the paint coating at isolated locations along the top edge of the girders. There was no sign of material degradation. It is recommended that these girders be used after rectifying the damaged coating areas.

8.2.3 Stockpiled LOC420 Girders at Maroochydore and Charlton

As discussed in Section 3.3.2 and Section 3.3.3, the stockpiled LOC420 girders have minor to moderate defects, mainly due to the deformation of the thick coating of the resin. Based on the characteristics of the design of the girder as discussed in Section 2.3, however, the deformation of the coating may not have detrimental impacts on the strength and stiffness of the main load-bearing components (including the hybrid composite modules and LOC panels).

These girders should not be used without understanding (and rectifying if required) the root cause and impact of the rust-like stain and the deformation of the coating of resin. It is recommended that TMR engage USQ to:

- Test the rust-like stain residue (found on stockpiled girders and on Oakey Cr Bridge) to confirm if the origin is phenolic or iron. If it is phenolic, then there has been leaching and moisture has entered the cross-section; however, the structural capacity of the girders is unlikely to be affected. If the residue contains iron, then the steel reinforcement may be experiencing corrosion. If this is the case, these girders should not be used and should be set aside in an exposed environment for ongoing durability assessment.
- Train TMR personnel (RoadTek) how to repair surface cracking on the girders.
- Confirm that the surface irregularities identified in Figure 3.24 and Figure 3.29 are related to a sub-optimal manufacturing process, and do not affect the structural capacity of the girders.

Other options that could augment these recommendations include:

- use some girders in their existing condition, i.e. without repair
- leave some girders in the stockpiled condition with more pronounced environmental impacts such as moisture and UV
- leave some LOC girders with rust-like stains in the stockpiled condition and investigate the issues by measuring volume changes (if steel reinforcement is corroded, volume will change due to swelling) – if testing does not conclusively demonstrate that phenolic resin leaching is the source of the rust-like staining.

8.3 Recommendations for Management of FRP Replacement Girders on In-service Timber Bridges

A matrix of the current condition state of bridge versus defects of FRP components is presented in Table 5.2. It is proposed that all FRP components be retained in their current state, to further improve TMR's understanding of FC durability issues over time. These girders should be salvaged for further investigation if any of the bridges are demolished. Deterioration monitoring should continue as part of the L2 inspection, including detailed defect mappings on each FRP components.

Heifer Creek #7 Bridge has the most significant FC defects and was not proof loaded. This bridge should be inspected annually with defects mapped in detail, and the inspection report should be reviewed by TMR E&T. Particular care should be taken to monitor changes in delaminated flanges and de-bonded angles. Any changes in these defects should trigger a detailed structural review. In addition, the reason for the sag in the girders should be investigated and carefully monitored.

For Oakey Creek Bridge, the nature of the brown staining on the FRP girders should be investigated – if phenolic leaching cannot be demonstrated as discussed in Section 8.2.3.

8.4 Recommendations for Management of Complete FRP Bridges

A similar approach to that recommended in Section 8.3 is recommended for the Taromeo Creek and Sandy Creek bridges, i.e. the FRP components should be retained in their current condition for further observation. The differential displacement apparent on the Taromeo Creek Bridge is, however, of structural concern. It is recommended that a L3 assessment be conducted to investigate the root causes and possible consequence of the differential deflections and, if appropriate, identify remediation options. The L3 assessment should include measurement of the movement of the girders under random traffic and modelling of the bridge to better understand its behaviour under traffic loads.

8.5 Other Recommendations

8.5.1 Review of ϕ Factor

The Civil Aviation Safety Authority of Australia guide (quoted by Karbhari 2009) states that a reduction factor of 1.8 must be used for FRP composite primary structures subjected to hot-wet structural testing, and that a factor of 2.25 be used in the absence of specific consideration of these environmental conditions during testing. Factors such as load environment, fatigue spectra, materials process conditions, and other uncertainty should also be considered when determining the reduction factor.

For TMR FRP components and structures, a material factor of 0.25 has been used (Shaw & van Erp 2009). This value was derived based on various factors to account for a variety of uncertainties in production and servicing condition. This value may be overly conservative. A review (and adjustment) of this reduction factor may facilitate more commercially viable FRP products for infrastructure applications. In the light of the discussion in Section 2, it is believed that it is not possible to have a general material reduction factor for FC components. Specific material reduction factors should take into consideration specific failure modes and their effects on reduction. Currently, there does not appear to be a strong motivator to resource this review, but resourcing should be considered if TMR anticipates a substantial investment in FC components.

8.5.2 Update Current TMR Guidelines

If TMR actively pursue the growth of FRP applications on its network, it is recommended that a thorough review of design criteria and specification be carried out, including the ϕ factor (discussed in Section 8.5.1). This should be accompanied by FRP industry stakeholder engagement. Specific details are provided in this Section.

TMR has published the following technical guidelines related to FRP materials:

- MRTS59 Manufacture of FRP Composite Girders (TMR 2017a)
- MRTS60 Installation of FRP Composite Girders (TMR 2017b)
- Chapter 16 of Design Criteria for Bridges and Other Structures: Design of Fibre Reinforced Polymer (FRP) Composite Girders (TMR 2018)
- Component 22O of TMR Structures Inspection Manual (TMR 2016).

Learnings form the ‘Bridging the Gap’ project have been reflected in recent updates to these guidelines. Some examples include:

- failure at the glue line under ultimate shear test is not permitted
- warning of failure is required (noting that the WCFT girders failed in bending without exhibiting ductility behaviour)
- fire resistance is required
- drilling is only allowed at pre-defined areas.

Component 22O of the TMR *Structures Inspection Manual* (TMR 2016) provides descriptions of defect rating for two FRP girders types including WCFT and LOC girders. The following comments are made:

- WCFT girders (stockpiled and used on existing timber bridges) have no steel reinforcement; therefore, any details related to steel included in Component 22O need to be revised.

- Deterioration/damage of the glue lines should be considered for rating the component as they are critical to the structural integrity and capacity of the girder.
- LOC girders (stockpiled and used on existing timber bridges) use LOC panels instead of LVL and do not use steel stressing strands. These details require updating.
- Surface cracking and deformation of the thick coating of resin of LOC girders should be addressed.

A defect rating system based on the severity and extent of defects adopted from the UK Highway Agency (2007) was proposed for rating FRP components as discussed in Section 2.5. Due to the variation in the design principles of different manufacturers as discussed in Section 2.1 to Section 2.4, the significance of defects and their extent would be better reflected if this system was used.

8.5.3 Learnings for FRP Strengthening

Given that TMR has an interest in FRP strengthening as a technology, there may be benefit in reviewing previous FRP strengthening projects completed in Australia over the past 20 years and comparing them with the findings of this investigation.

8.6 Conclusions

In-service investigation techniques were used in this project to investigate the performance of TMR FRP components and structures, including close visual inspection and serviceability load testing. The investigation covered all existing timber bridges with replacement FRP components and complete FRP bridges on the TMR network, as well as unused FRP girders stockpiled at various Districts. This investigation has:

- provided a baseline knowledge base for the performance of existing FRP components and structures under TMR management as summarised in Section 3 to Section 6
- confirmed that the interaction between replacement FRP components and existing timber components are satisfactory, with no significant issues noted.

Recommendations are provided based on an assumed TMR strategy (with respect to FRP) that **TMR retains investment in FRP and continues to improve its understanding of the material without significant further investment**. Based in this assumption, the investigation provides a basis for TMR to make an informed decision on the utilisation of currently-stockpiled FRP girders. Post-inspection defect mapping should be documented (similar to that in this report) when these girders are utilised to provide baseline data for future inspections.

Recommendations are made regarding the maintenance, monitoring and further investigations of the durability and long-term performance issues of these FRP components and structures. It is recommended that a benchmark review similar to that described in Sections 3 to Section 6 be undertaken over the next five to 10 years. In the first instance, this could be a desk-top review based on standard TMR inspection data.

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APPENDIX A FRP GIRDER DESIGN DRAWINGS

A.1 WCFT Girders

Figure A 1: WCFT FRP girder type S1

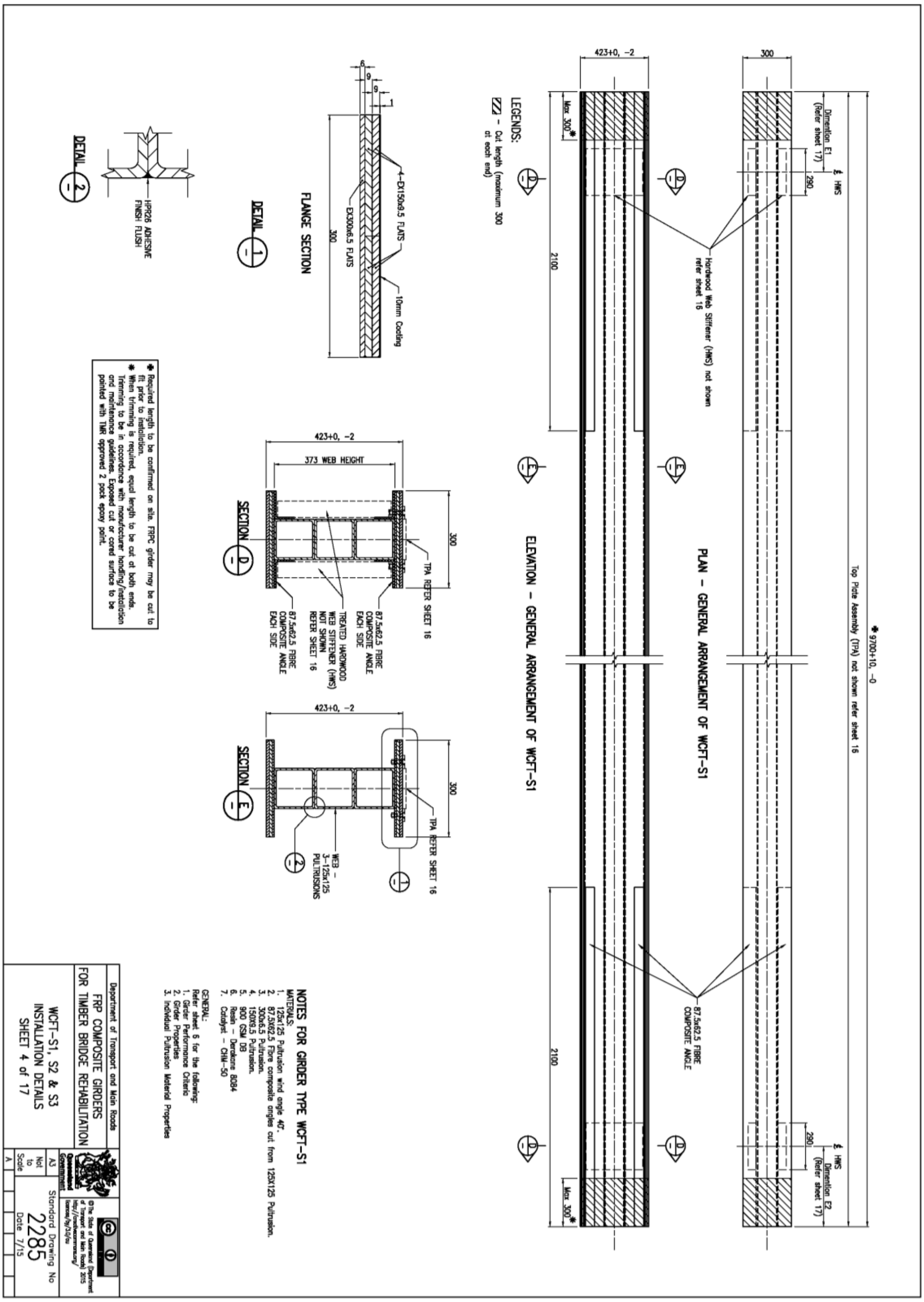


Figure A 2: WCFT FRP girder type S2

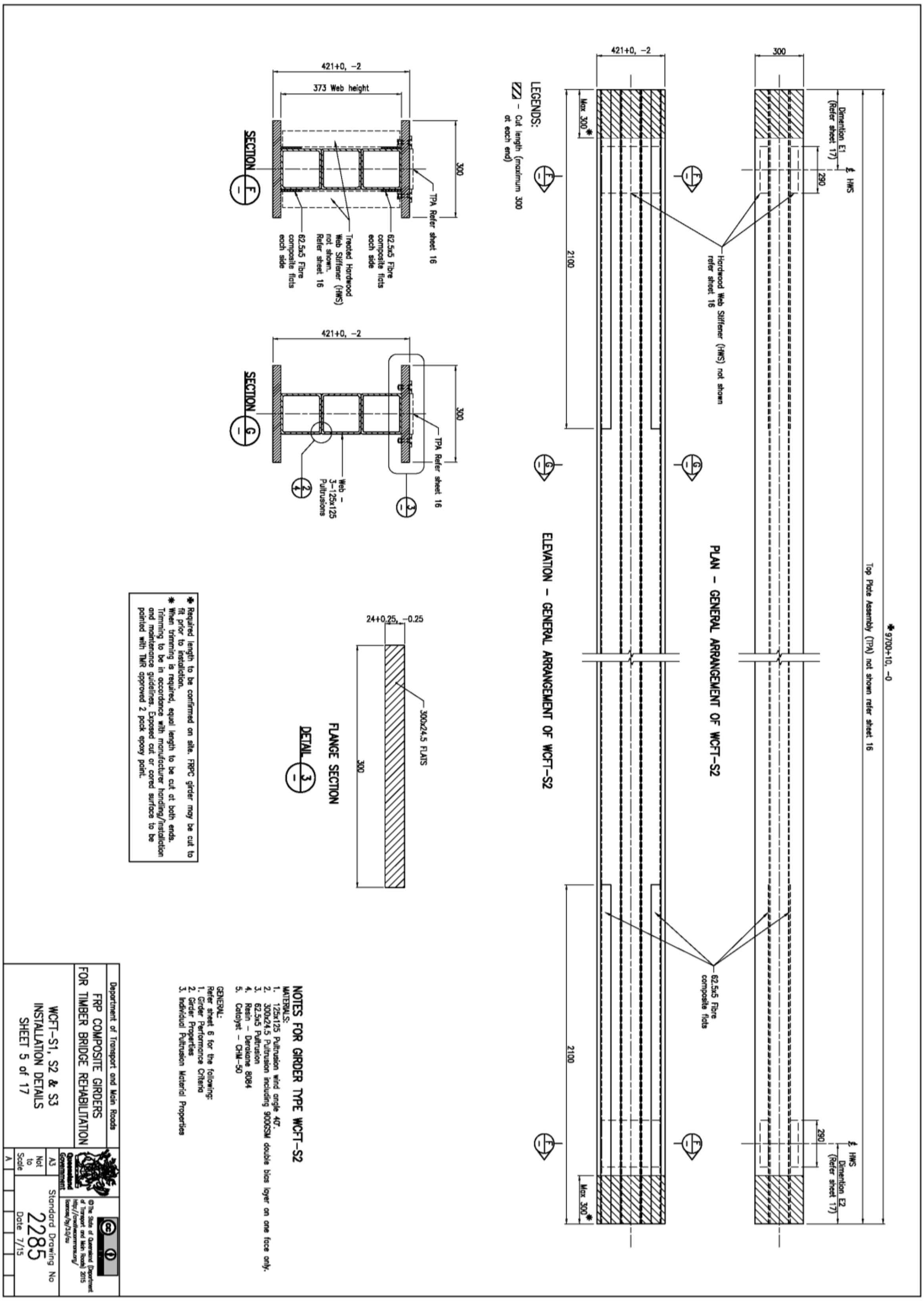
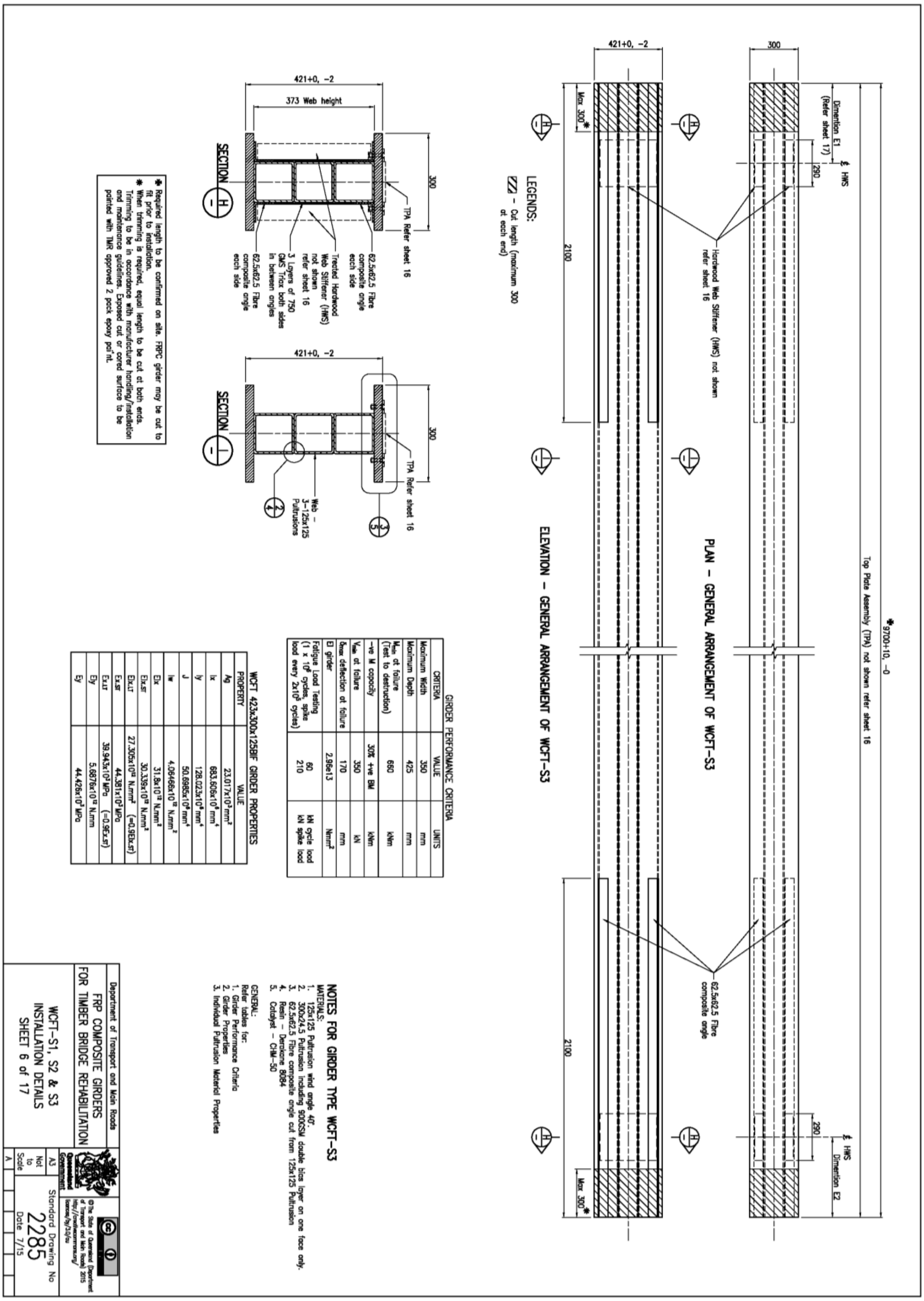


Figure A 3: WCFT FRP girder type S3



GIRDER PERFORMANCE CRITERIA

CRITERIA	VALUE	UNITS
Maximum Width	350	mm
Maximum Depth	425	mm
M_{min} at failure (Test to destruction)	660	kNm
−ve M capacity	30% +ve BM	kNm
V_{min} at failure	350	kN
δ_{max} deflection at failure	170	mm
EI girder	2.96e13	Nmm^2
Fatigue Load Testing (1×10^6 cycles, spike load every 2×10^5 cycles)	60 210	kN cycle load kN spike load

WCFT 423x300x125BIF GIRDER PROPERTIES

PROPERTY	VALUE
Ag	$23.017 \times 10^3 mm^2$
Ix	$683.606 \times 10^6 mm^4$
Iy	$128.023 \times 10^6 mm^4$
J	$50.6985 \times 10^6 mm^4$
Iw	4.06466×10^{12} N.mm²
EIx	31.8×10^{12} N.mm²
$EIx_{.ST}$	30.339×10^{12} N.mm²
$EIx_{.LT}$	27.305×10^{12} N.mm² (=0.9$EIx_{.ST}$)
$Ex_{.ST}$	44.381×10^3 MPa
$Ex_{.LT}$	39.943×10^3 MPa (=0.9$Ex_{.ST}$)
EIy	5.6876×10^{12} N.mm
Ey	44.426×10^3 MPa

A.2 Loc Composites Girders

Figure A 4: LOC FRP girder type LOC400

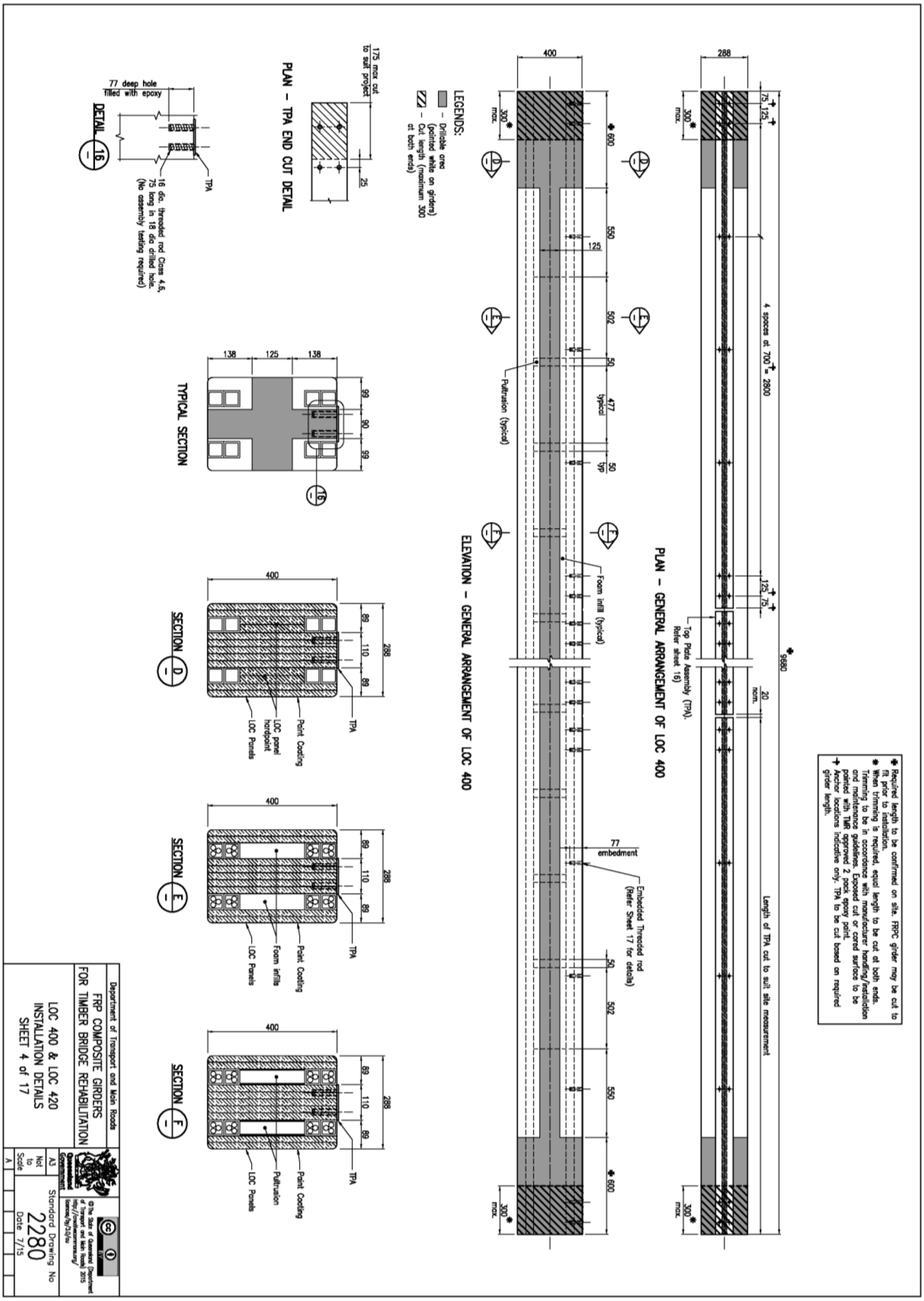


Figure A 5: LOC FRP girder type LOC420