Jamtsho, Lungten, Dhanasekar, Manicka, Palliyaguru, Nishantha, & Bayissa, Wirtu (2010) *Effects of opening in the cross girders of the flat bottom rail wagons to the load transferring mechanisms when used as road bridge deck.* In: Proceedings of the 21st Australian Conference on the Mechanics of Structures and Materials, 7-10 December 2010, Victoria University, Melbourne.

Effects of opening in the cross girders of the flat bottom rail wagons to the load transferring mechanisms when used as road bridge deck

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ABSTRACT: A Flat Bed Rail Wagon (FBRW) has been proposed as an alternative solution for replacing bridges on low traffic volume roads. The subject matter for this paper is to investigate the impediment to load transfer from cross girders to main girder, through visually identifiable structural flaws. Namely, the effect of having large openings at close proximity to the connection of the main girder to the cross girder of a FBRW was examined. It was clear that openings locally reduce the section modulus of the secondary members; however it was unclear how these reductions would affect the load transfer to the main girder. The results are presented through modeling grillage action for which the loads applied onto the FBRW were distributed through cross girders to the main girder.

1 INTRODUCTION

Addressing the call for replacing aged bridges in low traffic volume roads under Local Government jurisdiction, the Infrastructure Research Group of Queensland University of Technology (QUT) investigated the feasibility of using decommissioned FBRWs. The attractiveness of this type of bridge solution lies in the fact that fatigue plays less a severe role in low volume traffic roads. There have been a number of research projects completed in the USA using decommissioned Rail Road Carriage Cars (RRCC), particularly in the State of Iowa (Doornick et al., 2003). Although the reasons for implementing such solutions have been comparable, there were significant differences amongst the structural systems of RRCC and FBRW. The RRCC consists of deeper and wider members made from thicker steel sections compared to the FBRW. The width of the RRCC is also much larger, and when attached side by side it caters for a double lane road bridge deck. The FBRW employed here is much narrower, as it was obtained from Queensland Rail (QR), which services a narrow gauge network system; a connected double FBRW system would satisfy only a single lane width specified in accordance to the Australian Bridge Code (AS5100.2, 2004). Although three FBRWs could still be connected to form a double lane road bridge, it was decided to thoroughly examine the performance of an FBRW in a simpler single lane bridge configuration prior to extending its use for other wider bridge designs.

The project scope involved:

- Full scale testing of a single FBRW to SM1600 loadings, as stated in Australian Bridge Code (AS5100.2, 2004),
- Analysing a three dimensional finite element model which represented the experimental setup,
- A grillage analysis of both single and two side by side connected FBRWs,
- Construction of the bridge at a location in Rockhampton Regional Council as a publicly used demonstration project
- Execution of performance testing prior to opening for public usage.

The reasons for carrying out the rigorous analyses itemised in the scope above have been threefold:

- The FBRW contains relatively fewer redundant members compared to the RRCC,
- there has been next to no scientific research for FBRWs
- There's perceived excessive loading imposed by the Australian Bridge Code (AS5100.2, 2004) when future heavy loading is incorporated.

The key advantages of using FBRWs over a conventional bridge deck solution include:

- These offers less construction time and low initial deck cost as the wagons are available at scrap value from the QR

- Wagons can be easily placed on existing or new abutments with minor adjustments/modifications within a short period of time.

The project team employed a grillage model as well as a finite element model of both single and side by side connected FBRWs to present the effect of local loss of section modulus, due to openings and/or metal loss in secondary and decking members (cross-girders and folded plates), on the deflection of the main girder and cross-girders.

2 OVERVEIW OF FBRW SYSTEM

The structural components of the FBRW are categorised in to three types; primary members, secondary members and decking members. Primary members consist of a main box girder located at the centre of the FBRW, spanning in the longitudinal direction. The secondary members consist of inverted T sections spanning transversely and two edge Z-beams running parallel to the main girder and the channels oriented in both directions. The decking members are made up of a series of folded plates welded to the primary and secondary members. The connections of the decking members are poorly formed welded connections whereas all the other connections are in reasonably sound condition. Both the main and cross girders are tapered at the ends.

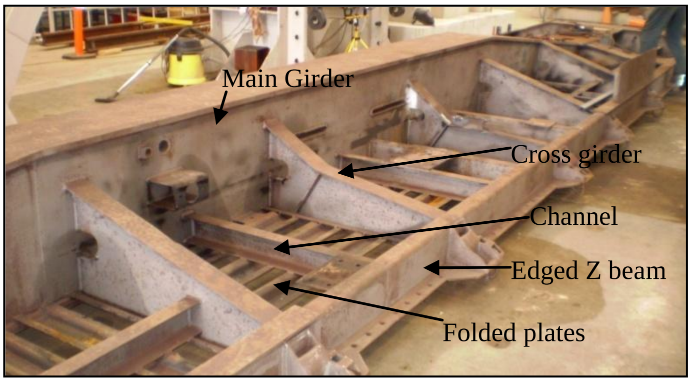


Figure 1. Structural Components of FBRW

3 DEVELOPMENT OF GRILLAGE MODEL

A grillage model of an FBRW was created in SPACE GASS, a 3D structural analysis and design program. The members were individually modeled and then connected to adjacent members. A rigid connection was assumed in the model – a design check of the welded connections has proved that the weld sizes were quite conservative, thus alleviating any fear of early relative rotation of members during serviceability/ultimate loading.

Tapered sections were modeled by sub-dividing the member into many sub-sections with their independent neutral axes computed and connecting their nodes together using rigid links. The master-slave option was employed in connecting two nodes to maintain structural compatibility. The master-slave approach adopted is shown in Figure 2.

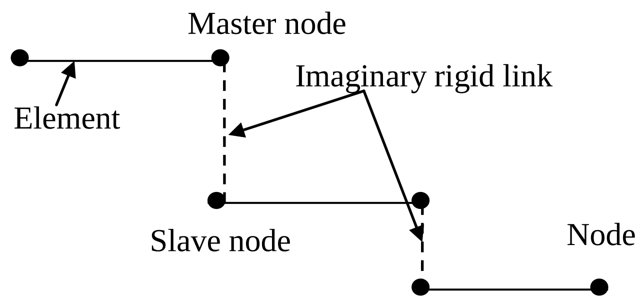


Figure 2. A typical connection of two nodes

Openings in the transverse members were idealised by computing independent section moduli and neutral axes and re-arranging them in SPACE GASS using as the common reference the centriod of the main girder. This modeling technique is illustrated in Figures 3 and 4.

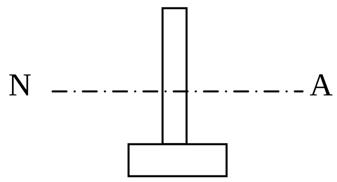


Figure 3. Cross section without opening

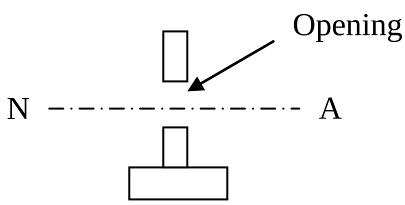


Figure 4. Cross section with opening

The overall reduction in section modulus due to these openings in the transverse girder is only about 1% of the total global section modulus. For example, the elastic section modulii of members with and without openings was 1.3490E+13 mm^3 and 1.3355E+13 mm^3 respectively. Support (boundary conditions) were modeled to simulate the elastomeric rubber pads used in the laboratory test (as shown in Fig. 5) and were idealized in the model using spring support. The spring stiffness 192.50 kN/mm is employed on each node restraint at the supports.

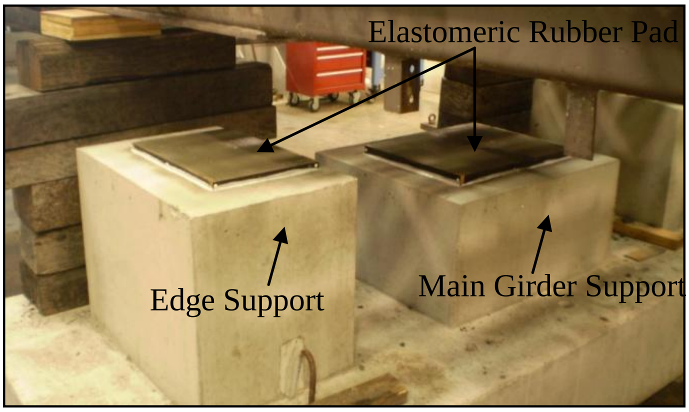


Figure 5. Elastomeric rubber pads for support

The single and double FBRW models are shown in Figures 6 to 8.

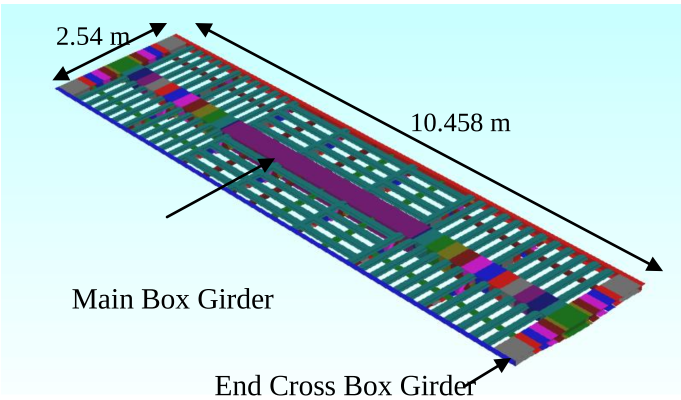


Figure 6. Single FBRW

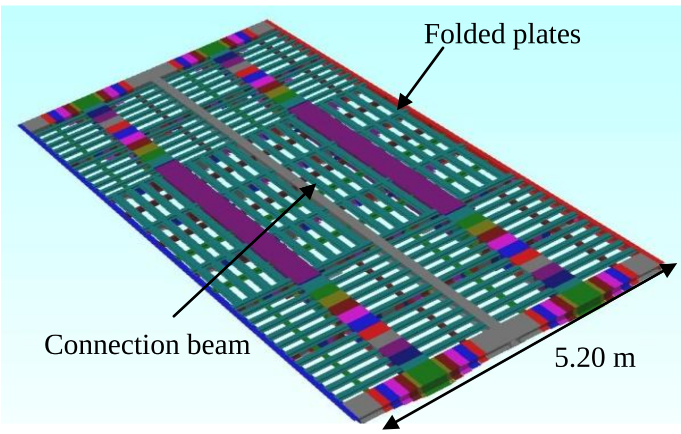


Figure 7. Double FBRW

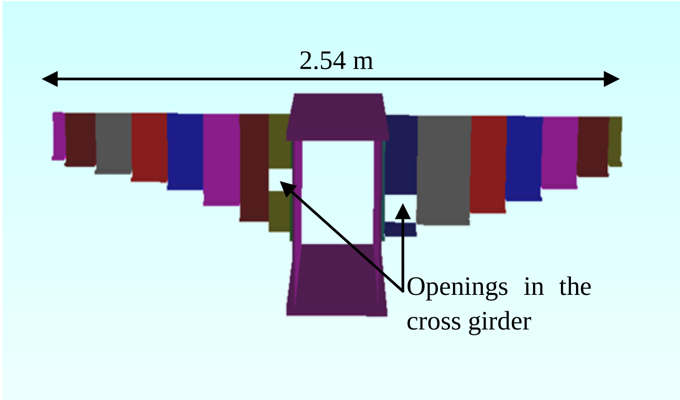


Figure 8. Cross girder with openings

4 DESIGN LOAD EVALUATIONS

AS 5100.2 (2004) – Bridge Design Part 2: Design Loads was widely referenced for evaluating the traffic moving loads for both serviceability and ultimate limit states. Although the bridge deck was required to be analysed and checked for SM1600 loads (AS 5100.2, 2004), for the purpose of this paper only W80 load was used on the models. The W80 comprised of an 80kN wheel load spread over a contact area of 400 mm x 250 mm. The loads adopted in the analysis inclusive of load factors (1.4 dynamic factor, 1.0 load factor for serviceability and 1.8 for ultimate) are presented in Table 1.

Table 1. Factored load considered for analysis

Type of load	Serviceability Load	Ultimate Load
W80	112.00 kN	201.6 kN

5 COMPARISON WITH EXPERIMENTAL RESULTS

In order to verify the accuracy of the SPACE GASS grillage model, the static responses predicted by the model were compared with the following:

- FE Abaqus model and
- Full scale laboratory testing

The design factored wheel load was positioned on the model exactly at the same location as that of the laboratory test. The strain readings obtained from the experiment and bending moment calculated from the grillage analysis were converted to bending stress using the basic bending stress formulae for a beam:

$$\sigma = E\varepsilon \tag{1}$$

$$\sigma = \frac{M}{Z} \tag{2}$$

where, σ = stress in MPa, ε = strain in microstrain, E = Young's modulus of elasticity in MPa, and Z = the section modulus in mm^3. These results are presented in Table 2.

Table 2. Comparison of results of main box girder under W80 loading

Structural Response	Grillage Model	FE Model	Test
Mid-span deflection (mm)	4.63	5.30	5.40
Bending stress at mid-span (MPa)	31.02	31.14	31.73

The difference in deflections obtained from analysis and by experiment was approximately 14.3%. This difference can be attributed to the relatively rigid connection assumed in the model compared to the less rigid connection of an actual FBRW. In the case of bending stress, the model reasonably compares with the results of both the FE model and experiment; the variation was only about 2.5%.

6 ANALYSIS RESULTS AND DISCUSSION

The W80 load was applied to three varying locations along the FBRW as stated below and illustrated in the Figures 9 to 11.

- Case (i) – W80 on the main girder
- Case (ii) – W80 on the cross girder, 994 mm from the centre line of main girder
- Case (iii) – W80 on the cross girder, 400 mm from the centre line of main girder

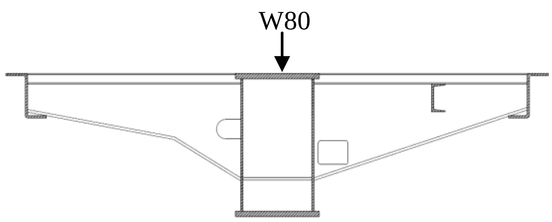


Figure 9. W80 on the main girder

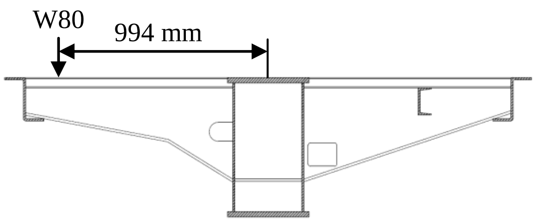


Figure 10. W80 on the cross girder, near edge beam

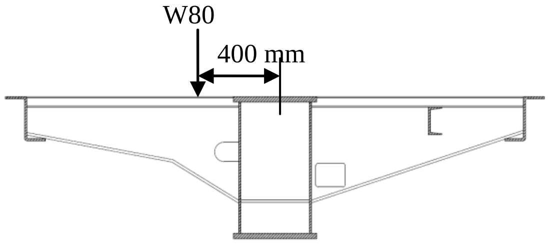


Figure 11. W80 on the cross girder, near the main girder

The vertical displacements and bending moments along the main girder, representative of global effects, with and without openings in the cross girder are presented in Table. 3.

Table 3. Deflection and bending moment of main girder under W80

W80	Serviceability Limit		Ultimate Limit	
	Deflection in mm		Bending Moment in kNm	
	with opening	without opening	with opening	without opening
Case (i)	4.63	4.63	175.26	175.35
Case (ii)	4.63	4.63	161.24	161.31
Case (iii)	4.62	4.62	124.76	124.64

The above results demonstrate that the W80 load as an individual load case was ineffective for global structural responses. However W80 being the heaviest single wheel load needs to be considered for local effects in the cross girder, particularly the effects of openings in the load transfer mechanism.

A serviceability load of 112kN was then applied on the cross girder at two locations, corresponding to case (ii) and case (iii) above. The results obtained from the analysis are tabulated in Table 4.

Table 4. Deflection of cross girder (with and without opening) under W80

Scenario	Deflection in mm	
	Case (ii)	Case (iii)
With Opening	5.063	1.621
Without Opening	5.039	1.613
Increase in deflection	0.024	0.008
Percentage Increase	0.5 %	0.5%

The analysis results indicated that the increase in deflection due to section loss in the form of openings in the cross girder was very minimal, in the order of only 0.5%.

The maximum vertical displacement profile along the cross girder with openings under a W80 serviceability load is presented in Fig. 12. The maximum deflections were far below the deflection limit of AS 5100.2 (maximum deflection is given by 1/300th of the span length for cantilever projection, or a calculated value of 3.41 mm), thus satisfying the serviceability requirement of the Australian Bridge Code (AS5100.2, 2004).

The maximum bending moment and shear force profiles along the cross girders under W80 ultimate load are presented in Figures 13 and 14. As shown the cross girder was able to counteract W80 loading successfully.

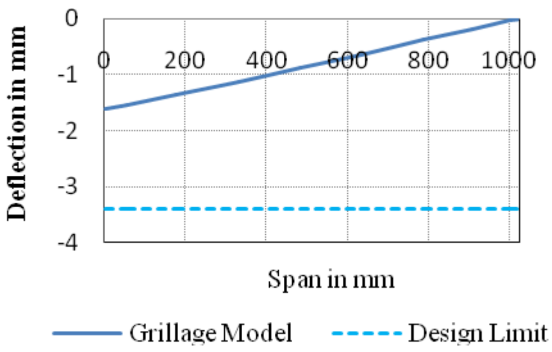
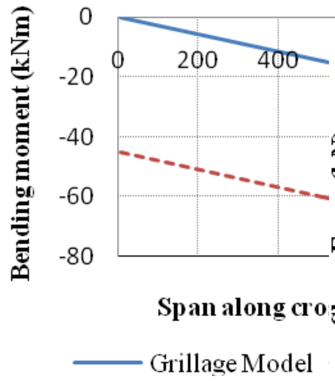


Figure 12. Deflection profile along cross girder



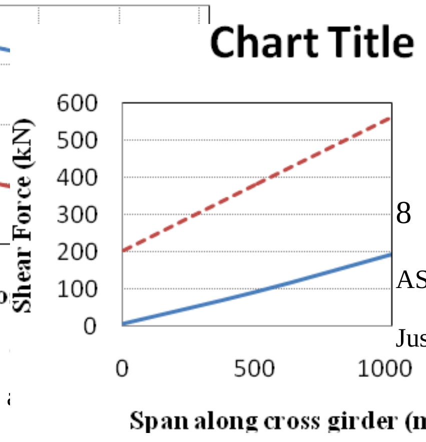


Figure 13. Bending moment a

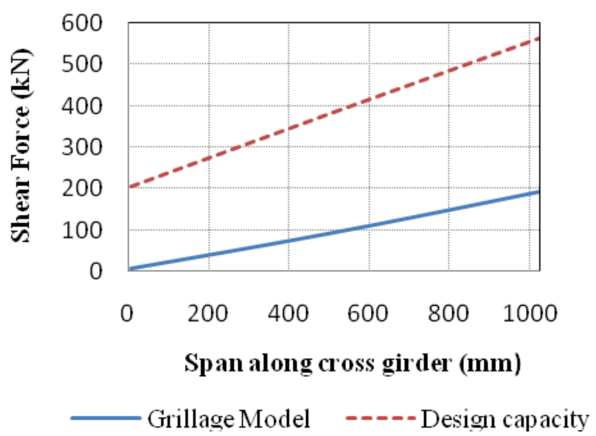


Figure 14. Shear force along cross girder

7 CONCLUSIONS

The following general conclusions were drawn from the above study:

1. Since the disused FBRWs available in Queensland have small structural load resisting members (shallow deep main box girder and inverted T-beams with large openings), a careful finite element study together with full scale laboratory testing for use of such a structural system in a single lane bridge was required prior to implementation at site.
2. Although the FBRW selected in this research was structurally adequate to resist high axle load in low traffic volume single lane road bridges under Local Government jurisdiction, for heavy traffic volumes detailed fatigue studies are recommended prior to implementation at site.

The following specific conclusions were drawn from the above study:

1. The openings in the transverse member have minimal effect in the load transfer and therefore these are not a sensitive issue in adopting disused FBRWs as the bridge superstructure in low traffic volume roads.
2. SPACE GASS was found to show limitations in modeling the openings in the cross-girder. A more appropriate software package which can incorporate a finite element modelling strategy is recommended for any future analysis.

8 REFERENCES

AS 5100 – 2004 Bridge Design Code – Section 2: Design Load. Austroads publication, Sydney Australia.

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