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Testing a novel FRP railway footbridge

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Abstract

Responding to the urgent need to replace pedestrian level crossings with footbridges on the UK rail network, a fibre reinforced polymer (FRP) bridge has been designed. This novel design incorporates a pretensioned, modified Warren truss structure formed from pultruded FRP sections. The pretension system ensures that the diagonal members in the Warren truss are always held in compression. This eliminates the need for bolted joints. To validate this design, a proof test was carried out, applying a load of 5.16kN/m2 to the bridge deck. The behaviour of the truss assembly was monitored with a strain gauges attached to the pretension system. This test demonstrated that the diagonal members remained in compression, validating the design analysis and proving the footbridge fit for purpose.

Keywords: Footbridge, strain gauge, pre-tensioned, fibre reinforced polymer.

1 Introduction

Within the UK rail network, particularly within rural settings, pedestrians are allowed to cross railway tracks on level crossings. This led to nine pedestrian deaths in 2012-13 [1]. In response to this, crossings are being replaced with footbridges. However, the 2018/19 railway safety report [2] noted >1500 incidents of pedestrians crossing unsafely. Approximately 3,400 pedestrian level crossings are present on the network.

A significant barrier to erecting traditional steel footbridges in rural sites is lack of access for heavy lifting equipment. If equipment and structures cannot be delivered to the site via the railway, access would result in unacceptable environmental damage.

Steel structures also require substantial concrete foundations with a high environmental impact. In response to this, fibre reinforced polymer (FRP) bridges have been proposed [3]. These much lighter bridges can be delivered via the track and erected using rail-based equipment. They also have less substantial foundations and an inherently lower environmental impact [4].

Although bridges formed from bespoke moldings have been produced [5], a lower cost option is to use pultruded FRP sections. These are available in a range of cross section shapes, replicating roll formed steel sections and aluminium extrusions. Channel and angle forms have previously been used to form structures with bolted joints. A challenge with this type of construction is the number of fasteners required and the localized loading of the FRP material.

An FRP footbridge was designed in accordance with relevant Eurocodes and UK National Annexes [6] to provide a twin track span of 17m and a deck width of 1.8m. This resulted in the modified Warren truss configuration shown in figure 1. The main truss structure of the bridge was formed from 132mm hollow square section pultruded FRP. To hold these sections together, internal, pretensioned rods were used. These rods ran the full length of each diagonal member and were fastened inside the horizontal chords. Analysis showed that with the nominal pre-load applied to each internal rod, the worst-case loading resulted in the tensile loads in these rods increasing by 34.5% where the diagonal was in tension and reducing by 33.3% for the diagonals in compression. The resultant axial loads in the diagonal members are shown in figure 2.

To demonstrate that the structure was behaving as intended, the load test described in this paper was carried out.

Figure 1. Prototype bridge at RailLive, 2021

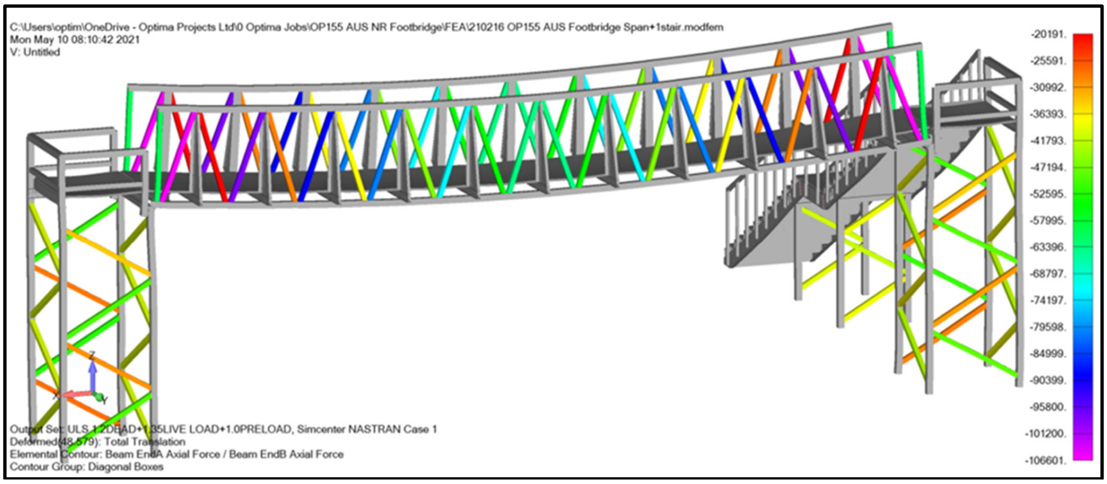

Figure 2. FEA predicted axial loads [6]

2 Methods

A key unknown with the novel footbridge design was how well the pre-load applied to the structure would be sustained as external load was applied. A number of factors including part geometric tolerance, joint settlement and creep were judged to be potentially important. To monitor the preload in the internal rods, a full bridge strain gauge rosette was attached to each rod, as shown in figure 3.

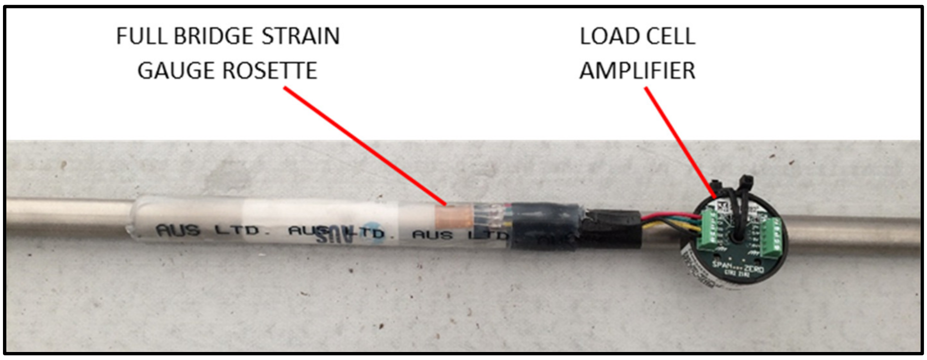

Figure 3. Tension rod with strain gauge rosette and load cell amplifier

Based on the maximum load that the tension rods were predicted to be subjected to, a full bridge rosette would give a sensitivity of 2.8 mV/V. To minimize the effects of electrical interference, a miniature load cell amplifier was positioned adjacent to the strain gauge rosette, as shown in figure 3. The tension rods were calibrated by applying a load within a loading frame, with an externally calibrated 100kN load cell in the load train, as shown in figure 4. As the bridge structure was assembled, power and signal cables to the load cell amplifiers were routed internally within the structure to a monitoring point at the end of the span.

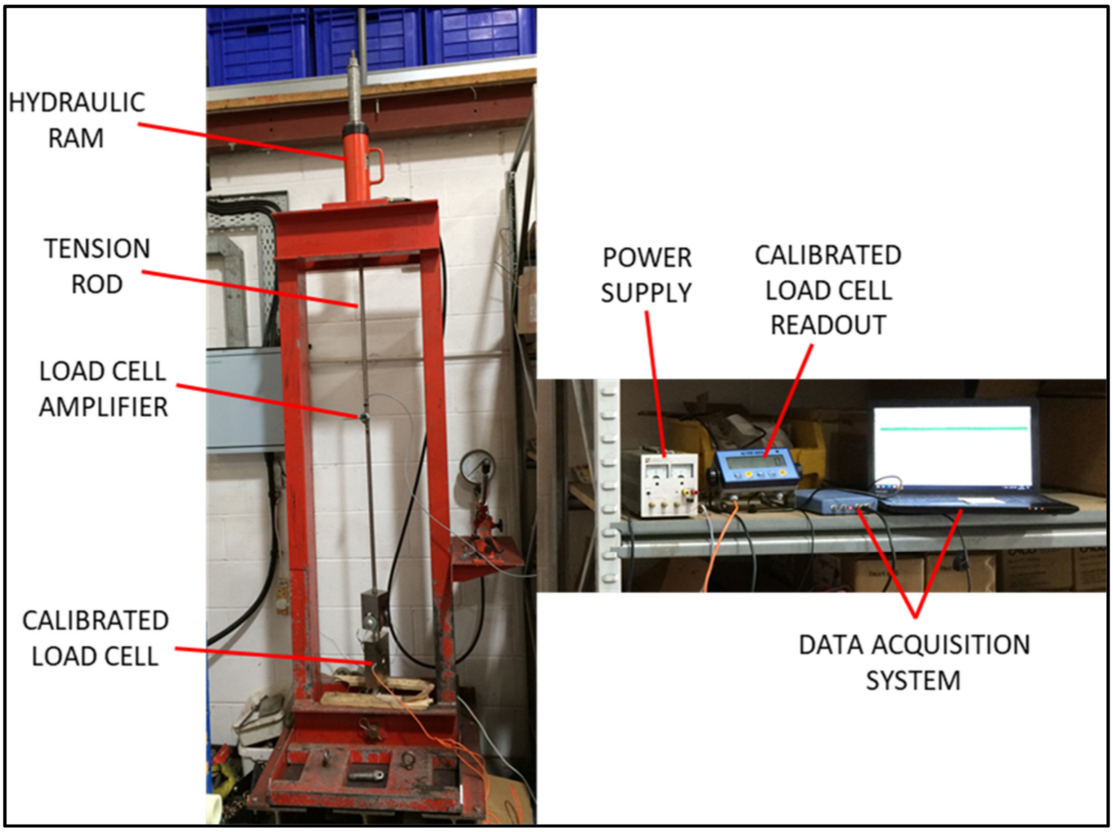

Figure 4. Tension rod calibration

To investigate changes in pre-load due to live loading, the bridge span was supported approximately 0.5m above ground level, via the 4 points where it would be connected to the pedestals. 16 intermediate bulk containers (IBC's) were then arranged along the span, as shown in figure 5. These containers each had a capacity of 1m3, which, when filled with water gave a distributed load of 5.16kN/m2. The bridge was designed for a live load of 5kN/m2.

To safely monitor displacement of the span, linear variable resistors were place between ground and the underside of the span. Three measurement points were used: the two sides and the deck centre in the centre of the span.
Load was applied to the bridge by filling the IBC's with water to pre-determined levels. The initial fill was designed to apply 20% of the total load, accounting for the empty mass of the IBC's. Subsequently, load was applied in 20% increments by adding 200 litres of water to each IBC.
Measurements were taken at each load increment. Due to the time required to add water to the ICB's, at 80% load, the structure was left over-night and loading continued the following morning

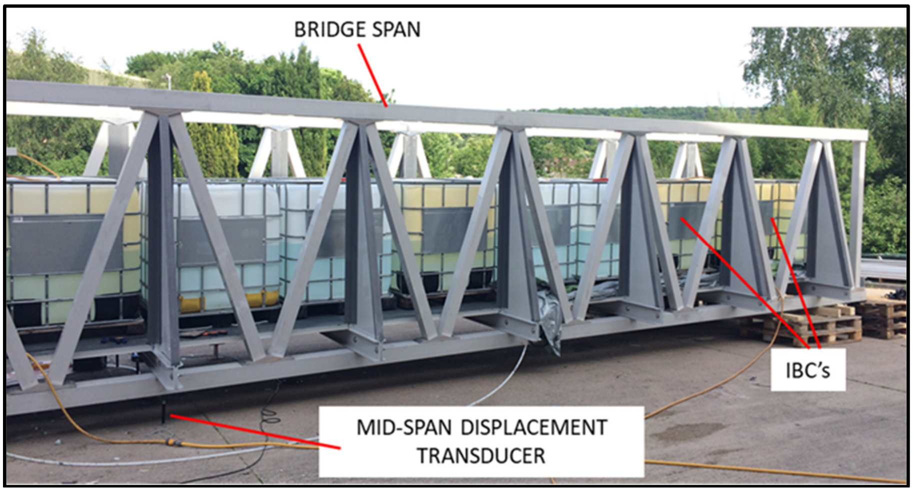

Figure 5. Bridge span test arrangement

3 Results

Figure 6 shows a typical tension rod calibration chart. The signal voltage is the output from the load cell amplifier. Good linearity, minimal hysteresis and a very small level of zero offset is shown. This gave confidence that the tension rods would give a reliable indication of the load applied to the structure.

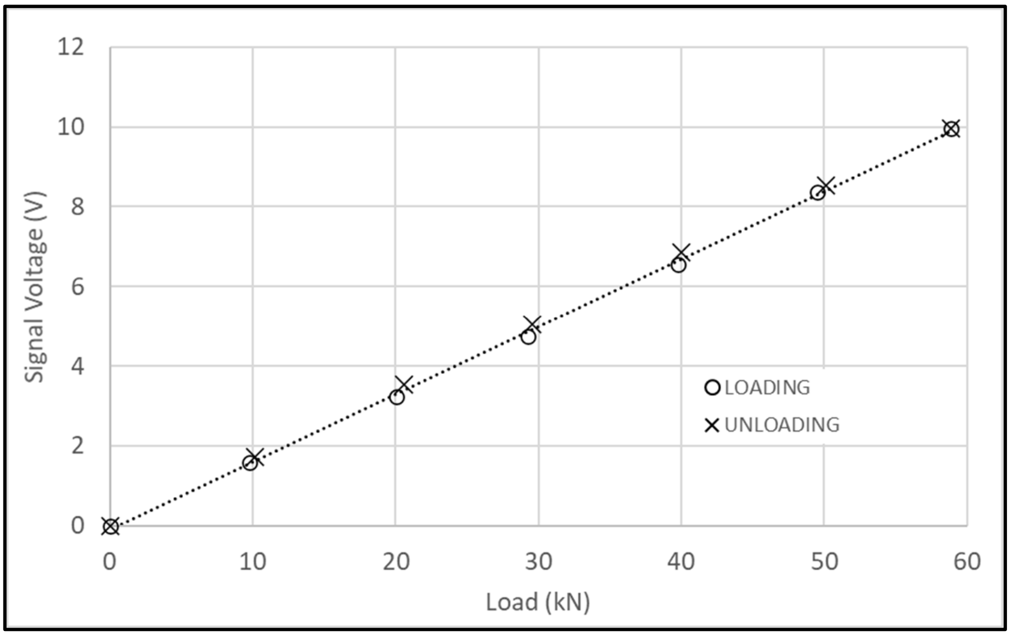

Figure 6. Tension rod calibration chart

Figure 7 shows the tension in the rods prior to the live load being applied. Damage to the wiring during span assembly meant there was no signal from rods 6 and 7. Except rod 3, rod tension was between 90% and 105% of nominal. Where preload has reduced, this may have been due to some settlement within the joint. The change in load is not uniform due to these measurements being taken when the structure was subjected to deadload. In a standard Warren truss arrangement, with bolted joints under deadload, the odd numbered members in figure 1 would be placed in compression and the even numbered members in tension. In the pre-loaded structure, this effect has increased the tensile load in the even numbered rods and reduced it in the odd numbered rods.

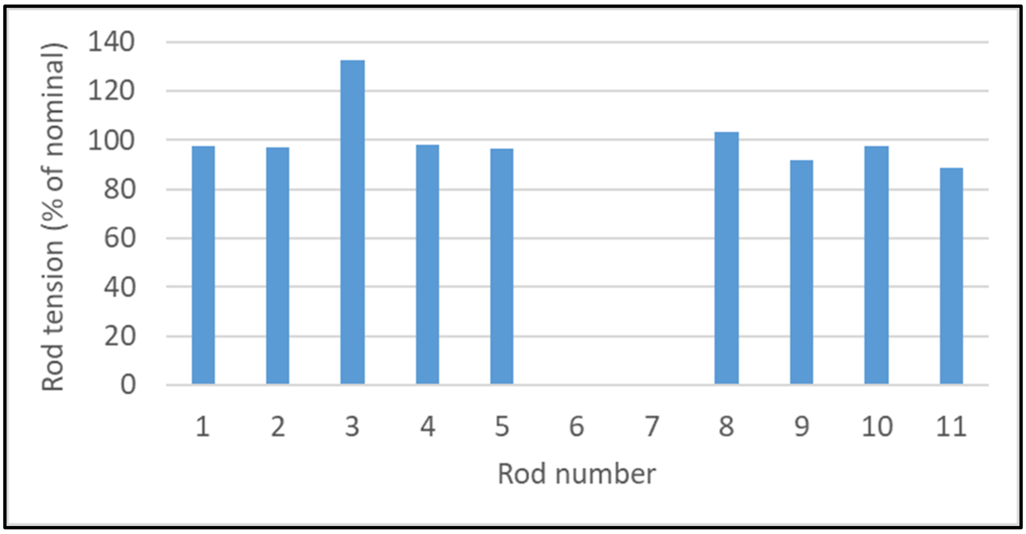

Figure 7. Load distribution within tension rods – no live load

Figure 8 shows the tension in the rods as the load was applied and subsequently removed from the bridge. The load applied at each point is summarized in table 2. The sharp rise in load in rod 3, followed by a sudden decrease indicates that there was some misalignment in the joints on this member which was recovered as load was applied. The sudden removal of this load has then resulted in a redistribution of loads in other rods/members, particularly those close to member 3.

Load point	Load applied (kN/m^2)	Load point	Load applied (kN/m^2)
1	0	7	4.13 (morning)
2	0.258	8	5.16
3	1.03	9	4.13
4	2.06	10	3.1
5	3.1	11	2.06
6	4.13 (evening)	12	1.03
		13	0.258

Table 2. Load points

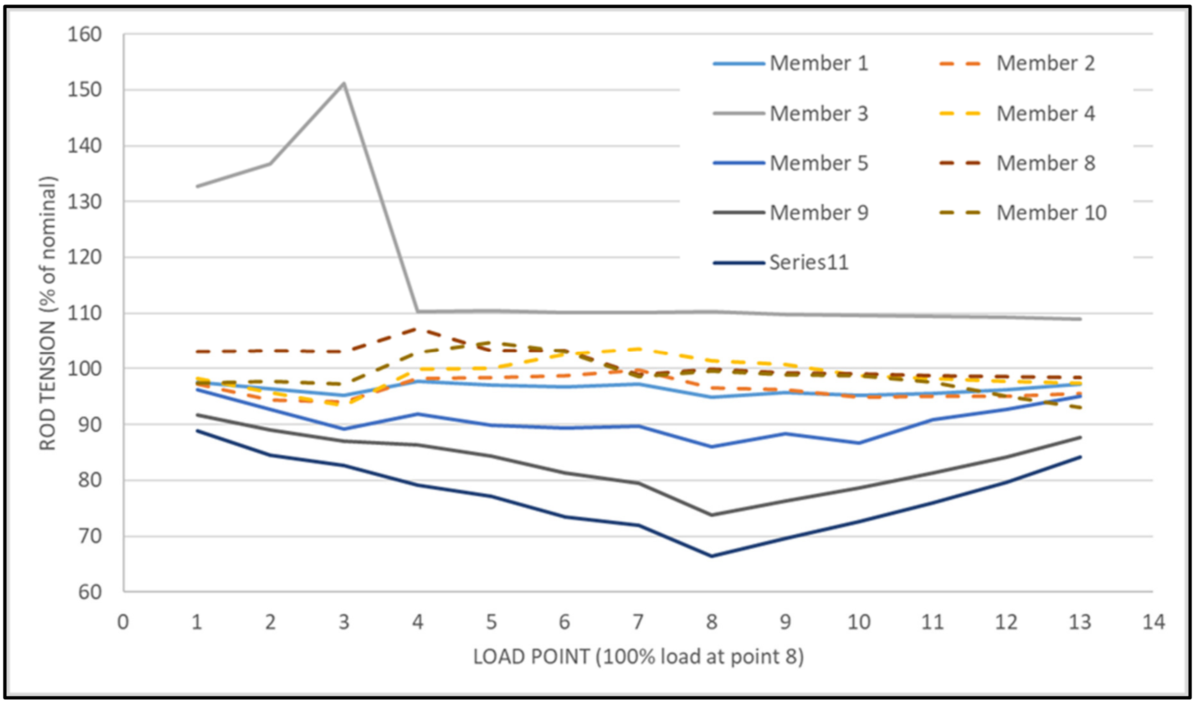

Figure 8. Rod tension over proof load cycle

As predicted by the FEA, the change in load in those members close to the centre of the span is small. The decrease in tension in rods 5, 9 and 11 was also predicted by the FEA (the associated members show an increased compressive load in figure 1). However, the FEA predicted a similar reduction in compressive load in the even numbered members that is not reflected in an increase in tensile load in the associated rods. This is to be expected when the science of bolted joints is considered [7].

4 Conclusions and Contributions

The deflection data collected is not discussed in the previous section. It was observed after the test program that this data was compromised due to permanent deformation of the timber sections used to level and support the bridge span. However, the elastic recovery recorded demonstrated that the vertical deflection of the span was 26 mm. The limiting deflection for a span of this length is 66.8 mm.

The review of the literature demonstrated that there is an urgent need for railway footbridges to replace level crossings on the UK rail network. Particularly within rural settings, it must be possible to install these bridges with minimal environmental damage. FRP bridges are seen as a good response to this demand as, unlike their steel equivalents, they can be installed with rail vehicle mounted lifting equipment. There is however a requirement to minimize the cost of any bridges installed.

A prototype bridge, fabricated from pultruded FRP sections and assembled using internal tension rods was demonstrated. Proof loading showed that this structure was able to withstand the required worst-case loads. Instrumentation installed within the bridge showed that the pre-load was maintained through the load cycle. This instrumentation can be used for continual structural health monitoring.

IBC's filled with water provide a convenient way of load testing this type of structure.

The results obtained demonstrated that pre-load can be affected by minor defects. This impact could have been alleviated by monitoring pre-load during the assembly process.

Further work required on the system includes:

- Additional verification of the design process by monitoring the compressive load in the diagonal members as well as the tension rods. This may allow the level of pre-load required to be reduced.
- Verification of the bridge deflection due to both dead and live loads by monitoring support deflection relative to ground in addition to mid-span deflection.
- The replacement of the wired load cell amplifiers with wireless alternatives. A challenge here will be providing power to the amplifiers..

Acknowledgements

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