Design of a Replacement Fibre-reinforced Polymer Footbridge

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Abstract. Saxe Street footbridge is a new glass Fibre-Reinforced Polymer (FRP) footbridge designed by WSP on behalf of Balfour Beatty on a Design & Build contract with Network Rail. The new FRP footbridge replaces a wrought iron iron footbridge built in 1884 that was suffering from extensive corrosion.

The new bridge carries a footpath over the Great Western Railway line in Teignmouth (UK) and consists of a single 12.7 m span simply supported deck made with pultruded FRP modular panels and connections and clad in moulded FRP panels. The design solution combined rapid and safe installation with a considerable reduction in future maintenance requirements that provided economic, operational and sustainability benefits.

This paper presents some of the challenges faced during the design stages. These challenges include understanding the effects of the material's behavior on the robustness and serviceability of the structure, together with its effects on the substructure and local site.

Site-specific challenges were encountered due to the lightweight nature and low stiffness of the new footbridge, linked to the structure's location over a railway and the stability of the existing substructure.

The new and old footbridges are shown in Fig. 1a and Fig. 1b respectively.

Keywords: Fibre-reinforced polymer · FRP · Footbridge · Design · Robustness · Lightweight · Buffeting · Sustainability

1 Introduction and Background

The new Saxe Street footbridge is a single span simply supported Fibre-Reinforced Polymer (FRP) deck presenting a clear span of 12.7m (Fig. 1a), which crosses the rail track west of Teignmouth train station, in Devon (UK).

The new footbridge replaces an old wrought iron structure (Fig. 1b), supported on stone masonry padstones and abutments, that was constructed in 1884 and consisted of two main wrought iron girders and nine transverse wrought iron cross girders, with wrought iron arches spanning longitudinally between them to support infill and surfacing.

The old bridge was affected by significant and extensive corrosion to the main girders, plate bearings and other parts of the superstructure, imposing an onerous ongoing

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a) b)

Fig. 1. Saxe Street footbridge, a) new replacement FRP footbridge, b) old wrought iron footbridge.

maintenance liability on Network Rail who owned the asset. The condition of the superstructure was deemed to be no longer acceptable and the decision to replace the existing wrought iron footbridge was taken.

Following the decision taken by Network Rail to replace the bridge, WSP were commissioned by Balfour Beatty, as part of a Design & Build contract, to provide consultancy services to deliver the detailed design of a replacement FRP bridge deck.

2 Decision to Adopt FRP Superstructure

The design of the new footbridge was influenced by a number of site-specific conditions and constraints. These conditions resulted in the choice to use FRP for the replacement superstructure, but also required careful consideration of the material's nature and behaviour.

The choice of an FRP structure was driven mainly by the material's low maintenance requirements and resistance to corrosion, which is particularly beneficial considering the exposure of this coastal site.

In addition, the bridge spans the Great Western Railway and the reduced maintenance requirements also decrease the need for frequent future track possessions, with a resulting positive impact on costs and health and safety (H&S) risks for the personnel on site.

Moreover, the choice of an FRP bridge assembled entirely off site and lifted in position has minimized disruptions to train services during construction and reduced significantly the activities to be carried out on site.

3 Site Constraints, Sustainability and Maintenance

The design of the new footbridge was subject to various site-specific constraints. This not only relates to the geometrical constraints due to the adjacent built environment but includes the interfaces with existing services routes running through the old bridge (drainage, cables) and the structure's location crossing a rail track.

Some examples of the considerations required during the design due to both the use of FRP in the footbridge and the site-specific conditions include:

- serviceability criteria – use of a lightweight material over a railway, and therefore susceptible to train buffeting effects;

- lack of material ductility – additional checks for robustness were crucial;
- fire and thermal effects – susceptible material sited over railway and therefore train exhaust fumes – potential fire risk and changes to the material's behavior (Tg) under heat;
- existing services – required consideration of required gradients, existing infrastructure and future maintenance access for the services; and
- effects of the new superstructure on existing substructure – removal of a heavier and stiffer superstructure, therefore removal of stabilizing and 'pinning' effects on abutments. Therefore, additional checks on excavation levels and backfill levels and crucial.

The presence of the sewer and drainage pipes required bespoke design solutions to allow for their future maintenance.

Another key driver in the design was to increase the minimum vertical clearance between the bridge soffit and the rail track, which not only increases the safety of the structure when subjected to the exhausts from trains, but considering the light weight nature of the FRP bridge also helps in reducing the aerodynamic effects of trains travelling below the bridge (train buffeting).

The design challenges caused by the unlikely event of a train stopping directly under the bridge, or of a fire taking place below or on the structure, have been analysed in order to avoid reaching, as far as reasonably practicable, the glass transition temperature (T_g) of the material by means of fire retardant resins and coatings and additional sacrificial plates.

As a consequence, an overall holistic design approach was key to ease future maintenance requirements, satisfy multiple design constraints and increase the safety and life span, ensuring the enhanced durability of every single asset affected by the bridge replacement, including services pipes, the north and south bridge accesses, vehicle access restrictions and the existing abutments.

4 Holistic Design Approach

4.1 Initial Design Concept

WSP's scope of work was to deliver the detailed design of a previously developed concept, which consisted of a design approach inspired by St Austell footbridge (Cornwall, UK), designed by Parsons Brinckerhoff (which has now become WSP) in 2007. St Austell footbridge was the first FRP structure ever built on the UK Rail network. The design concept, adapted to the Saxe Street site, comprises a U-frame configuration using Strongwell's Composolite interlocking modular pultruded FRP panels.

The initial design concept received by WSP, included numerous design assumptions. Through detailed design it became apparent that some of these assumptions required updating. The geometry of the initial proposed general arrangement was not sufficient to accommodate the sewer and drainage pipes within the bridge deck, due to a lack of clearance in the U-frame section, as the diameters assumed were significantly smaller than the actual pipes diameters.

Additionally, the design concept required changes that included provision for adequate access to the services and changes to the proposed bridge gradient in order to accommodate the existing drains invert levels at the North and South abutments (Fig. 2).

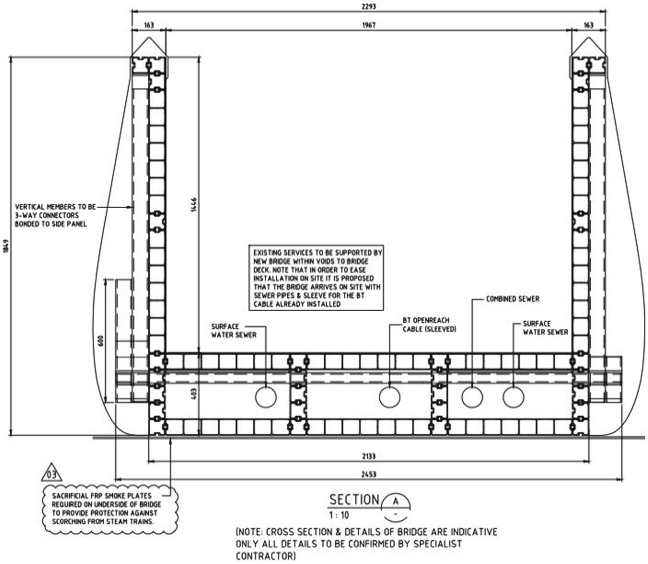


Fig. 2. Initial proposed general arrangement received by WSP.

4.2 Development of the Final Concept

The inaccuracies in the initial design assumptions required the services void within the proposed bridge deck cross section to be increased to a greater depth than the one initially assumed. The void has been designed with 50 mm gap above and below the pipes to provide tolerance and clearance for pipe seating brackets. Furthermore, to provide future maintenance access to services within the bridge deck, removable deck panels were specified over the services within the superstructure. These footway panels have been specified with a non-slip finish and with tamper-proof fixings to unauthorised removal. In addition, the weight of the removable footway panels was limited to approximately 20kg to enable manual lifting.

The proposed general arrangement of the replacement footbridge has been designed to achieve the required headroom clearance whilst avoiding changes to the service pipe invert levels at the north abutment. The increased headroom clearance requirement of 4890 mm has been achieved by raising the footbridge at the south abutment. This design rationale avoided costly changes to the upstream sewer and drainage network. The increased drainage and sewer levels at the south abutment have been accommodated by heightening the drop in the downstream manholes.

The U-frame section was maintained to provide sufficient stiffness to comply with deflection requirements using Strongwell's Composolite interlocking pultruded FRP panels. Following the rationale adopted for St Austell footbridge, transverse frames were placed at regular intervals of approximately 1.1 m to provide enough resistance to distortional buckling of the cross section.

The footbridge, which sits on elastomeric bearings, provides an internal clear width of approximately 1950 mm, a minimum parapet height of 1500 mm. Due to its relatively light weight (approximately 60kN) holding down bolts have been placed at the abutments supports to prevent uplift due wind and to train buffeting forces.

Sacrificial FRP panels have been bonded to the bridge soffit to protect the structural parts from the heat of the trains' exhausts. Additional non-structural FRP profile elements have been used to provide capping features and form the external moulded cladding. These provided both a more pleasing aesthetic whilst protecting the inner structural members from environmental exposure, hence allowing the use of less onerous partial safety factors (Fig. 3).

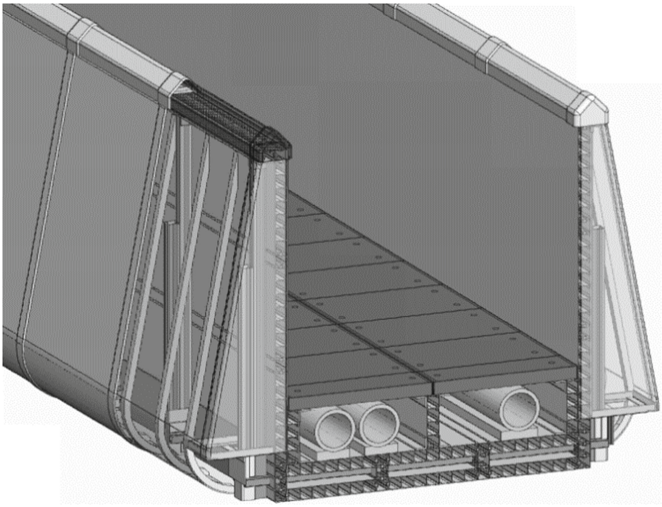


Fig. 3. Final proposed footbridge 3D view.

As anticipated some non-structural design decision have also been taken not only to improve the safety and the durability of the bridge, but also the overall durability of the items affected by the removal of the bridge. Vehicle access restriction has been improved using new bollards, staggered barriers and maximum gradients of the re-surfaced area adjacent to the bridge, have been designed in accordance with the requirements needed for disabled access. Surface water drainage has been enhanced using new kerb details and damp-proof levels in the new brick masonry walls. These details have all contributed to achieving a final product which satisfies the ongoing and future needs of the asset (Fig. 4).

Fig. 4. View of the new FRP bridge including the new vehicle access restraint system, barriers and the resurfaced area.

4.3 Effects of the FRP Structure on the Abutment Stability

The old wrought iron bridge had an estimated total weight of about 450kN, whilst the newly installed FRP footbridge only weighs approximately 60kN.

The existing stone masonry abutments were assessed to be in sound condition and were therefore retained. However, the significant weight decrease of the new bridge had an impact on the stability of the abutments, reducing the restoring effects provided from the superstructure.

Following stability checks it was deemed necessary to partially excavate the soil behind the abutments and reinstate using lightweight backfill once the bridge deck was installed, with geotextile membranes separating the various layers.

The use of an open excavation and of lightweight backfills up to a certain depth throughout the various construction stages, allowed to achieve the required level of safety for the abutment stability prior to bridge installation (open excavation with no backfill) and after bridge installation (both abutments backfilled). In the final layout, the lightweight backfill has been placed to a depth of approximately 1.5 to 2 m from the surfacing, ensuring a reduction of the lateral soil pressure.

Additionally, once the backfill was excavated prior to the removal of the old wrought iron bridge, the excavation revealed the relatively poor quality of the stone masonry in the back of the existing abutments. It also showed that the transverse section of the abutments was not compliant with the record drawings. This not only raised concerns on the reliability of the old construction drawings, but also on the resistance of the stone masonry and on the constructability of the new bearing locations. A precast concrete cill unit had to therefore be designed and placed on top of each abutment, and their shape had previously been designed based on the abutments' sections indicated on the old record drawings. An even surface was needed to place them in the correct locations and to achieve a better load distribution through the substructures. To enable the correct installation of the cill units, two reinforced concrete plinths (one at each abutment) have been cast against ground using a 100 mm blinding stratum and anchored to the abutments using 10 mm dia. stainless steel bars placed into pre-drilled holes that were injected prior to casting the plinths.

4.4 Robustness

The lack of ductility in the FRP material meant that care had to be taken in the design to ensure the structure was not disproportionately reliant on a localised member or joint. To check this, the design included analysis of various situations representing failures of joints along the U-frame and using the combination of actions for an accidental design situation (Fig. 5).

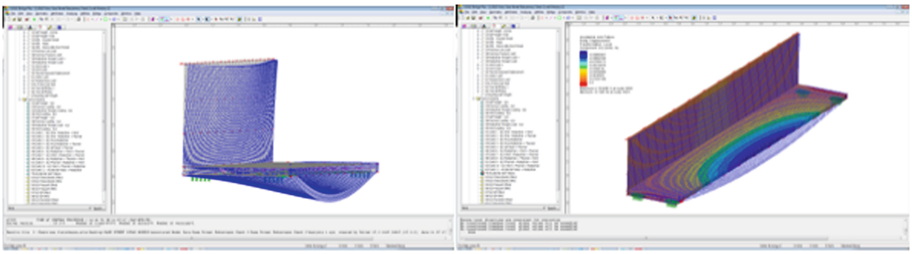


Fig. 5. Analysis model checking the effect of failure of various key joints.

The robustness checks confirmed that even accounting for the localised failure of key joints and connections, the structure performs satisfactorily.

5 Serviceability Vibrations

The serviceability vibrations checks carried out dealt with pedestrian and passing train induced vibrations. The low mass of the FRP bridge is an advantage for the handling and transportation of the assembled bridge, but also plays the opposite role in the two aforementioned checks.

5.1 Pedestrian Vibrations

The low mass of the bridge causes quite high fundamental frequencies for both the first lateral and the first vertical mode, making the pedestrian induced vibrations checks implicitly satisfied (Table 1).

Table 1. Most significant natural frequencies.

Description	Mode	Hz	% Mass
Lateral	1	4.9	43
Vertical	9	13.2	55

5.2 Train Buffeting Vibrations

The modal properties of the structure have been used in conjunction with a simplified analysis to check whether the response, in terms of acceleration, of the structure to trains' aerodynamic loadings was within the limits prescribed in the UK National Annex to BS EN 1991–2 for pedestrian comfort criteria.

Characteristic values of aerodynamic actions due to air pressure changes as a train passes a structure are recommended in EN 1991–2 clause 6.6. However, the uniformly distributed load (UDL) proposed in EN 1991–2 has been proved to be overly conservative for the UK as shown by Shave et al. (2009) who carried out testing on a footbridge over the railway at Goring (UK), with a headroom of 5.23 m. This study concluded that the loading associated with a single train travelling at 125 mph could be reasonably modelled by an upwards UDL of intensity 0.06kN/m^2 over a length of 5 m, followed immediately by a downwards UDL of the same intensity. This load model is based on the approach recommended in EN 1991–2 but with lower intensities by a factor of approximately 15.

For Saxe Street footbridge, the operating speed of the rail is only 60 mph and hence the values obtained in Shave et al. (2009) would be conservative, but the headroom for Saxe Street footbridge is 4.89 m. As a consequence, using the relationship in EN 1991–2 which buffeting loads inversely proportional to the headroom, the final characteristic buffeting pressure for a single train was increased to 0.08 kN/m^2.

For the simplified analysis, the width of the bridge, the UDL length and the train speed have been used to determine the period of the input. Once this was known, the aerodynamic force during time was modelled as a sinus wave, properly factored, and the response of a single degree of freedom system to a sinusoidal force in time was used to evaluate the maximum accelerations that always resulted to be within the limit of 0.7 m/s^2.

6 Testing

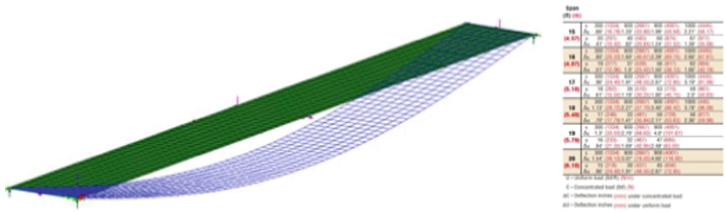
Static load testing of the superstructure was undertaken in the factory prior to transportation to site. The structure was supported on rubber bearings and the deck was load tested using water, filled to a depth of 500 mm, giving a total load of 13.0 tonnes; equivalent to a design serviceability loading of 5kPa. The purpose of the testing was to confirm that the fabricated structure was adequate for crowd loading and to verify the deflections as predicted by the analysis models.

Vertical deflections were recorded at various locations along the span and width of the structure and during loading and unloading at increments of 20% of the total load. The structure behaved as expected with a maximum mid-span deflection recorded under full serviceability footway loading of 7.51 mm. The allowable deflection under serviceability loading is 42.3 mm, so the deflections were well within the serviceability limits.

7 Comparison with FE Models

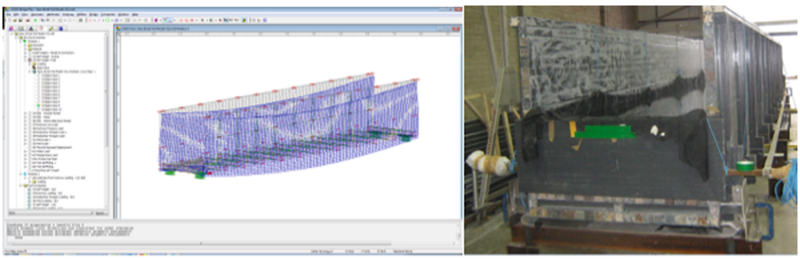
Verification checks were undertaken to compare the outputs from the analysis models with the true behaviour of the material and of the assembled structure (Figs. 6 and 7):

1. Comparison of modelled deflections against manufacturer's load-deflection data to verify material behaviour: shell element test models were built using specified stiffness properties. The geometry and assigned loads were applied to the test models in accordance with the load-deflection data. The analysed test model deflection results were very similar to the deflections as quoted in the load-deflection data, with differences of less than 1mm calculated under the geometrical and loading variances.
2. Comparison of the modelled deflections against the water load test results to verify the behaviour of structure: deflection results from the full structural analysis model were checked against the water load test results. The analysis model gave a maximum vertical defection of 9.0 mm under serviceability footway loading, compared against the water load test deflection value of 7.51 mm.



Span (m)	Applied Load (N / m)	Deflection quoted in Composolite load tables (mm)	Deflection from LUSAS analysis based on ACCS rigidities (mm)
4.57	875	31.5	31.2
5.18	773	45.8	45.5
6.10	437	48.6	49.3

Fig. 6. Analysis test model comparing analysed deflection values against load-deflection data.



a) b)

Fig. 7. a) FE model showing deflections, b) example of water load testing

8 Installation

The assembled bridge was installed by Balfour Beatty, as shown in Fig. 8, during a single night closure, hence limiting interruptions to the railway line. The bridge was lifted and placed on elastomeric bearings fixed to the precast concrete cill units. The bridge was then secured to the substructures through stainless steel fixings and brackets.

Fig. 8. Bridge installation and lifting operations

9 Conclusions

The design adopted for Saxe Street footbridge was strongly inspired by St Austell footbridge, but some substantial changes were made to address the needs of this specific project. Particularly the final design solution linked the efficiency of FRP as a structural material to its lightweight and durable nature and to allow for service pipes and their maintenance within the deck.

However, the lightweight nature of FRP resulted in some design challenges for the stability of the existing abutment and for the serviceability vibrations of the bridge. The analysis of the abutment stability and the subsequent backfill excavations exposed a relevant problem when dealing with dated assets: the reliability of record drawings. Particularly in this case the top of the abutments' cross section was not as represented in the record drawings and this caused delays and bespoke design solutions that needed to be quick, easy and cheap at the same time.

Guidelines and standards on the use of FRP are still not fully developed as of today, and this is one of the reasons why relatively high partial safety factors are required in the available documents and standards. The use of these higher partial factors, together with the specification of additional structural details to increase the structure's robustness, resulted in a conservative underestimation of the final stiffness of the bridge as shown by the results of the load testing, with the analysis resulting in approximately 20% higher deflections compared to the true, tested deflections.

The use of FRP as main structural material requires more effort and considerations from designers in terms of analysis and attention to details (to protect the material from external environmental conditions) but presents significant advantages for durability, construction and installation thereby improving the sustainability of the asset in its life cycle but also of the entire construction process.

Saxe Street footbridge therefore demonstrates that FRP can be effectively and successfully used as an alternative material for footbridges, achieving excellent structural and durability performances and meeting the user and asset owner's needs.

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