Interpretation of Sensor Data From In-Situ Tests on a Transversely Bonded FRP Road Bridge

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ABSTRACT

The Frampton Cotterell FRP road bridge deck comprises pultruded GFRP units which are laid longitudinally and are adhesively bonded transversely, in contrast to previous GFRP deck bridges where the pultruded units were laid transversely. This novel layout dictates that transverse distribution of live loading occurs only through the deck's flanges and entails possible transverse tension which should be controlled to avoid cracks through the bonded deck-deck joints. The present paper assesses these structural actions by interpreting strains and deflections recorded during lorry testing of the bridge. Transverse distribution is evaluated by comparing transverse profiles of recorded longitudinal strains and predicted longitudinal moments, with the conclusions qualitatively reinforced using a deflected surface based on the test recordings. Evidence of the deck acting as a continuum free of propagating joint cracks comes from the fact that the strains recorded during complementary lorry runs along the bridge satisfy the superposition principle, and that the recorded strain influence lines replicate an idiosyncratic feature of the moment influence line without redistribution effects. That feature was then exploited to inform the strategy for a braking test which produced valuable vibration data for the bridge. Test data integrity is corroborated by cross checking deflections recorded from different types of sensors. It is concluded that since longitudinal placement of pultruded decks enhances the versatility of FRP bridges, this sensor layout and data interpretation process may form part of a wider strategy for health monitoring of such bridges.

KEYWORDS : Field monitoring; Testing protocol; Sensors; FRP bridges; Data integrity

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1. INTRODUCTION

Carbon and glass fibre reinforced polymers (C/GFRPs) are highly durable materials of superior specific stiffness and strength that have transformed the service lives and structural efficiency of safety critical components in the aerospace, automotive and marine industries. These materials now show strong potential to underpin a similar transformation in the construction industry by enabling rapid installation (which reduces traffic delays [1]) of light, durable, low-maintenance road bridges. The high durability is crucial in the UK where roads account for 90% of passenger journeys and 70% of freight [2], and where a 17% increase in transported freight (to 1.97 billion tonnes) was registered over only a 12-month period ending in 2017 [3]. The rapid installation capability is crucial in the USA where 614, 387 bridges are at an average age of almost 43 years, near their 50-year design lives, and so will soon need replacing [4]. Wide use of FRP bridges can improve road network performance, which in turn ensures smooth functioning of the entire (communications, energy, transport, waste, water management) infrastructure system [5].

Multiple forms of FRP deck have been produced [6], but the technology is still maturing and so Mufti [7] has emphasised the role of structural health monitoring (SHM) to underpin acceptance of the changes in design and construction methods needed for FRP bridges. Such monitoring can influence decisions on maintenance, repair and rehabilitation [8], especially in highly stressed zones of in-service FRP bridges where brittle fracture of the FRPs should be guarded against [9]. In one case [10] monitoring led to discovery of significant temperature-induced movements of the FRP deck in service, while for another bridge [11] monitoring led to timely detection of cracks in the wearing surface and so to a successful repair strategy before the problem became pronounced.

More widely, Farhey [12] states that instrumentation and monitoring is the only tool that can enable reliable structural condition assessment and performance evaluation on which decisions, for example on bridge interventions, can be based. DeWolf et al. [13] argue (a point which motivated the work reported in the present paper) that in the absence of such monitoring data, it becomes necessary to define actual behaviour using conservative assumptions that can increase costs by introducing or expanding the scope of planned bridge interventions. Farhey later states that while SHM for bridge diagnostics seems to be accepted, it is not yet a typical field practice [14], owing to the perception of such monitoring as a time, labour, cost and logistically intensive activity [15].

Sridhar et al. [16] show how remote structural monitoring draws on interdisciplinarity between the fields of structural mechanics, sensors, statistics and online data transmission. Strain sensors are of fundamental importance to the monitoring process. De Freitas et al. [17] showed how short and long-term monitoring of an in-service orthotropic steel deck bridge was enhanced by use of such sensors, while Farreras-Alcover et al. [18] illustrated how potentially abnormal behaviour of a bridge may be detected by applying statistics to new strain monitoring data from the structure.

Further to the examples mentioned above, SHM via repeat testing and long-term monitoring has been applied to different forms of FRP road bridge. These include bridges with timber decks on FRP beams [19, 20], GFRP decking on FRP beams [21, 22, 23], sandwich FRP decks [24-28], FRP decks on steel beams [29-34], a hybrid FRP-concrete arch bridge [35] and a concrete deck on FRP beams [36]. Instrumentation comprised various combinations of strain gauges, deflection transducers, accelerometers and impact hammers. The tests and sensor data were used to :

- Quantify the transverse load distribution characteristics of the bridges;
- Proof-test the as-built bridges and inform on a strategy for future monitoring;
- Infer dynamic and global stiffness characteristics, including effects of deck-beam composite action;
- Validate both short- and long-term performance predictions for the bridges.
- Detect any structural changes within the bridges;
- Inform on any changes needed to existing standards, to expand their application to FRP bridges;
- Improve the designs of future FRP bridges.

In another novel use of SHM [37], the field data collected over four years from an in-service FRP bridge were used to calibrate accelerated laboratory durability test data. This enabled use of the extended lab data to assess deterioration rates for FRP decks over longer periods in practice.

2. RESEARCH OBJECTIVES OF THE PRESENT STUDY

Several of the bridges referred to above are pultruded GFRP deck-on-beam structures laid out so that the fully pultruded web and flange deck sections are effective in the transverse direction, across the main longitudinal beams. By way of example Fig. 1 [38] shows the view from below of the single lane traffic bridge above the M6, in the UK, within which the pultruded GFRP deck units span transversely across two main longitudinal steel beams. The transverse lines defining the base of the bonded joints between deck units can clearly be seen. Although this layout maximises transverse distribution of lorry loads between the main longitudinal beams for the given deck system, the above studies (e.g. [31]) show that this transverse load distribution capability is well below that for concrete slab bridges because GFRP has a low modulus, typically 12% that of steel.

Now in order to enhance a road bridge's versatility, for example to carry service pipes more efficiently along its span, it is preferable to lay the pultruded GFRP units longitudinally, with these juxtaposed units adhesively bonded to each other transversely. This approach facilitates placement of the units in vertically staggered horizontal planes to create natural channels for carrying the pipes. This layout can have two important consequences for structural action, as follows :

- Only the flanges of the GFRP deck in the horizontal direction normal to pultrusion, enable transverse load sharing. Also, the flange modulus in this normal direction is typically less than that parallel to pultrusion. This reduced deck section and the lower active flange modulus combine to further reduce the deck's transverse load distribution capability, relative to the layout with the deck units running transversely.
- Lorry loads often induce positive transverse moments in the deck, meaning transverse tensile stresses near the bases of the adhesively bonded joints between adjacent longitudinal deck units. It is crucial to limit these tensile stress so as to avoid initiation and propagation of cracks through those joints.

Largely because this deck layout is rare, to the best of the author's knowledge there are thus far no reported SHM studies of these two critical features for bridges with longitudinal GFRP pultrusions connected via transverse adhesive bonds. For the much more common case of the deck units laid transversely, deck-beam composite action along the span means that compressive – not tensile – stresses (this time in the longitudinal direction) develop across the bonded deck-deck joints. For these reasons the three key novelties of the present study are to :

- Apply SHM to a bridge comprising longitudinally oriented, transversely bonded GFRP pultrusions.
- Use the field strains and computer predictions to understand transverse load distribution in this bridge.
- Infer continuum deck behaviour - free of deck joint cracks - from the wider SHM data sets.

In working through these novelties the influence of the applied asphalt overlay on improving the deck's stiffness is discussed. Given the importance of the SHM data, a strategy to check data integrity is also provided. In what follows the bridge, short-term lorry tests, data collection and interpretation to the above ends are described.

3. DESCRIPTIONS OF BRIDGE, INSTRUMENTATION AND TESTS

3.1 Details of the Bridge

The bridge reported on in this study crosses the Frome river in the village of Frampton Cotterell, located approximately 120 miles due west of London, England. Fig. 2 presents the plan and cross section layouts of this GFRP deck bridge, which replaces a concrete bridge that had reached the end of its working life at the same location. Owing to its low weight, this bridge uses the same supports as did its concrete predecessor. As shown in Fig. 2(a), this single span simply supported bridge is 8.7 m long, 11.99 m wide and runs in a roughly east-west direction. It carries two lanes of contraflow traffic, flanked by walkways along both edges.

Fig. 2(a) indicates a section A-A at midspan of the bridge. This cross section, shown in Fig. 2(b), reveals that the central 7.28 m wide carriageway is a four-layer structural system in which each layer – including each layer made up of the triangulated ASSET pultruded GFRP deck units - is oriented longitudinally and is adhesively bonded to its immediately adjacent vertical neighbour(s). This longitudinal layout of the units enabled the carriageway and walkway layers of the deck to be staggered vertically, creating channels within which the service pipes can be carried across the river under the walkways. This is a key benefit of the present bridge form. In the alternative, more widely used layout defined by the deck spanning across the longitudinal beams, such channels are more difficult to realise and instead the service pipes must be suspended from brackets anchored to the deck's lower flange, requiring extra features such as holes drilled through the flange.

In Fig. 2(b) it is seen that at each edge of the section, three ASSET triangles are laid onto the downstand, at the same level as the carriageway. Not shown are the soft stone parapets which have been laid onto the resulting edge double-layers of ASSET. The parapets are connected to these ASSET units by vertical threaded steel rods which are anchored within the deck units by grouting and within the parapet via intimate contact with the lime mortar used to bind the soft stones together. These vertical anchor rods are spaced at 1 m centres along the length of the structure. Once the service pipes had been laid during construction of the bridge, the channels evident in Fig. 2(b) were covered to provide the walkways. Note, in Details A and C of Fig. 2(c), the further use of multi-layer CFRP plates bonded to the soffit and / or top of the deck as structural enhancements to the double layers of ASSET.

As shown in the middle diagram of Fig. 2(c), from top to base the longitudinal layers comprise 0.5 m wide, 10 mm thick GFRP plates laid side by side, then pultruded, double-triangulated ASSET

GFRP units, then GFRP square hollow section (SHS) girders (or beams) spaced at just over 0.9 m centres transversely, and finally 20 mm thick, uni-directional, multi-layer CFRP strips bonded to the soffits of the SHS girders. Henceforth the terms girder and beam are used interchangeably in this paper. Each 20 mm thick strip comprises four thinner strips, each of 5 mm thickness, stacked vertically and bonded to each other. These strips exploit the higher modulus and rupture stress of CFRP to stiffen and possibly strengthen the deck. In order to avoid congestion on the diagram, the 100 mm thick asphalt overlay on the top 10 mm thick GFRP plating layer is not shown. So as to facilitate later discussion, the SHS girders are labelled FG1 to FG7 in proceeding from the north to the south kerb in Fig. 2(b).

Henceforth, the term T-beam will be used to describe the hybrid longitudinal beam of T-section formed by any SHS member, the 20 mm thick CFRP strip bonded to that SHS, the overhead ASSET units lying between the halfway points to the nearest SHS neighbours, and the 10 mm thick top layer GFRP plate bonded to these ASSET units. Detail B in Fig. 2(c) shows the key features of this T-section, except that the total number of ASSET triangles will be six or seven, almost symmetrically distributed about the mid-width of the SHS. As a point of interest, note that ASSET is the acronym coined for this pultruded GFRP deck system at its inception (circa 2000) from the term "Advanced Structural SystEms for Tomorrow's Infrastructure".

Now, as explained earlier, transverse distribution of tyre loads between these T-beams occurs via only the upper and lower ASSET flanges acting together in flexure in the vertical plane, normal to the direction of pultrusion. Importantly, this requires transverse moment continuity in the deck across the bonded joints between the longitudinal ASSET units. That feature is a clear distinction from most other pultruded FRP deck bridges, where the deck units run transversely across the main girders, and so the bending stiffness of the full, continuous deck section in the direction of pultrusion is exploited to enable load distribution in the transverse direction of the bridge. Given that the elastic modulus of, vertical separation between and thickness of the ASSET flanges are all modest, it is likely that the relatively thick asphalt overlay (if properly bonded to the top layer GFRP plates) could improve this transverse flexural stiffness of the deck (despite the low modulus of asphalt) and so could potentially influence transverse load distribution. Well bonded asphalt would also improve the bending stiffness of each T-beam, and so might reduce deflections under lorry loads. These issues will be raised later, in discussing the physical significance of the test results.

As shown in Fig. 2(d), the ASSET unit is a double triangle in section. Hence the three-triangle arrangement used to support the parapet at each edge of the deck (Fig. 2(b)) entailed lopping off a triangle from one unit. More widely across the deck, bonding of any deck unit to its neighbours was done along the entire lengths of the unit's lips, grooves and inclined edge webs, all evident in Fig. 2(d). That good bond integrity translates into ample transverse moment continuity between units is of key importance.

3.2 Installation of the Structure

The entire bridge deck as shown in Fig. 2(b) was fabricated within six miles of the bridge site, at the National Composites Centre, where all four (GFRP plate, ASSET, SHS and CFRP strip) layers described above were bonded together. Fig. 3(a) and (b) show, respectively, the completed deck being transported by lorry from the fabrication facility to the site and in-situ craning of the deck onto the abutments. Fig. 3(b) affords good views of the cross-sectional layout of the deck as described above and shown in Fig. 2(b), along with a clear view of the uni-directional, multi-layer CFRP strips bonded to the undersides of the SHS girders. Note also from Fig. 3(b) that the ends of the hollow units have been sealed off by grouting, to inhibit moisture ingress while the bridge is in service. The grout extends up to 0.4 m inwards along each hollow section.

Also in Fig. 3(b), note the temporary timber deck laid at an elevation near the tops of the abutments, and which extended in plan across the full width and length of the bridge. This timber deck was supported by a network of steel scaffolding poles (hidden from view in the photo) resting on the riverbed underneath. Once the FRP deck had been craned onto the bearings, the timber deck was used as a working platform from which the strain gauges used in the present study were mounted onto the underside of the bridge deck on the CFRP strips, the ASSET units and the SHS units all at midspan. While this instrumentation of the bridge took place from underneath the deck, other activities occurred above the deck such as laying of the asphalt surfacing, construction of the stone parapets and assembly of the service pipes in the walkway channels. Following on from the time-efficient prefabrication and craning operations, this strategy of synchronising works above and below the deck led to a rapid bridge installation scheme. This in turn enabled quick removal of a 7 km traffic diversion (a key benefit of using GFRP deck bridges) needed during the bridgeworks.

Fig. 3(c) shows a side view of the completed bridge over the river, showing clearly one of the stone parapets on the double-layer ASSET edge-stiffening. The soft stone has been re-used from the predecessor concrete bridge's parapets. Note the visual continuity of the parapet wall with the walls extending along the road at both ends of the bridge.

3.3 Instrumentation Layout

Using a high-speed data logger, both deflections and strains were recorded from the bridge during the lorry tests. To those ends, electrical resistance strain gauges were placed on the soffits of the CFRP strips bonded to SHS girders FG4, FG5, FG6 and FG7 (Fig. 2(a), (b)), so that one half of the structure in cross section was gauged. All of these strip gauges were located at midspan of the bridge, also were oriented to measure longitudinal strains, and were protected with small dollops of silicone gel. The leads from all gauges have been bundled within conduits under the bridge and routed to a central protective housing installed on the side of the road near the bridge. Recall, from Section 3.2, that this gauging and cabling occurred after the bridge was placed onto the supports. Thus, unfortunately, the dead load strains could not have been registered. Within the housing, the gauge leads terminate at plugs which enable connection to the data logger. Hence as part of the preparation for the lorry tests, this housing was accessed to provide continuity between the gauges and the logger.

Now while the strain gauges are permanently attached to the bridge, the displacement – measuring sensors had to be installed specifically for the test. To that end, potentiometers, or POTs, were set up on poles from underneath the deck, to measure lorry-induced displacements of the bridge. Each POT was a model SLS190/0050/L/66/10 sensor, manufactured by Penny and Giles, and a stated accuracy of ±0.5% within the full-scale reading. This POT was a contact sensor, with its operation relying on the tip of a spring-loaded plunger bearing gently against and moving with the soffit of the deck.

The vertical POTs were distributed to form a 2D array of measurement points in plan, labelled POT1 to POT9 inclusive in Fig. 2(a). The idealised, target layout was a rectangular 3 x 3 grid of deflection measurement points along lines parallel to the length and width of the structure, with each longitudinal line containing POTs at midspan and both quarter span locations. This would have included POTs along both the longitudinal and transverse centrelines of the bridge in plan, also along longitudinal lines coincident with either an edge girder (FG1 or FG7 in Fig. 2(a)) or a kerb line. In the event, the local roughness of the river bed under the bridge precluded the placement of the pole-holders for the POTs at these idealised locations. A sense of this roughness can be gained from Fig. 4(a), which shows placement of the poles on the river bed in progress. The metal plate bases of the poles were firmly pushed onto the riverbed, to ensure that they would remain properly seated and so were unlikely to allow movement of the poles for the duration of the tests.

Hence the final distribution of POTs contained perturbations to the above idealised layout. As seen in Fig. 2(a), the POTs were arranged in a mildly higgledy-piggledy manner in plan, with POTs 7, 8 and 9 having been the only trio to actually lie along a single longitudinal line (the south kerb line). Otherwise, two pairs of the POTs lay along SHS girders, namely POTs 4 and 6 along FG5 and POTs 1 and 2 along FG1. Despite these perturbations, the idea of a 2D array of deflection measurement points was retained, thereby enabling a deflected surface for the structure under any lorry loads to be obtained by application of surface-fitting algorithms to the data.

Now prior to use in the tests the POTs were individually calibrated. As a further check on the integrity of the deflection data from these POTs, which are contact sensors, a decision was made to also use non-contact sensors based on laser technology, to independently quantify deflections at POT locations 1, 4, 5 and 6 in Fig. 2(a). Each laser device was a model ILR 1181 sensor, manufactured by Micro-Epsilon, and with a stated capability of measuring to 0.1 mm accuracy. These laser sensors were supported on the same poles as were their immediate-neighbour POT sensors.

By the same token, some of the strain gauge readings could have been double-checked by complementing these gauges with fibre Bragg gratings. However this was beyond the scope of the present project. Moreover, the principle of using two different types of sensors as the basis of assessing the reliability of recorded data for a given quantity is already tested via the POT and laser transducers.

3.4 In-Situ Lorry Testing Strategy

Two salt-gritting lorries belonging to South Gloucestershire Council, the owners of the bridge, were used for in-situ testing. Each lorry has three axles, with a single tyre at each end of the front axle and twin tyres at the ends of both rear axles. The container of each lorry was filled with grit, giving a static gross weight (measured by weighbridge) just shy of 250 kN for each lorry. Fig. 4(b) shows one of the lorries being driven along the bridge such that it straddled the structure's longitudinal centreline. This image shows that the test was conducted at night. In fact, the entire set of lorry tests continued from late one Saturday night into the early hours of the following Sunday morning. This minimised inconvenience to road users from the tests, and also minimised traffic management needs during the tests. Fig 4(c) shows the plan layout of and loads in elevation through the axles. It is seen that the front axle load, at 66.5 kN, is almost three quarters of that (91.3 kN) carried by each rear axle.

Different tests were conducted in which the lorries were driven, either singly or as a pair, along the bridge at various speeds and along different lines. The tests are described as follows.

- To start off with, one lorry was driven at crawling pace near the south kerb, specifically along track A in Fig. 4(d), to establish the lorry location along this line at which maximum strain was recorded from any of the midspan CFRP strip gauges. This happened when the lorry's first rear axle was effectively at midspan, with the front axle of the lorry on the verge of stepping off the far end of the bridge. In subsequent static tests that longitudinal location of the lorry was retained, while the transverse location of the lorry was shifted to, and held stationary at, in turn, each of tracks B to G in Fig. 4(d). This enabled collection of influence line data for strains and deflections. Note that A and G are transversely symmetric with respect to the middle longitudinal girder FG4. Ditto B and F. Moreover, the tyre loads for C are transversely symmetric about FG4.
- One lorry was driven at crawling pace first in the middle of each lane, then as in Fig. 4(b), namely straddling the bridge's longitudinal centre-line.
- The lorries were then driven at 30 mph along the bridge as follows, namely :
    - In series, along the centre of one traffic lane.
    - Individually, along the centres of both traffic lanes.
    - In parallel, along the centres of both traffic lanes.

The above-described second and third set of tests at 30 mph were deliberately pursued to enable use of superposition as an important check on the behaviour of the bridge as a continuum free of propagating transverse cracks. For any given strain gauge, for example, the superposition of strain influence lines for the lorries running individually along the adjacent lanes in the same direction should equate to the strain influence line for the lorries running in parallel along the two lanes. During each test the data from all strain and displacement sensors were recorded at 1 kHz. Some lorry runs were repeated to enable later checks on consistency of the recorded data.

4. Transverse Load Distribution Characteristics

For the longitudinal location of one lorry which gave peak moment at the midspan (strain gauged) section of the bridge, Fig. 5 shows the recorded strain profile across beams FG4 – FG7 inclusive for each of lorry tracks C and F as defined in Fig. 4(d). For Track C the strains at FG4 and FG5 are similar because the axle width covers both locations, but beyond FG5 the sharp transverse drop-off of longitudinal strains is evident, with over 60% fall in strain only two beams away to FG7. For Track F, there is a monotonic and sharp drop-off in strain from FG4 to FG7 because the lorry is now near the remote kerb transversely across from FG4 to FG7.

In order to predict the T-beam moments due to the associated transverse load distributions, a grillage model of the deck was built in the software GSA. The model is shown in Fig. 6(a). Line beam members were placed along the centrelines in plan of each girder FG1 - FG7. Each such member was assigned properties of the hybrid T-beam comprising at least the SHS, bonded CFRP multi-layer strip, ASSET profiles halfway between its nearest neighbours and top layer GFRP plate. Since the transverse spacing of the SHS members is low relative to the span of the bridge, zero shear lag effect was assumed in determining the effective ASSET section "flange" width for use in each T-beam. The approach to and including each walkway was divided into longitudinal members representing the double-layer ASSET zones (e.g. the edge supporting the stone parapet, with or without the parapet included) and the single layer ASSET zones in-between (e.g. under the walkway). Transverse elements comprised at least the top and base flanges of ASSET in the direction normal to pultrusion. As Fig. 6(a) shows, the vertical offsets between the centroids of the main carriageway and sub-walkway elements were allowed for using stub elements of appropriate heights. Pin and roller supports were assumed at the ends of the bridge.

In order to establish the potential influences of the asphalt overlay and the stone parapets, three different grillage models were used, as follows :

- One using only the properties of the FRP elements as described above.
- Another assuming full composite action between the 100 mm thick asphalt surfacing layer and the deck in both the longitudinal and transverse directions. In order to follow reality, the asphalt was assumed only across the width of the carriageway.
- A third also assuming full composite action in the longitudinal direction between the stone parapets and the double layer edge ASSET members.

Table 1 presents the material moduli assumed in calculating the line beam section properties for these analyses. For the parapet, Fig. 3(c) shows that quite thick mortar layers were used between the soft stone blocks. Hence the elastic modulus used for the parapet material assumes a soft stone-mortar composite. The FRP material properties are manufacturer's data, while the asphalt and stone-mortar composite data were estimated from the literature [39, 40]. As the tests were conducted at night when temperatures were reduced, the asphalt modulus was taken near the higher end of the range specified for this material. On a hot day this would ofcourse be liable to drop.

For the peak moments due to the lorry along tracks C and F, Fig. 6(b), (c) show the midspan moments carried by the longitudinal members across the width of the structure. The distinct drops in moment response away from the loads broadly reflect the sharp drops in strain response evident from the plots of Fig. 5. For the transversely symmetric load layout on Track C, Fig. 6(b) shows that the three middle T-beams (FG3, 4 and 5) carry the highest moments, almost equal to each other. Outwards from this trio the moments drop significantly, owing to the deck's low transverse load distribution capability. For example, the moment carried by FG5 exceeds that carried by its neighbour FG6 by almost 45%, with continued transverse drop-off to zero moment carried by the edge parapet members. Note also that the predicted load sharing is insensitive to the presence or absence of asphalt and parapet stiffness contributions. For Track F, almost in the centre of the north lane, Fig. 6(c) again shows highly uneven load sharing. In this case the deck's low transverse stiffness allows the nearby edge member to carry a modicum of moment. Note the palpable increase in this moment when the stiffening effect of the stone parapet is incorporated within the model. This allowance for the parapet is also seen to introduce a kink on the moment-sharing diagram, the reason for which is not clear. In all cases the more distant edge member carried virtually zero moment.

It is instructive to compare the transverse load distributions implicit in the recorded strain profiles of Fig. 5 with those from the corresponding moment profiles of Fig. 6. This is possible if the strains in each Fig. 5 plot are normalised with respect to the peak value on the plot in question, ditto the moments on each Fig. 6 plot. Fig. 7 compares these normalised strain and moment profiles. On this plot C and F refer to the lorry tracks, while M and S refer to moment and strain. Hence CS refers to the strains recorded for the lorry on track C, while the FM refers to the predicted moments for the lorry on track F. By comparison with Fig. 5 it is seen that for recorded strains above 55$\mu\varepsilon$ the correlation between normalised strain and moment profiles in Fig. 7 is good, while there is increasing divergence between these profiles for strains below that 55$\mu\varepsilon$ threshold. The good correlations for most of the Track C plots and for the upper part of the Track F plots indicate the value of grillage analysis, a popular tool with designers, in identifying transverse load distribution behaviour.

The divergences at lower strain levels may be due to the greater impact of errors in the recorded strains, also to the presence of axial force strains in the readings (the net axial force on the entire section must be zero, but the individual T-beams may develop axial forces), also to improvements possibly needed to the grillage model in material properties, in the way that it represents the biaxial structural action of the deck and in the representation of the axle loads.

An important statistic from Fig. 7 is that for both near and far lorry loads (with respect to the gauged beams FG4-FG7), for this specific bridge, the recorded strains suggest average transverse drop-off rates away from the load of very nearly 30% between adjacent T-beams. This is quite significant and can have important implications for global fatigue of the bridge.

Further evidence of the low transverse load distribution capability comes from the deflected surface of the deck as recorded under lorry loading. Fig. 8 shows this deflected surface for the maximum midspan moment location of the lorry (the first of the two rear axles at midspan) along track C. This plot has been achieved by applying a two-way high degree polynomial-fitting routine to the raw data from the nine POTs, while assuming zero deflection at both supports of the structure. For clarity, the direction of the road is included on the Figure. This deflected surface suggests that while the carriageway deflected significantly, the walkways did not. Indeed, a striking feature of Fig. 8 is the indication of almost zero deflection along the edges of the structure parallel to the road. On their own these small deflections could have been the result of a high local stiffness of the double ASSET layer and stone edge parapets. However coupled to the information above the origin is the low transverse stiffness of the deck. This ready back-up inference of transverse load distribution characteristics is a useful feature of the deflected surface.

5. Evidence of Uncracked Continuum Behaviour

That the deck behaved during the tests as a continuum, free of undesirable levels of progressive transverse cracking at the bonded joints between pultruded units, may be deduced from multiple facets of the recorded data as shown in the ensuing sections. This starts with comparisons between the recorded and predicted deflections, the latter output from the continuum analysis grillage model. It then continues with checks that various subsets of the test data satisfy different requirements of continuum mechanics.

5.1 Predicted and Recorded Midspan Deflections

Using the grillage models described in the previous section, the predicted and recorded deflections at POT5 are compared in Table 2 for the lorry along each of tracks C and F. At each location, the lorry was located (Fig. 6(a)) to induce maximum moment across the midspan section of the bridge.

Table 2 shows that, for the lorry on track C, inclusion of the asphalt leads to a 33% drop in predicted POT5 deflection, from 7.1 mm to 4.7 mm. Subsequent inclusion of the parapets has no further effect on the predicted deflection, which again suggests limited transverse distribution of the load effects across to the parapets. With all the structural elements included in the model, the predicted 4.7 mm deflection exceeds the measured value by 31%, compared to 97% when only the FRP components are included.

The 31% disparity may lie in factors such as the assumed material properties and the support boundary conditions (the steel bars cantilevering out from the abutment into the grouted ends of the deck may have introduced some rotational restraint to the bridge at its ends). The overall trend is similar for the lorry on track F, except that the asphalt leads to 69% reduced deflection from the model, while the parapet this time leads to a further 4% reduced deflection. It may be that the closer proximity of track F to one of the parapets leads to transmission of the load effects to that nearby parapet.

Since the grillage model is of a continuum nature, this close gravitation of the predicted deflections to the recorded values is evidence of the validity of the deck acting free of cracks between pultruded units. This is backed up by the further continuum checks on the strain data as below.

5.2 Influence Line Checks on Strain Data

For the lorry travelling at 30 mph nominally in the middle of the south lane, Fig. 9(a) shows the influence line for the strain recorded from the soffit of the CFRP strip at midspan of FG5. The line is presented with respect to the lorry's front axle location (lower horizontal axis) and to recorded time (upper horizontal axis). Axle location, namely the distance (m) that the front axle has rolled along the bridge from the starting support, was calculated as the product of lorry speed, converted from mph to m/s, and time in seconds. It is shown later in this section that the use of a 30 mph speed was quite reliable.

In Fig. 9(a), the overall pattern of a strain increase associated with the lorry advancing along the bridge, followed by a drop as the lorry tends towards exiting the bridge looks reasonable. However, during the initial increase, there is a distinct and unexpected drop in the strain with lorry advance, between the points labelled M2′ and M3′ on the plot.

The explanation lies in Fig. 9(b), which shows the influence line for midspan moment with respect to front axle location. The regime on this plot of current interest extends from the origin to point M3. The labels M2 and M3 on this plot correspond to M2′ and M3′, respectively, on Fig. 9(a). At the origin, the front axle had just rolled onto the bridge. Between the origin and M1, only the front axle was on the bridge, inducing significant midspan moment increases as it rolled along. At M1 the first rear axle rolled onto the bridge while the front axle had closely approached, but had not yet reached midspan. Hence, from M1 onwards the two axles reinforced each other to increase midspan moments as they both advanced along the bridge, as long as they both remained on the same side of midspan. This explains the increased gradient of the M1-M2 regime relative to the origin-M1 regime. At M2, the front axle reached midspan. This threshold layout of axles relative to the bridge is illustrated in Fig. 9(c).

From then on the front axle's advance was away from midspan, which on its own led to *decrease* of the midspan moment, while the first rear axle's advance was still towards midspan, which alone continued to *increase* the midspan moment, bearing in mind that the last axle had not yet rolled onto the bridge. Hence beyond M2 the increment of midspan moment was a function of the *difference* between the two axle loads, rather than of the sum of those loads as in the M1-M2 regime. This led to a sharp drop in gradient of the moment influence line in proceeding from the M1-M2 regime to the M2-M3 regime. At M3, the lorry's final axle rolled onto the bridge.

It is this sharp drop in gradient of the moment influence line which is responsible for the dip defining the corresponding M2′ - M3′ regime of the strain influence line in Fig. 9(a). Now the abscissae of M2 and M3 on the moment influence line Fig. 9(b) are 4.35 m and 5.25 m, respectively. It is evident from Fig. 9(a) that these are reasonably well matched by the abscissae of points M2′ and M3′ on the strain influence line. This coincidence of abscissae renders the moment-based explanation for the dip on the strain influence line quite striking, and it also points to the reasonable nature of the 30 mph assumption used to locate the lorry's front axle in Fig. 9(a). Very significantly, that the strain influence line mimics this peculiarity of the moment influence line is further evidence of the continuum nature of the deck. Had any transverse cracks existed across and propagated through the bonded joints between longitudinal pultruded units during the tests, then depending on their locations these cracks might have alternately closed and opened as a function of lorry location along the bridge, causing the strain influence line to deviate from the moment influence line.

More widely, it is instructive to compare salient points on the strain and moment influence lines. To that end Fig. 9(b) also shows the peak point M4 on the moment influence line, which occurred when the first rear axle was at midspan. On Fig. 9(d), the moment influence line points M2 and M4 are compared in normalised form to the first (M2′) and second peak points on the strain influence line. On the horizontal axis distance is normalised with respect to the span of the bridge, while on the vertical axis both strain and moment are normalised with respect to their peak values on the plots of Fig. 9(a), (b), respectively. Recall that the moment data points were derived purely by applying *equilibrium* considerations to the lorry *loads*, while the strain points were derived by the independent process of applying *kinematic* considerations based on a 30 mph lorry speed. Now on Fig. 9(d) it is seen that, at each of M2 and M4, the strain and moment points are almost coincident. This has two key implications, namely that :

- The assumption of a 30 mph lorry speed along the bridge is a good approximation (similarity of abscissae).
- The recorded strain was directly proportional to the midspan moment (similarity of ordinates) as the lorry rolled along the bridge.

It must be emphasised that the M2-M3 regime of almost constant midspan moment exists because the distance between the first two axles of the lorry is only marginally less than half the span of the bridge. Had the bridge been longer such that all three axles could have been present on the structure *before* the front axle rolled across midspan, then the low gradient zone may quite likely not have been a feature of the moment influence line.

5.3 Superposition Checks on Strain Data

Figures 10 present the strains recorded from the soffit CFRP plating of SHS girder FG4 during some of the tests in which the lorry was driven at 30 mph along the bridge. In Fig. 10 it was considered physically meaningful to use both time and the lorry's front axle location along the bridge as the horizontal axis variables. This can partly imply an influence line for strain with respect to lorry location along the bridge. To that end the front axle location has been calculated as the product of lorry speed and time after the lorry stepped onto the bridge. Henceforth, the terms strain influence line and strain vs axle location plot are used synonymously in this paper.

In Fig. 10(a), the strain as a function of front axle location is presented for three different runs of the lorries, namely one lorry along the centre of the north lane (very near track F in Fig. 4(d)), then one lorry along the centre of the south lane (very near track B in Fig. 4(d)), and finally both lorries in parallel along the centres of both lanes. The single lorry strain profiles are seen to be quite similar to each other, though not identical. Given that FG4 is along a nominal line of symmetry of the structure, this slight deviation between the single lorry plots probably reflects small differences in loads between lorries, as well as the lorries travelling along tracks which differed to small extents from each other during the three tests.

In Fig. 10(b), the strain vs axle location profile obtained by superposing the single lorry results from Fig. 10(a) closely matches that from the parallel lorry run. This consistency with the superposition principle is an important indicator of reliability of the data and, taken along with the other evidence provided above, an indicator of the absence of propagating cracks in the deck during the tests. It further strongly suggests consistency of lorry speed between the three tests. Along with the lorry driver's observations of the speedometer during the test, the proposal that 30 mph is a close approximation to this speed comes from the fact that the peak points on the strain influence lines lie near that on the moment influence line of Fig. 9(b).

6. CONTACT AND NON-CONTACT SENSOR DEFLECTION RECORDINGS

Note that pre-test calibrations were performed for the non-contact (laser) and contact (spring-loaded plunger) deflection sensors. As a further check on the integrity of the deflection recordings, it is instructive to compare these recordings from both sets of sensors during the tests as functions of time while one lorry crawled onto and off the bridge, say along each of tracks E and F in turn.

This is done for POT5, which was the nearest deflection measurement location to the centre of the bridge in plan. Lorry movement along track E led to among the highest recorded deflections at POT 5. Movement along track F reduced the POT 5 recordings and so enabled the sensor agreements to be checked when lower peak deflections were sustained over time as the lorry sat at the most onerous location along that line. Fig. 11(a) shows the plots for the lorry on track F, for which a maximum deflection of only about 2.6 mm was registered. On the left graph the four key stages of the test have been identified, starting with the lorry's approach to the bridge, followed by the lorry being manoeuvred toward the desired location on the bridge, then with the lorry stationary at the desired location (recall this was with the first rear axle at midspan) for approximately one minute, after which the lorry was driven off the bridge.

Now on this left graph of Fig. 11(a), the laser data as presented has already been filtered to delete all frequency content above 45 Hz. Despite this, it is seen that while the plot for the contact sensor is a thin, well-defined line, the laser plot has a distinct bandwidth. It is evident by eye, though, that the mid-width trajectory of the laser plot is quite close to the contact sensor's line. This is confirmed in the right graph of Fig. 11(a), which has been obtained by taking the averages of each set of two hundred consecutive points along the laser plot on the left graph of Fig. 11(a). That process has almost collapsed the wide strip of Fig. 11(a) onto a line which agrees well with the contact sensor plot. The time period between 120 seconds and 150 seconds inclusive shows the least best agreement between the two data sets. Within this period, the average ratio of laser-to-non contact sensor deflections is 1.07, with a CoV of 0.025, both indications of good agreement.

Note the short, but palpable dip labelled on the right graph of Fig. 11(a) at the conclusion of the manoeuvring process. This dip signified a temporary drop in deflection. It probably arose from momentary load reductions on the rear axles (then located near midspan) associated with braking of the lorry to stop the vehicle at its desired location on the bridge. The compensating increase in load on the front axle would have had little effect at midspan, owing to the remoteness of that front axle located at the far end of the bridge. A close look at the contact sensor plot within the dip suggests small vibrations of the bridge, as might be expected from the oscillating loads due to local vibrations of the lorry as it was quickly brought to rest. Note also the slight increase in deflection while the lorry was stationary on the bridge, the origin of which isn't clear except for possibly some form of "rapid creep" (a contradiction in terms) within the structure.

Fig. 11(b) shows the results for the lorry travelling along line E, for which a peak deflection of about 3.6 mm was registered. In this case, the consistency of the maximum deflection reading was checked by repeating the test, with the lorry reversed almost to the start of the bridge between two consecutive attempts at the test. The left graph of Fig. 11(b) is labelled from above to indicate the time period of this occurrence. The observations made from Fig. 11(a) apply here too, with even better agreement between contact and non-contact sensor readings near and at peak deflection. It is seen that the agreement extends to periods of rapid deflection change, for example when the lorry was driven onto and (in particular) off the bridge.

In future tests the contact and non-contact sensors could be placed on different poles, as a further check on admissibility of the readings by eliminating inaccuracies stemming from the pole setup itself. Given the finite sizes needed for the pole bases, and hence the different locations of these bases, the poles would themselves require bends higher up to ensure the different types of sensors were targeting as near as possible the same spot on the soffit of the bridge deck. Meanwhile, the good correlation between the grillage predicted and recorded deflections as indicated earlier is further evidence of the likely reliability of the deflection sensors.

7. LORRY BRAKING WITHIN MIDSPAN ZONE

The final aspect of the above tests here drawn attention to concerns the provision of data to enable possible vibration identification of the bridge over time. To that end, for each of girders FG5 and FG6, Figures 12 compare the strain influence lines for the 30 mph runs without and with braking. On Fig. 12(a), focused on FG5, the start and end of the braking process are labelled BS and BE respectively. It is evident that the braking started at just about the location where the dip explained at length earlier on in this paper started on the plot with no braking. Now braking reduced the lorry's speed to an unknown extent while travelling along the bridge, so in this case the influence line is with respect to time only, not to axle location (which would require confidence in the lorry speed data).

Indeed, for the no-braking plots, the rooting of the strain dips in fundamental mechanics meant that the starts of those dips (with the front axle at midspan) could be used as starting points for the braking activity. This coincidence in Fig. 12 between the start of braking and the start of the no-braking dip means that the braking-induced strain rise is even more readily appreciated. This rise was due to transfer of load onto the front axle by the braking. Once the brakes were released (point BE in Fig. 12), thereby transferring load off the front axle, the strain influence lines then exhibited dips. It should also be stated that since the lorry would have slowed down once braking had occurred, the assumption of a continued lorry speed of 30 mph is, strictly speaking, not accurate. However, without the benefit of a tracker to indicate the modified lorry speeds in real time, it wasn't possible to assume alternative lorry speeds. Hence the abscissae on the latter parts of the strain plots, which are based on 30 mph, should be treated with caution.

Note also the significant oscillations on the braking plots of Fig. 12, both during and after braking. Indeed, it is seen that the oscillations continued to be reasonably pronounced even after the lorry exited the bridge. This suggests that the braking produced good data on the free vibration behaviour of the bridge. That is important, since the vibrations present in the data from the normal 30 mph runs along the bridge are subtle, almost to a vanishing point. Hence it is recommended that lorry braking around midspan could be used as a source of vibration data for FRP bridges with longitudinal pultruded units connected by transverse adhesive bonds.

8. SUMMARY AND CONCLUSIONS

The key conclusions to be drawn from the results of the study presented in this paper are :

- The normalised transverse profiles of the recorded longitudinal strains at the soffits of the T-beams and of the grillage predicted longitudinal moments in these T-beams correlate well at recorded strains exceeding 55$\mu\varepsilon$. This suggests that the reduced transverse load distribution capability of this bridge form is predictable using an analysis tool (the grillage) which is popular with designers. As evidence of this low transverse load distribution capability, the recorded strain profile suggests a significant 30% drop in flexure between adjacent T-beams in the transverse direction away from the axle loads.
- A deflected surface of the deck was obtained by surface-fitting through the 2D array of recorded deflections. This surface shows sharp drops in deflection over short transverse distances away from the axle loads, which reinforces the idea of the low transverse load distribution capability of this bridge form, as deduced from the recorded strain and predicted moment sharing data.
- The above-described grillage analysis suggests that the 100 mm asphalt layer on the deck might have an important stiffening effect on the structure. In one case inclusion of this asphalt layer in the model led to predicted deflections within 31% of the recorded value, down from within 97% of this recorded value without the asphalt included. This gravitation towards good

correlation with the test data along with the continuum nature of the grillage model together strongly suggest that the bridge deck behaved free of cracks propagating through the bonded deck-deck joints due to any transverse tension which might have developed across those joints.

- The recorded strain influence lines mimic the moment influence lines, including an idiosyncratic feature of the moment influence line. This is further evidence that the structure is behaving as a continuum, because any propagating cracks at the bonded joints would have induced strain redistributions causing the trends in strain behaviour to deviate from those in moment behaviour.

- The recorded strains satisfy the superposition principle when considering complementary lorry tracks along the bridge. This is yet further evidence of the continuum nature, free of the redistributing effects of cracks.

- The use of both electronic, non-contact displacement sensors and mechanical, contact displacement sensors is a useful strategy for checking the reliability of deflection measurements, because these lorry-induced deflections in the FRP bridge were small (within 4 mm). The ratio of deflection readings between the two sets of sensors exhibited an average of 1.07, with a CoV of 0.025.

- Sharp braking from 30 mph, with the front axle near midspan, provided a good source of vibration data for the bridge. The vibrations appeared to continue even after the lorry had rolled off the bridge, so that the data could potentially be used for free vibration analysis of the structure.

The work presented in this study may be extended in various ways. First, in addition to independent measurements of deflection as pursued in this study, future work may also consider independent measurements of strain, for example using optical fibres with Bragg gratings alongside the presently used electrical resistance strain gauges. Further, a more detailed 2D array in plan of deflection measurement points may be used. Surface fitting through the resulting data could be used to establish the minimum number and layout of measurement points needed to reliably define that surface.

Very importantly, the present tests have been concerned with global facets of the structure's response to lorry loads. Instrumentation geared towards measuring local response, for example local biaxial flexing of the FRP deck's thin-walled top flanges in response to concentrated tyre loads transmitted through the surfacing, would be useful.

In closing, it is recommended that the monitoring and data interpretation approach presented in this paper may form part of a wider strategy to assess the health and performance of road bridges with longitudinally oriented, transversely bonded GFRP deck pultrusions.

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