Proceedings of Bridge Engineering 2 Conference 2009
April 2009, University of Bath, Bath, UK

A CRITICAL ANALYSIS OF THE ABERFELDY FOOTBRIDGE, SCOTLAND

J. M. Skinner[1]

[1] *University of Bath*

Abstract: This paper is an analysis of the world's first major advanced composite footbridge. Following an assessment of the aesthetics of the bridge, structural design, loading, strength and serviceability are considered. Particular focus is placed upon the construction, durability and dynamic behaviour of the footbridge. Possible future changes to this unique bridge are presented.

Keywords: *Aberfeldy, Advanced composite, ACCS*

1 Introduction

Fibre-reinforced polymer composites were first used in World War 2 in the construction of British Spitfires [1]. In the last few decades they have been used in the construction industry, mainly as non-structural components. The Aberfeldy Footbridge was the world's first major advanced composite footbridge and remains the longest span advanced composite bridge in the world.

Figure 1: The Aberfeldy footbridge [2]

The request for the design and construction of a footbridge over the River Tay came from the golf club in the town of Aberfeldy in Scotland. The 9 hole golf course lay on one side of the River Tay but the club wanted to extend the course to the other side of the river to create a further 9 holes. An existing hump-backed bridge was no longer a suitable crossing and desiring a cheap replacement the club approached Prof. Harvey at Dundee University.

The design of the footbridge became the final year project for the bridge engineering students of Dundee University. However it soon became apparent that the length of span required ruled out traditional materials which would require using equipment heavier than was appropriate for the site [3]. This led to Maunsell Structural Plastics being appointed the lead engineers. They provided the Advanced Composites Component System (ACCS), a system which uses prefabricated panels joined by mechanical toggles and bonded connections.

2 Aesthetics

When discussing the aesthetic qualities of a bridge it can be difficult to avoid subjectivity. One perspective is that of Fritz Leonhardt, who set out his ten areas of aesthetics of bridge design in his book 'Bridges'. The following analysis will use these principles to assess the aesthetic qualities of the Aberfeldy Bridge.

The form of the bridge may be considered one of the most important criteria in bridge design. A bridge's form must impart, to those commencing a journey across it, the feeling of stability and safety. The Aberfeldy Bridge does this, the form of the structure being clear and simple and the transfer of the loads from one element to another

[1] Jonathan Skinner – jms22@bath.ac.uk

obvious, Fig. 1. Observing from beneath the bridge the structure supporting the deck is unambiguous. The panels forming the deck span between the main cross beams, which at their ends, meet the stays. Walking across the bridge the user passes through pairs of cables which narrow to meet at the top of the two A-frames, Fig. 2.

The main elements of the bridge appear well proportioned. The deck passes gracefully over the river, being shallow enough in depth to suggest it is reliant on the support of the cables. The frames might seem awkward in shape due to the modular components they are constructed from but when viewed together with the deck they seem balanced in size. Unfortunately there is an illusion created by the parapet and handrails of the bridge. The spindles create an opaque barrier and viewing the bridge from a distance deceives the eye, making the deck seem deeper than it truly is. A more aesthetically pleasing solution would have been to have toughened glass infill panels spanning between uprights so as not to interrupt the line of sight.

Figure 2: Walking over the bridge [4]

One of the key shortcomings of cable-stayed bridges is that they are often designed in elevation and when observed from oblique angles the cables criss-cross disturbing the order of the design. Even though the Aberfeldy Bridge has two planes of cables exacerbating this problem the issue is negated in two ways. The first is to limit the number of cables to five in each plane and the second is to arrange the cables in a fan system with the cables meeting near the top of the frame, Fig. 3. Other visual disturbances within the structure include the cable to edge beam connection. Designed from standard ACCS toggle components and fixed to the outside edge of the bridge, the smooth line of the deck is broken up destroying the simplicity of the aesthetics.

The inability to be able to refine the design due to the use of standard components is one of the major flaws of the bridge. Elliptical or curved profiles for the columns of the A-frames are not possible for this very reason. Yet when non-standard pultruded sections have been used for the parapet and handrails the results have been have disappointing, the parapet pierces through the handrail at every point where they connect together.

Figure 3: Cables designed in a fan arrangement [5]

As pedestrians are able to linger and enjoy a footbridge from all aspects as they cross it, the surface texture and the refinements of the bridge are crucial to its aesthetic appeal. The Aberfeldy Bridge is frustrating in this respect. As previously mentioned the parapet and handrails are not refined in design and owing to the erosion of the resin from these elements the glass fibres have become exposed in places, causing the handrail to be rough to the touch. The wearing surface of the deck is manufactured from black rubber conveyer belting, chosen because it adds significant dead weight to the deck [3]. Nevertheless the texture it provides although appropriate for the bridge is uninspiring.

An excellent aspect of composites is that they can be made in any form, texture or colour. The colour of the frame and deck components was chosen by the planning authority to blend with the trees in the autumn. Considering the possibilities available this seems to be an opportunity missed. The bridge could have been beautifully crisp, although this is difficult to envisage considering the north facing elements are currently marred by mould. A crisply finished bridge, more accustomed to being found in a cityscape than the Scottish Highlands, is not ill fitting for a golf course; a manicured landscape in which a sport is played with highly engineered equipment.

Character as a quantity is difficult to define. Although the Aberfeldy Bridge lacks the boldness of the Humber Bridge or the beauty of Calatrava's Alamillo Bridge its simple form is memorable and distinct. The A-frames display a striking form against the landscape as the deck gracefully traverses the River Tay. A final testament to the character of the bridge is the affection of the locals who much admire it [6].

2.1 Summary

A bridge's compliance with all ten of Fritz Leonhardt's rules of bridge aesthetics does not necessarily grace it with the title of beautiful. Nonetheless it would be surprising for a bridge which disobeys these principles to be stunning. The Aberfeldy Bridge is for the most part successful aesthetically; simple and clean in its form it avoids structural confusion. Refinements in texture and colour would assist in improving its appearance but the bridge is ultimately limited by the components that it is formed from.

3 Structural design

A cable-stayed bridge supports the deck by a set of cables travelling from the deck directly to the frames or pylon, Fig. 6. This arrangement has the advantage over suspension bridges, of containing the horizontal forces within the structure, negating the requirement for horizontal anchorage. However this demands the deck to be capable of accommodating high compression forces which is difficult when using composites because the compressive strength of the material is dominated by the tensile strength of the resin restraining the fibres.

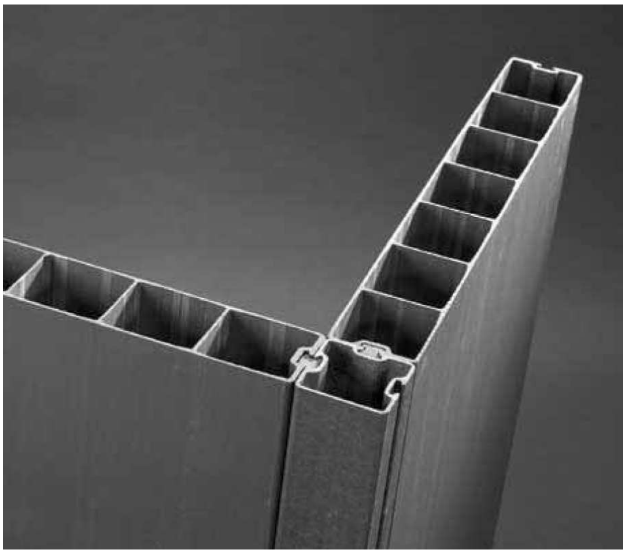

Figure 4: Advanced Composites Component System [7]

Aberfeldy is constructed from ACCS components, glass fibre reinforced polymer (GFRP) pultruded elements and Parafil, aramid fibre reinforced polymer (AFRP), cables.

The ACCS, Fig. 4, consists of prefabricated composite building panels which are connected together with connectors and toggles before being permanently bonded together. On the Aberfeldy footbridge the A-frames, deck and beams are all formed from this system. Each section of deck is fashioned from building panels spanning a metre between primary and secondary beams. The primary beams are formed of 4 connector sections bonded together and located every six metres where the deck meets the supporting cables. Edge beams, produced from five connector pieces bonded together, span between the primary beams and bear the secondary beams supporting the deck, Fig 5.

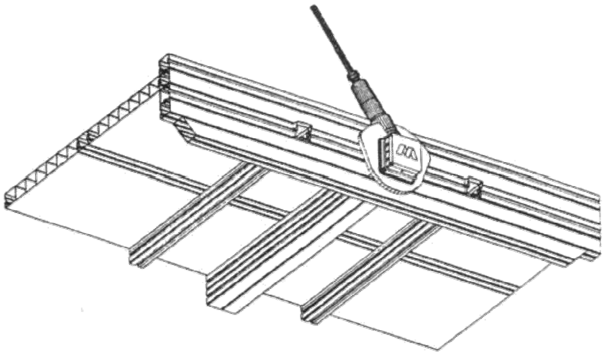

Figure 5: Aberfeldy bridge deck arrangement [8]

The desire to maximize the use of advanced composites dictated the use of Parafil Kevlar cables over a high tensile steel alternative. Kevlar has a good strength to weight ratio but has a lower stiffness than steel which means serviceability is more likely to dominate the design.

4 Loading

To enable a structural analysis of the footbridge to be carried out the different loading conditions experienced by the bridge must be considered. BS5400-2:2006 [9] is used to assess the different loads.

4.1 Dead and superimposed dead loads

Reference [5] states that the combined dead and superimposed dead loads that act upon the bridge are 2kN/m, of which 1kN/m is concrete ballast. This is particularly low for a footbridge and as is explained later, has a huge effect on the bridge's dynamic behavior.

4.2 Live load

4.2.1 Pedestrian loading

As this is a footbridge pedestrian loading has to be

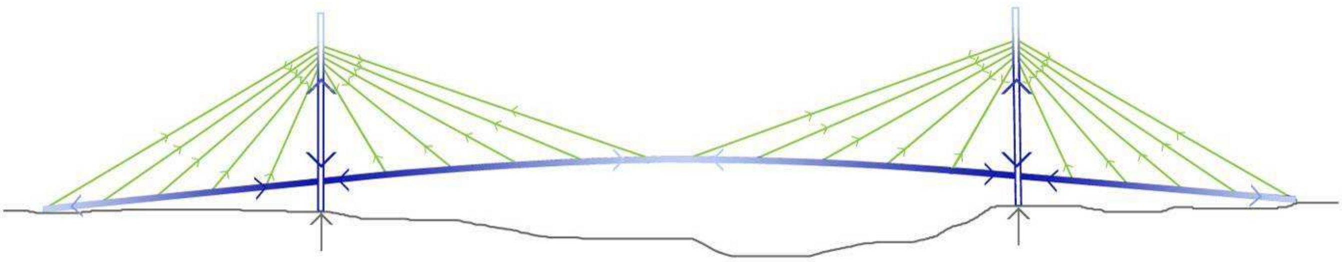

Figure 6: Distribution of forces throughout the footbridge

considered. For a footbridge less than or equal to 36m a uniformly distributed live load of 5.0kN/m² is applied. However for bridges which exceed 36m in length it is assumed that the probability of a load this high being applied across the whole bridge decreases as the length of the bridge increases. This is taken into account by a correction factor, *k*.

$$k = \frac{\text{nominal HA udl for loaded length} \times 10}{L + 270} \qquad (1)$$

As the largest span of the bridge deck is 63m the nominal HA udl, *W*, is calculated using Eq. (2) to be 23.8kN/m.

$$W = 36\left(\frac{1}{L}\right)^{0.1} \qquad (2)$$

Substituting *W* into Eq. (1) and *k* is found to be 0.74. Therefore the pedestrian live load acting upon the bridge is:

$$w_{live} = 0.74 \times 5.0 = 3.57 kN/m^2$$

As the width of the deck accessible to those crossing the bridge deck is only 1.6m, the load per metre length is 5.7kN/m.

4.2.2 Parapet loading

As the bridge is only intended to be used by pedestrians the parapets only have to withstand a nominal load horizontally of 1.4kN per metre run.

4.2.3 Golf cart loading

Originally the designers did not to allow for the load of a golf cart in their calculations. Since the bridge links two halves of a golf course this was a major oversight. Assuming that only a single golf cart is crossing the bridge at any one time and that its weight is 500kg, this produces a point load for each wheel of 1.25kN.

There is also a chance that when crossing the bridge the golf cart would collide with the parapet. It would not be appropriate to allow for the full vehicle collision load as stipulated in BS5400-2:2006 [9] because both the speed and weight of the vehicle being considered are far smaller than for a highway bridge. The quasi-static fore that would be applied to the parapet during a collision is given by Eq. (3).

$$F = m\left(\frac{u - v}{t}\right) \qquad (3)$$

Assuming that the initial velocity of the golf cart, *u*, is limited to 15mph or 6.5m/s, the time taken to dissipate the energy during the collision, *t*, is 0.2s and the cart does not crumple during the collision, the quasi-static force is:

$$F = 500\left(\frac{6.5 - 0}{0.2}\right) = 16.25\text{kN}$$

4.3 Wind load

Wind loading upon the bridge may cause problems because the footbridge has very little dead load to dampen the motion generated by gusts of wind. However in its favour the footbridge has only a very small projected area due to its slender deck. Wind loads applied to footbridges less than 30m in span are calculated according to clause 5.3 of BS5400-2:2006 [9], using static analytical methods. As the Aberfeldy footbridge spans 63m the effects of dynamic response due to turbulence should be considered but for simplicity the methods covered by clause 5.3 will be used.

4.31 Dynamic pressure head

The dynamic pressure head, *q*, is calculated from Eqs. (4,5). From the basic wind speed chart [9] the basic hourly mean wind speed, V_b, is 23m/s. Other information required to find the dynamic pressure head includes; the height of the site, 86m above sea level, the distance of the site from the sea, 50km and a 120 year return period.

$$q = 0.613 V_d^{\,2} \qquad (4)$$

$$V_d = (S_b T_g S_h')(V_b S_p S_a S_d) \qquad (5)$$

Table 1: Values for Eq. (5)

$S_b = 1.27$	$S_p = 1.05$
$T_g = 1.00$	$S_a = 1.09$
$S_h' = 1.00$	$S_d = 1.00$

The maximum wind gust speed, V_d, found by multiplying the data in Table 1 by the basic hourly mean wind speed is 33.4*m/s*. This gives a dynamic pressure head of 0.68*kN/m²*.

4.32 Transverse wind load

Equation (6) is used to calculate the nominal transverse wind load, P_t, which acts horizontally at the centroid of the main span. A_1, the solid area is the sum of the solid area in projected elevation and the live load depth (1.25m for footbridges). If the depth of the deck is 0.4m A_1= (0.4 + 1.25) × 113 = 186.5m². The drag coefficient, C_D, is 2.0.

$$P_t = q A_1 C_D \qquad (6)$$

$$P_t = 0.68 \times 186.5 \times 2.0 = 253.6 kN$$

4.33Longitudinal wind load

It's important to calculate the forces generated by longitudinal wind loads as they contribute towards the size and type of bearings specified. The longitudinal wind load is derived by Eq. (7) below. P_{LL}, the longitudinal wind load acting on the live load area, and P_{LS}, the longitudinal wind load acting on the superstructure, are calculated using Eqs. (8,9) respectively.

$$P_L = P_{LL} + P_{LS} \qquad (7)$$

$$P_{LL} = 0.25 q A_{11} C_{D1} \qquad (8)$$

$$P_{LS} = 0.5 q A_{12} C_{D2} \qquad (9)$$

Table 2: Values for Eqs. (8,9)

$A_{11} = 2.75m^2$	$A_{12} = 50.0m^2$
$C_{D1} = 1.45$	$C_{D2} = 2.00$

Using the values in Table 2, P_{LL}=0.68kN, P_{LS}=34.0kN and P_L=34.7kN. These low values are not surprising as the bridge is narrow, providing only a very small projected area in the longitudinal direction.

4.34 Vertical wind load

BS5400-2:2006 [9] provides Eq. (10) for calculating the nominal vertical wind load acting either up or down at the centroid of the main span.

$$P_V = qA_3C_L \tag{10}$$

The plan area of the bridge, A_3, is 271m². The lift coefficient, C_L, is 0.4 (found using Eq. (11) and the assumption that the super elevation, α, is 0°). Hence the nominal vertical wind load is 96.7kN.

$$C_L = 0.75\left[1 - \frac{b}{20d}(1 - 0.2\alpha)\right] \tag{11}$$

The total dead load of the bridge deck is 2kN/m, 1kN/m of which is concrete ballast. As the bridge spans 113m, without this ballast it is clearly in danger of uplift.

4.32 Wind loading combinations

There are four combinations to consider when considering wind loading they are; P_t alone, P_t with ± P_V, P_L alone and 0.5P_t with P_L ± 0.5P_V.

4.4 Temperature effects

Changes in the overall temperature of a bridge and difference in temperature between top and bottom surfaces are important in bridge design. In the design of composite structures it is essential to remember that the material has anisotropic properties. This extends to the thermal expansion of composites, the longitudinal coefficient of thermal expansion dominated by the fibres and the transverse coefficient dominated by the resin. Of equal interest is the negative coefficient of thermal expansion for the AFRP cables used on the bridge.

For a 1 in 50 year return period the maximum and minimum effective bridge temperature can be approximated from Ref. [9] as 36°*C* and -18°*C* respectively. If a datum temperature at the time of construction is assumed to lie at the midpoint of these two values then the change in temperature is ± 27°*C*. The strain in the deck in the longitudinal direction will be calculated from Eq. (12) and although the coefficient of thermal expansion in the transverse direction is very high, the width of the deck is small translating to the expansion being small enough not to be of concern. In the longitudinal direction the coefficient of thermal expansion, α, is 7.9 x $10^{-6}K^{-1}$.

$$\varepsilon = \Delta T\alpha \tag{12}$$

$$\varepsilon = 213\mu\varepsilon$$

$$\delta = L\varepsilon \tag{13}$$

$$\delta = 113\times10^3\times213\times10^{-6} = 24mm$$

The extension of the deck should be accommodated with an expansion joint. If the joint was to become jammed and the bridge deck to then expand or contract then a stress would be induced within the deck. Using Eq. (14) and assuming E=17140N/mm² [7] the stress would be 3.65N/mm².

$$\sigma = \varepsilon E \tag{14}$$

As mentioned previously, Kevlar cables have a negative coefficient of thermal expansion. Therefore a temperature increase causes the cables to contract and to stiffen the structure. Considering the tension cables have relaxed since their installation, this would improve the stiffness of the bridge. However if the temperature falls then the bridge deck contracts and the cables extend causing the bridge to deflect further at the mid-span than if it had been constructed from steel which has a positive coefficient of thermal expansion. For the cables, the extension is significant but the force induced in the cables is small.

$$\varepsilon = -2.1\times10^{-6}\times-27 = 57\mu\varepsilon$$

$$\delta = 113\times10^3\times57\times10^{-6} = 6.4mm$$

$$\sigma = \varepsilon E = 57\times10^{-6}\times86000 = 4.9N/mm^2$$

$$P = \sigma A = 4.9\times114.6 = 0.56kN$$

Up to this point only changes in overall temperature have been considered. A second scenario covers the effects of a temperature difference between the top and bottom surfaces of the deck. In this case the difference in the induced stress above and below the neutral axis generates a bending moment in the deck.

4.5 Partial safety factors

As fibre reinforced polymer (FRP) structures are designed using a permissible stress approach, all partial load factors are set to unity. However to account for the variability in material strength there are a number of partial material safety factors that will now be discussed.

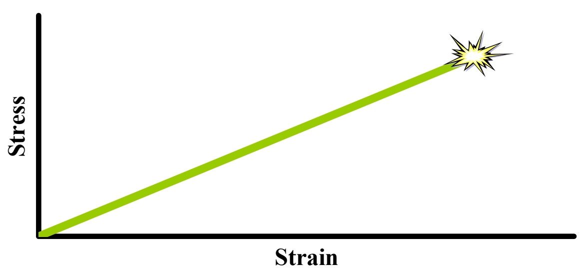

Figure 7: Typical stress-strain trend

4.51 Manufacturing partial safety factor

The first partial safety factor is used to take account of the manufacturing process. All the components were pultruded in a factory, a well controlled process and environment. This means the uncertainty surrounding the quality of the process is small and therefore γ_{mm}=1.1.

4.52 Reliability of the fibre mode of failure

FRPs fail in a brittle manner as shown in Fig.7. Because the fibres exhibit no plastic deformation the reliability of the failure strength is critical. This leads to high material safety factors, for GFRP γ_{mf}=2.0 and for AFRP γ_{mf}=2.0.

4.53 Partial safety factor for durability

A number of problems associated with durability affect the long-term strength of a composite. Problems include the capability of GFRP to absorb water [5] and for local stress concentrations to form at joints or connections ,all these will affect the long-term strength of the structure. Damage may also be sustained by the structure causing debonding and delamination. To allow for all these effects partial safety factors of γ_{md}=1.4 for GFRP and γ_{md}=1.2 for AFRP are introduced.

4.54 Overall partial safety factor for material strength

The overall partial material safety factor for the composite is given by Eq. (15). For GFRP γ_m=3.1 and for AFRP γ_m=2.6.

$$\gamma_m = \gamma_{mm}\gamma_{mf}\gamma_{md} \tag{15}$$

4.6Creep and creep rupture

Although all materials creep, materials with a higher stiffness display less of an effect. Additionally FRPs relative sensitivity to load exposure over an extended period of time cause them to exhibit creep rupture.

Creep rupture is prevented by limiting creep. This is achieved by reducing the working strains under long-term loads to a level where creep is no longer a problem. The following working-stress limits have been suggested as being suitable to be imposed upon a wholly polymeric structure where f_{fu} is the strength parameter under consideration for a 50 year design life.

GFRP – max. allowable stress = 0.20f_{fu}
AFRP – max. allowable stress = 0.30f_{fu}

In addition to these limits under long-term loading the short-term Young's Modulus should be reduced to allow for long-term creep. The following factors have been suggested as appropriate.

GFRP – long-term E = short-term E/1.8
AFRP – long-term E = short-term E/1.5

4.7 Load combinations

Although the limiting factors upon stresses during long term loading are severe, the ratio of dead load to live load is so high that the critical combination is found when dead and live loads are applied simultaneously.

It is also clear by noting the magnitudes of loads found in sections 4.3 and 4.4, that while significant stresses are induced into the structure by a large change in temperature, combination 2 will be the worst case.

Table 3: Load combinations

Combination 1	Permanent and live loads
Combination 2	Combination 1 and wind loads
Combination 3	Combination 1 and temperature loads

5 Strength

In the original design the load of a golf cart was not included. This section will compare the effect of this additional load with the pedestrian loading taken in the design stage.

5.1 Bending of the edge beam

The loads acting upon the beam are half of the dead and live loads and the vertical wind load, which is assumed to act uniformly across the main span. To find the bending stress in the beam it is assumed that bridge is a continuous beam with supports at each cable position. Figure 8 illustrates the bending moment diagram for this arrangement. Reference [7] indicates the elastic modulus for each edge beam is 292.8 × $10^3 mm^3$. Eqs. (16,17) are used to find the maximum hogging moment and bending stress.

$$M = \frac{wl^2}{12} = \frac{(3.85+0.77)\times 6^2}{12} \tag{16}$$

$$M = 13.9kNm$$

$$\sigma = \frac{M}{Z} = \frac{13.9\times 10^6}{292.8\times 10^3} \tag{17}$$

$$\sigma = 47.5N/mm^2$$

$$\sigma_a = \frac{\sigma}{\gamma_m} = \frac{169}{3.1} = 54.5N/mm^2$$

If the golf cart is assumed to act at the centre of the span then the additional moment in the edge beam is found by Eq. (18). This is an additional stress of 6.5N/mm^2 which summed with the current stress does not exceed the beam's capacity. However it is advisable that only one golf cart be allowed on the bridge at any one time.

$$M = \frac{Pl}{8} = \frac{2.5\times 6}{8} = 1.9kNm \tag{18}$$

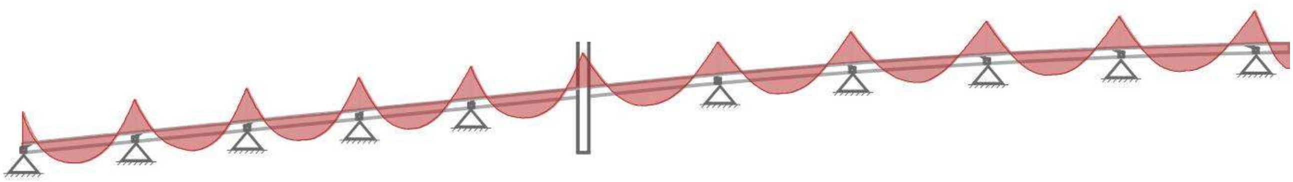

Figure 8: Bending moment diagram for half of the bridge

5.2 Cable Tension

The tension force and stress in a Parafil cable is found by Eqs. (19,20). It is assumed that each cable supports the load acting over half the width of the deck and the cables are distributed at 6m centres. Taking the cross-sectional area of a cable as 114.6mm²[10] and the tensile capacity of the cable as 1.9kN/mm² [10], it is found that the stress in the cable is within the allowable limits.

If the load of a single golf cart is also assumed to be supported by a pair of cables then the additional stress within each cable is 47.5N/mm². Hence the total stress for each cable is 575.5N/mm². This is not higher than the allowable limits and indicates that the cables are adequate for the additional load. However the connection between cable and deck should be checked to ensure that it is strong enough to transfer the shear forces.

$$P = \frac{wl}{\sin\alpha} = \frac{(3.85 + 0.77)\times 6}{\sin 27.3°} \tag{19}$$

$$P = 60.4kN$$

$$\sigma = \frac{P}{A} = \frac{60.4\times 10^3}{114.6} \tag{20}$$

$$\sigma = 527N/mm^2$$

$$\sigma_a = \frac{1900}{2.6} = 731N/mm^2$$

5.3 Compression in the deck and buckling

Composites are anisotropic, the direction of the fibres dictating the properties of the material. In the design of the bridge deck the majority of the fibres will be aligned along the length of the deck with far fewer fibres in the transverse direction. This dictates that the bridge will be strong in tension but weak in compression because the strength of the composite in compression is defined by the ability of the resin to prevent the longitudinal fibres from buckling.

If the deck is assumed to buckle between each set of cables, then the maximum compressive force due to the horizontal reaction at the cable supports occurs in the section of deck which spans between the A-frame and the adjacent cable. This is illustrated in Fig. (6), the intensity of colour of the deck representing the magnitude of the compressive force. The maximum stress from this effect, σ_c, can be found by Eq. (21), the cross-sectional area of each edge beam and the deck are 4096mm² and 22940mm² respectively [7].

$$\sigma_c = \frac{P\cos\alpha}{A} \tag{21}$$

$$\sigma_c = 10\left(\frac{60.4\times 10^3 \times \cos 27.3^0}{31132}\right)$$

$$\sigma_c = 17.2N/mm^2$$

Similarly the compressive stress due to the hogging moment in the deck, σ_b, is calculated from Eq. (23). Using information from Ref. [7] and Eq. (22) the second moment of area for the deck and edge beams is found to be 276.3 × 10⁶*mm⁴*. The neutral axis for the section is 188mm from the top side of the edge beam and the edge beam is 400mm deep.

$$I_{total} = \sum\left(A\bar{y}^2 + I_{local}\right) \tag{22}$$

$$\sigma_b = \frac{M\bar{y}}{I_{xx}} = \frac{(2\times 13.9)\times 10^6 \times 212}{276.3\times 10^6} \tag{23}$$

$$\sigma_b = 21.3N/mm^2$$

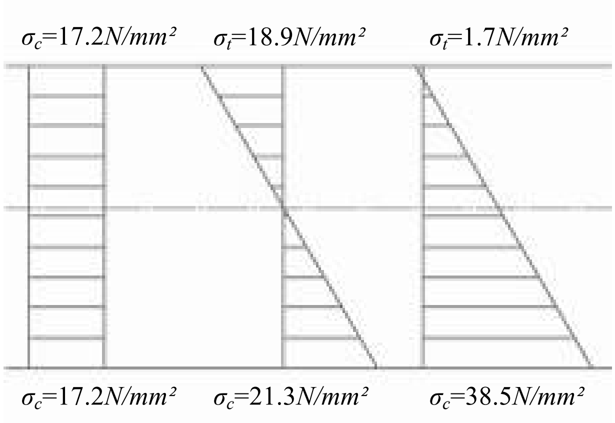

Figure 9: Stress profile for the deck in hogging

Combining the compressive stresses produces a stress profile for the deck in hogging, Fig. 9. The critical buckling load is calculated by Euler's formula, Eq. (24). Since the allowable stress is far smaller than the applied stress it could be considered that the bridge might buckle under a high loading. However for buckling to occur the deck must contract in length. As the deck is continuous at the centre of its span this will not take place and so the deck cannot buckle.

$$P_E = \frac{\pi^2 EI}{l'^2\gamma_m} = \frac{\pi^2 \times 17140\times 276.3\times 10^6}{6000^2 \times 3.1} \tag{24}$$

$$P_E = 419kN$$

$$\sigma_a = \frac{419\times 10^3}{31132} = 13.5N/mm^2$$

5.4 Asymmetrical loading

Under asymmetrical loading a bending moment is induced in the A-frame. This calculation is beyond the scope of this paper but would be conducted by finding the horizontal forces for each cable reaction at the A-frame and then applying a virtual work method to find the moment at the base.

6 Serviceability

Compared to other materials, the strength to weight ratio of advanced composites is very good. In contrast their stiffness is low and this should dominate the design of the footbridge. Deflection of the edge beam between the cable supports under dead, pedestrian and wind loading is given by Eq. (25), the second moment of area of the edge beam is 75.0 × 10⁶*mm⁴* [7].

$$\delta_{max} = \frac{wl^4}{384EI} \tag{25}$$

$$\delta_{max} = \frac{5.35\times 6000^4}{384\times 17.14\times 10^3 \times 75.0\times 10^6}$$

$$\delta_{\max} = 14.0mm$$

Each cable will also extend under loading. By using Eq. (26) and assuming the length of the longest cable is 30m and the Young's modulus for the cable is 125.6 x $10^3 N/mm^2$ [10] the extension of the cable can be found. The extension is then resolved to find the vertical component and hence the vertical deflection of each cable is 57.8*mm*. Clearly this deflection is of some concern. Although strong, the cables lack stiffness and a cable with a larger cross-sectional area or one formed from steel instead of Kevlar might be considered more appropriate.

$$e = \frac{Pl}{AE} \qquad (26)$$

7 Construction

One of the aims of the project was to enable the students from the department to assemble the bridge themselves. This was realised by the advanced composite components being light, negating the need for heavy lifting equipment. The small tolerances required in the construction of the A-frames led to the legs and cross beam being delivered to site preassembled. Using only a winch and telescopic forklift truck the frames were lifted to their final vertical position.

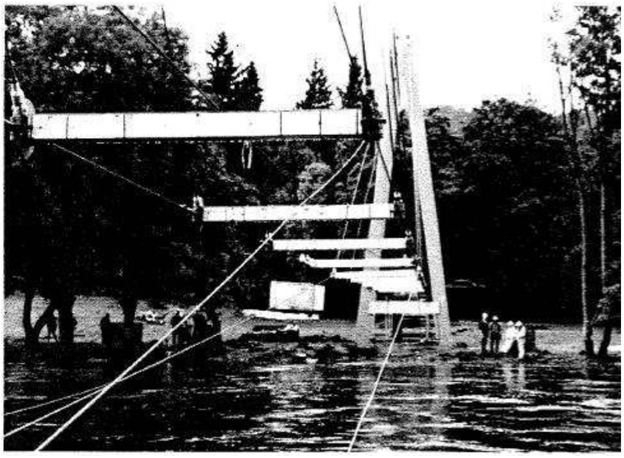

Figure 10: Construction of the footbridge [11]

A combination of suspended construction and incremental launching was used. Cross beams were bonded together and cables and guys attached to their ends to position them every 6 metres across the river. The deck was bonded together beneath a 65 metre long temporary ramp, constructed from standard scaffolding components. It was found that keeping the components dry during the curing process was critical, so a cover was placed over the ramp for protection [3].

As the bridge was incrementally launched the alignment of the sections during the bonding of the bridge is crucial. As the main span of the bridge is 63 metres in length, if the tolerances were 1mm per metre length of the bridge, the horizontal misalignment between the frames could be 63 millimetres. Failure to meet these tolerances would lead to the deck missing its bearings, a considerable problem.

A winch was used to haul the bridge deck over the river from the opposite bank. To lower and align the deck a unique system was used to raise and lower the deck into place over the cross beams [3]. This arrangement is illustrated below in Fig. 11. Once in position the deck was bonded to the cross beams and the weathering surface placed on top.

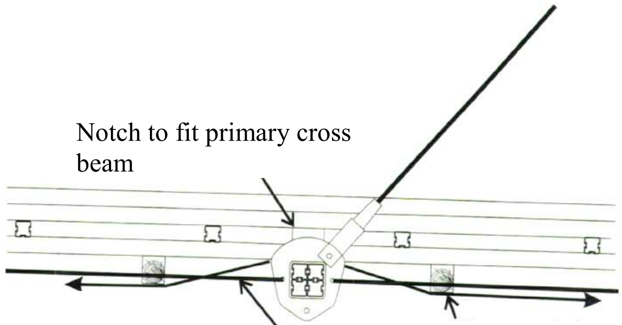

Figure 11: Deck positioning method [3]

8 Foundations

A site investigation was undertaken and the series of trial pits excavated indicated that a layer of sand approximately 1m deep, concealing a thick layer of boulder clay covered the site [3]. Since the bridge is constructed from relatively lightweight materials pad footings are adequate, the size of which are dictated by the weight required to resist the uplift forces under asymmetric loading rather than contact area required to minimise settlement.

At each cable connection location along the outside spans of the bridge a pad footing is sited. If the central span is fully loaded with pedestrians and the outside spans are not the uplift force for each pair of cables is 6*m* x 5.7*kN/m* = 33.6*kN*. According to Ref. [3] the size of each pad footing is 1.2x0.8x4.5=4.3*m*. Assuming the weight of concrete is $24kN/m^3$ the load resisting the uplift is 103.7*kN*. This provides an acceptable factor of safety of:

$$F.O.S. = \frac{103.7}{33.6} = 3.1$$

9 Natural frequency

Constructed from advanced composite materials the bridge has a high live to dead load ratio. To help dampen the structure, concrete ballast was added doubling the dead weight of the bridge but the ratio remains higher than for a similar bridge constructed from conventional materials. This coupled with the bridge's slender proportions means that dynamic effects are important.

For all footbridges in the UK Annex B of BS5400-2:2006 [9] stipulates that the dynamic behaviour must be controlled by ensuring that the natural frequency of the bridge is above 5Hz. Below 5Hz the natural frequency is nearing that of vibrations caused by footfall and resonance is likely to occur, making it difficult to walk across the bridge. This problem can be exacerbated by lock-in. In this scenario those crossing the bridge are forced to walk in time with its movement, further increasing the amplitude of the oscillations. Ideally the natural frequency is also limited to below 75Hz. Above

75Hz the bridge is unlikely to collapse but it might cause uncomfortable psychological effects.

Apart from adding additional dead weight to the structure to improve its dynamic response, a frictional joint between the handrail and upright was designed, Fig 12. The intention of the connection was to help stiffen the parapet and hence prevent flexing of the deck. However the connection between parapet and deck lacks rigidity, this leads to the sleeved handrail frictional joint acting ineffectively.

Figure 12: Frictional joint [5]

As the bridge is a demonstration project, two studies have been undertaken focusing upon the dynamic performance of the bridge, including the likelihood of lock-in occurring [12]. Both demonstrated that the bridge is lively but acceptable for its intended use.

Reference [5] states the fundamental natural frequency of the bridge is approximately 1.5Hz. A simplified approach for finding the fundamental natural frequency of the structure is to use Eq. (27). The bridge is evaluated for the clamped-clamped mode and the resulting fundamental natural frequency is 95.6Hz. Clearly this method is inappropriate for evaluating the fundamental natural frequency of lightweight footbridges; a different approach must be considered.

$$\omega_n = (\beta_n l)^2 \sqrt{\frac{EI}{ml^4}} \tag{27}$$

Table 4: Values for Eq. (27)

$(\beta_n l)^2 = 22.37$	$E = 17.14 \times 10^3\, N/mm^2$
$I = 276.3 \times 10^{-4}\, m^4$	
$m = 200 kg/m$	$l = 6m$

A final consideration is the damage caused by forced vibrations, this type of vandalism although less likely to take place on a private golf course is not unforeseeable. Therefore the bridge bearings must be tough and capable of withstanding the vertical and horizontal loads associated with these forced vibrations.

10 Durability

As the footbridge is an innovative application of composites in construction, it will be regarded as a demonstration for the feasibility of future bridges to be constructed from these materials. Durability is a significant factor when discussing the appropriate use of advanced composites and the Aberfeldy footbridge demonstrates some important pitfalls to avoid in future designs.

An oversight in the original design was not to allow for the load of a golf cart or small tractor transporting sand. Subsequently the bridge was overloaded on several occasions and cracks formed in the top surface of the GFRP deck parallel to the webs of the cellular sections. The bridge was strengthened in1997. GFRP pultruded plates were bonded to the topside of the deck as well as CFRP sheets applied to the deck edge beams either side of the stay connections to allow for the increased cable reactions [12].

In addition to the cracks forming in the deck, damage was caused to the uprights of the hand railing after a golf cart collided with it. Having not been designed for impact loading this has caused delimitation of the posts. A protective kickboard would have helped to alleviate the chance of such damage [12].

Mould and moss growth is a considerable problem for the footbridge. The composite structure is capable of absorbing 1.5% of its own weight in moisture [5]. Combined with poor detailing this has led to the growth of mould, lichen moss and algae on both the primary structure and parapets. The north facing structure and grooves of the modular system are particularly badly affected, Fig. 13. It has been suggested [12] that the addition of mould inhibiting additives in the resin would have combated the problem and it is almost certain that this will be specified for future composite bridges.

Figure 13: Mould and lichen growth [5]

Weathering of the bridge is one of the major components of a bridge's durability. The erosion of the resin has exposed the glass fibres of the parapet and handrails. In comparison the standard ACCS components have performed well and show little wear.

Figure 14: Parapet-deck connection [5]

The parapet connection to the deck was constructed by passing the section through a sleeve fitted into the deck and fixing with a dowel. Many of these connections have worked loose as a result of repeated deck displacements. In some cases the rail displaces as far as 20mm [6]. Greater moment capacity through the use of 2 dowels at each connection would improve the stiffness and reduce the deflection of the hand rail.

11 Future Changes

Aberfeldy is a demonstration bridge. To this end it is likely to be continually monitored and improved.

Already the deck has been strengthened to allow for the unforeseen load of a golf cart. Other obvious changes are the connections of the parapet to the deck, as they would benefit from a stiffer connection. Eventually the complete replacement of the handrails and parapet may be required as the glass fibres become more exposed. As mould growth worsens a cleaning program may be required for the bridge to maintain an acceptable appearance for those using it, although this will be at the discretion of the golf course owners.

12 Conclusion

After 17 years, the primary structure of the Aberfeldy footbridge is still in good condition. It has clearly demonstrated the capabilities of advanced composites in long span bridges.

However the GFRP parapet has proved less durable than the primary structure. Weathering of the resin exposing the fibres has highlighted the importance of good specification. Secondly the connection between the parapet and base has proved inadequate. This has rendered the frictional joint between upright and handrail, intended to improve the damping of the structure, redundant.

Other problems include the mould and lichen growth visible on the bridge. A combination of poor design and lack of maintenance have exacerbated the problem. Future designers might consider specifying a mould inhibiting resin for similar climatic conditions.

It was always understood the bridge was likely to behave in a lively manner due to the lightweight components used in its construction. Although there is a clear argument for improving the dynamic response of the bridge, as it is only intended to be used on a private golf it can be considered acceptable.

As a demonstration project the Aberfeldy footbridge has highlighted both the feasibility and pitfalls of designing long span advanced composite footbridges. It remains to date the longest spanning all composite bridge in the world.

Acknowledgements

I would like to thank Mark Evernden, Tim Ibell and Antony Darby for their assistance provided through the excellent Bridge Engineering and Advanced Composites in Construction lectures and accompanying notes.

References

[1] Mirmiran, A., *et al*, 2003. *World Survey of Civil Engineering Programs on Fiber Reinforced Polymer Composites for Construction,* Journal of Professional Issues in Engineering Education and Practice, Vol. 129, pp. 155-162

[2] Cambuslang, J. G., 2007. *Bridge over the Tay at Aberfeldy*. Available from: flickr.com/photos/westiesjohn/1571206419/ [Accessed Feb 2009]

[3] Harvey, W. J., 1993. *A Reinforced Plastic Footbridge, Aberfeldy, UK*. Structural Engineering International, 4/93, pp. 229-232

[4] Law, J., 2008. *Footbridge over the Tay, Aberfeldy Golf Course.* Available from: flickr.com/photos/12113806@N04/2355227862/ [Acessed Feb 2009]

[5] Stratford, T., 2008. *Aberfeldy footbridge: 16 years in service, Long Term in Service Performance of FRPs in Construction*. Presentation

[6] National Group for Composites in Construction, 2008. *Aberfeldy footbridge,* Available from: www.ngcc.org.uk/DesktopDefault.aspx?tabindex=84&tabid=558 [Accessed Feb 2008]

[7] Strongwell, 2008. *Composolite: Fibreglass building panel system*, Available form: www.strongwell.com [Accessed Apr 2009]

[8] Head, P. R., 1994. *Cost effective advanced composite structures designed for life in the infrastructure*, BPF congress

[9] BS5400-2:2006 *Steel, concrete and composite bridge - Part 2: Specification for Loads*. BSI.

[10] Balderson, T., TBalderson@linearcomposites.com, 6th Apr 2009. *Parafil Physical Properties*. Email to J.M. Skinner (jms22@bath.ac.uk)

[11] Burgoyne, C. J.; Head, P.R., 1993. *Aberfeldy Bridge – an advanced textile reinforced footbridge.* TechTextil Symposium, Frankfurt, Paper No. 418.

[12] Cadei, J., Stratford, T., 2000. *The Design, Construction and In-service Performance of the all-composite Aberfeldy Footbridge*. Advanced Polymer Composites for Structural Applications in Construction, R Shenoi, S. Moy, L. Holloway, Eds., pp. 445-455