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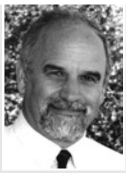

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# Performance of a stress-laminated-timber arch bridge

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**For the past three years the authors have been involved in the optimisation of the performance of stress-laminated-timber arch (SLTA) structures by utilising the strength properties of timber in an arching action for use as vehicle and pedestrian bridges. During this time over 20 permanent bridges have been built and eight have been load tested. The overall aim of this extensive research programme has been to develop structural uses for low-grade, UK-grown, timber and it has been shown that arches, using timber in compression, are an extremely effective technique for bridges. Timber structures have a very high sustainability value while being low cost and employing less early capital. These bridges on public roads can help increase public confidence in timber as a viable structural material. As part of a series of field and laboratory tests on SLTA bridges, a 20 m span arch bridge was designed and constructed at the Glentress Forestry Commission site near Peebles, in August 2004. The bridge has since been subjected to a series of extensive static and dynamic loads evaluating its response to crowd and vandal loadings. The results have confirmed predictions that the strength and stiffness of this type of construction was well beyond the strength normally expected from a slender timber structure. This paper details the construction and compares the analysis, design and load testing of the latest 20 m span full-scale SLTA test bridge at Glentress. The extensive testing programme, augmented by analytical work, aims to develop reliable design guidelines for arch structures using UK softwood.**

## 1. STRESS LAMINATION

### 1.1. Basic configuration

Mechanical stress lamination of timber is a technique in which a number of individual sawn sections of timber are compressed together by high-tensile steel bars to form a large load-sharing member or orthotropic plate. The high-yield steel (HYS) bars are passed through predrilled holes in the wide face of the timbers which are laid side by side on their narrow face. The bars are jacked against anchors on the outside timbers which have to be hardwood to sustain the very high local bearing stresses. Load is transferred from one laminate to the next by friction forces between them which make the whole into a solid load-bearing timber deck with the ability to transfer load laterally and longitudinally. The internal laminates are generally softwood.

### 1.2. History

The technique was developed during the 1980s in North America[1,2] mainly for replacement bridge decks. During the 1990s further developments were made in a number of European countries and in Australia.[3–5] Stress lamination is a very efficient way of sharing and distributing load which means that low-grade short lengths of variable quality timber can be used, as the natural strength-reducing characteristics of timber are dispersed throughout the orthotropic plate.

All developments in mechanical stress lamination of timber for bridge decks have used flat decks or beams in bending. They have either been plate decks, built-up decks or cellular decks. The plate decks can only span to about 6 m using full highway loading and normal maximum timber sizes, up to around 250 mm deep. Due to the restriction on maximum available timber sizes the built-up and cellular decks were developed to span further while supporting the same highway loads.[3,5] These decks however entrap moist air which can create a rot problem.

Various design rules have been developed by a number of researchers to deal with butt joints and lateral transfer of loads to produce reliable bending and shear resistance for bridge decks for heavy highway wheel loads.[4,5] However, several limitations remain if this form of construction is to be successfully used in the UK.

Prior to mid-2002 there had been no known examples of stress-laminated-timber bridge structures in the UK. Initial investigation was prompted by a need for low-cost forestry and rural public road bridges. These had originally been built as stone arches and, traditionally, were replaced by steel and concrete. Home-grown timber is now plentiful in the UK, although the quality and sizes are limited. Mechanical stress lamination techniques similar to those used in the USA and Australia looked to be of interest.

As mentioned, these bridges have generally been favoured for short flat rural spans mainly because of need and the limits of available timber sizes. Derivations have been developed for longer spans using cellular construction[5] and as a composite with inverted steel 'T' beams[6] and by stressing glued-laminated beams.[7] The span limitation was immediately a problem compounded by the size limitations of UK-grown timber. Built-up and cellular decks were considered but neither has an immediate future in the UK because there is no established glue-lamination industry to produce beams for built-up decks and the climate rules against cellular decks. The UK is much wetter than other locations at which these structures have been used and, as a result, rot would become a problem through poor drying and ventilation. This led the authors to investigate the implementation of stronger engineering properties of timber (compression and end-bearing) in an arching action which would avoid and surpass the limitations of decks in bending.

## 2. RESEARCH PROGRAMME

### 2.1. Markets

In the UK, particularly in Scotland, plantation timber production is increasing in an atmosphere of saturated traditional markets and low-cost imports. Home-grown timber needs new high-value markets and more trained timber engineers. This has resulted in many new initiatives including the first dedicated Centre for Timber Engineering at Napier University. Coincidentally the minor public road system in the UK is underfunded, as in many other countries, and low-cost replacement bridges are needed. In the UK, old minor public road bridges are of stone arch construction, which stands up to light modern traffic but many rural areas are now host to very large agricultural vehicles and 44 t forestry lorries. Stress-laminated-timber bridges could provide a solution.

The major developing market for Scottish timber is housing but there is public resistance to investing in a house built from a material which is perceived to rot. By constructing showpiece bridges it is hoped to restore public confidence in timber as a viable structural material. Examples of recent stress-laminated-timber arch (SLTA) bridges are shown in Fig. 1.

Fig. 1. Examples of recent stress-laminated-timber arch bridges

### 2.2. Trial bridges

A trial, 6 m span arch was built in 2002 from 100 × 50 Sitka spruce timbers as an exhibit for a forestry show and later tested in the laboratory. The results were beyond expectations and displayed the expected structural arch behaviour. Therefore a 15 m span, of similar geometry, was built and load tested.[8] The results confirmed the findings of the previous test so a number of 2·2 m span bridges of variable geometry (rise) were constructed in the laboratory and tested for load and friction effects with differing tensions to further understand their arching effects.[9]

Encouraged by the results, a 20 m span arch bridge was designed and constructed, first for demonstration at the Royal Highland Show in Edinburgh (June 2004) and then transported for testing to the Glentress Forestry Commission site near Peebles, in August 2004. The bridge has since been subjected to a series of static and dynamic loads evaluating its response to crowd and vandal loadings. This paper details the construction, load testing, stiffness and strength properties of the latest 20 m span full-scale, SLTA test bridge at Glentress.

Currently a series of 6 m spans, with varying geometries, are being built in the laboratory to confirm all of the past findings, explore lateral load distribution and evaluate their effects on the deck stiffness.

### 2.3. Commercial and future bridges

During the three years of the test programme some 20 commercial footbridges have been built to similar specifications and have helped develop confidence in all aspects of the design. The construction techniques have also been improved, which will be invaluable for future projects. These are to feature in the intended design guidelines for stress-laminated arch structures using UK softwood.

Examples of developments and built efficiencies include the following.

(*a*) The foundations for these bridges provide the thrust to support the arch so they must be strong and stable. However, because the structures are not rigid masonry arches, they can tolerate some movement, which will lead to more efficient foundation design in future.
(*b*) *In situ* building methods have been developed for long spans which utilise the initial arching action and thus avoid the need to support the entire weight of the deck on scaffolding.
(*c*) Bridges up to 10 m are better built in a workshop and craned into position.
(*d*) Details have been developed for fixing handrails, shedding rainwater, preservative treatments and waterproofing the deck to avoid rot.

The programme is designed to form a sound basis for the design of SLTA so that heavy vehicle bridges can be designed for secondary public roads. These are most likely to be of the form of an arch supporting a flat stress-laminated running deck. One commercial footbridge has already been built to this design to trial the system (Fig. 1).

## 3. CONSTRUCTION OF THE TEST BRIDGE

### 3.1. Test bridge span

Having successfully built and tested a 15 m span with 250 mm timbers and established that elastic linear analyses provided comparative deflections it was assumed that this would hold true for larger spans. Commercial orders were received for three 20 m span arch bridges so the decision was taken to build a test bridge to full scale (20 m span) and use it as an exhibit at the June 2004 Royal Highland Show (RHS) in Edinburgh.

Analyses and design calculations were carried out using a countryside crowd loading of 3·2 $kN/m^2$. This permitted a 200 mm deep deck (an optimum solution in regard to material availability) with a natural frequency of just over 4·0 Hz.

### 3.2. Loading

Design loading for footbridges in the UK is governed by BS 5400,[10] various Department for Transport memoranda and recommendations in codes of practice. The British Standard gives only one uniformly distributed load, 5 $kN/m^2$, for the design of the main structure but this is aimed at urban locations. Various codes suggest 2·5 $kN/m^2$ for normal loading and 3·2 $kN/m^2$ for crowd loading in the countryside, both of which have been shown to be adequate over the past 25 years. The design loading chosen for this bridge was 3·2 $kN/m^2$.[11]

However, slender structures designed for lighter loads will have low values of natural frequency, which, along with low stiffness and mass can be excited easily. In this context it is vandal loading which could become critical. Other research[12] indicates that the maximum frequency to which a small group of vandals can synchronise their jumping is 2·5 Hz, therefore the natural frequency of a small footbridge should be greater than this to avoid the possibility of vandals damaging the bridge by coordinating their load frequency with the natural frequency of the bridge and causing resonance.

### 3.3. Stability

In order to increase the overall stability of the structure it was decided to widen it to about 3 m at the supports, reducing to a width of 2 m at mid span; as it would also provide a more attractive shape to the bridge. Handrails were fitted, similar to those envisaged on the final structure, as this would have an effect on the overall stiffness of the bridge.

The test bridge was first built at the RHS ground and because foundations could not be excavated, tie bars connected the springings, or end bearings of the arch, which were formed using rolled steel channels and provided the lateral thrust. This same configuration had been used by the authors to form the lateral tie for the 15 m bridge previously tested and created some slip problems.[8] As expected, a relatively large amount of movement occurred when the tie bars holding the bridge ends together took the strain. This in turn caused a large increase in the vertical deflection of the bridge.

A great deal of care, in general, is required to ensure that the arch shape is correct from the outset. The construction at the RHS had started off too flat by a team of workmen. This resulted in significantly larger deflections and in turn caused a lower natural frequency which was to be expected but the effect on its 'liveliness' seemed disproportionate. This highlighted the importance of the geometrical shape (arch profile) of the structure in satisfying both serviceability and strength design criteria.

After the RHS the bridge was taken apart and reconstructed by four men in five days in a quarry at Glentress in Peebles in the Scottish Borders about 25 miles from Edinburgh. The foundations (mass concrete 400 mm wide by 750 mm deep) were anchored into the quarry base. They were purposely not built to the size that would be used in permanent construction to add a further facet to the tests. Some lateral movement was expected but would be realistic in terms of an actual structure and it would be measurable.

### 3.4. Timber

One important aim for this research is to find uses for home-grown plantation timber so Sitka spruce of C16 grade was used. The permissible compression strength of this material is within the requirements of the design. The bending capacity can be a limiting factor with high line loadings at the quarter point to one-third point inducing excessive bending stresses on the arch, depending on its rise. Sitka spruce is not durable so preservative treatment is essential followed by waterproofing of the deck for permanent bridges. All laminates are kiln dried to below 18% moisture content before being sawn and drilled accurately. The laminates are then pressure treated with preservative and allowed to dry again before construction. For commercial bridges up to this point in time Scots pine, larch or Douglas fir has been used because of their greater durability and treatability.

For the arch construction a single size of timber was used and staggered in groups of four to form a module which would repeat itself throughout the deck. Four holes were drilled in each timber at a diameter to easily allow the bars through. The holes were drilled on the radius of the circular curve of which the deck was a segment. In Fig. 2 the 20 m span bridge is shown during construction. The timbers were rough sawn 50 mm thick × 200 mm deep × 2 m long.

### 3.5. Stressing lateral bars

One of the important aspects of design for SLTA bridges is the function of the lateral stressing bars to achieve the necessary effects.

(*a*) Sufficient tension to stress all of the laminates.
(*b*) Sufficient friction to transmit longitudinal and transverse stresses.
(*c*) Sufficient extension to maintain tension when laminates dry and shrink.
(*d*) Resistance to corrosion.

Fig. 2. Bridge at Glentress during construction

Bars made from a high tensile steel are available at a reasonable cost so the first bridges were built using 900/1100 $N/mm^2$ Dywidag stressing bars at either 16 or 20 mm diameter. They are not normally protected although they can be especially ordered with electro-plating, which may fend off corrosion for 5 to 10 years. In the past some were sleeved and greased, as for ground anchors, but although this helped to achieve even tensioning the sleeves interfered with the building process. Eventually Gewi bars were favoured because they are galvanised and although their yield stress is only 500/600 $N/mm^2$ the bar strain to maintain tension was considered less important in an arch bridge[9] than in a flat bridge. The test bridge used 16 mm diameter Gewi bars stressed to 90% proof stress.

The tension on the bars has to create enough pressure to provide sufficient friction between laminates to transfer stress. This is a balance between providing just enough without overstressing the cellular structure of the timber, practical values needed to pull the deck laminates together and a reserve to allow for losses over time. The values for arches are very different from flat slabs. Research on flat decks[13] has shown 700 $kN/m^2$ as a maximum but in the knowledge that most of it could be lost over the first few months values of 1000 to 2000 have also been recommended.[14] Recent work in this programme at Napier University established that only minimal tensions will provide sufficient friction in an arch and recent construction work shows that a minimum amount of tension is required to overcome the friction between bar and timber. The moisture content, species and preservative treatment all have a significant effect on the tensions necessary to maintain friction. The bars in arches are at closer centres because of the curve and edge distances.

During the construction in order to pull the laminates together and maintain tension after relaxation, it was decided to apply 100 kN tension force to the bars, via bearing plates of 200 mm diameter galvanised mild steel plates, corresponding to transverse pressure of 1000 $kN/m^2$.

The bridge had been in place for three months before testing so the bars were retensioned. To create a further facet to the testing only half of the bars which were randomly selected were retensioned to examine the difference it might make in the stiffness characteristics of the bridge. For the subsequent loading, four months further on, all of the bars were retensioned so that the bridge performance under similar loading conditions could be compared.

### 3.6. Spacing of stressing bars

The spacing of the lateral tension bars is critical. They must be close enough to provide effective friction but they are expensive so their number must be minimised. Their centres are also related to the arch curve and depth of the deck. Laminate length is as long as possible without creating too large a projection on top of the deck. The holes through the laminates for the transverse bars are drilled off centre to create the arch shape and their centres are partly governed by how close they are to the edge of the timber. The optimum spacing governed by the parameters for this test bridge was approximately 500 mm.

## 4. TESTS AND RESULTS

As a result of the difficulties of measuring the lateral thrust for a full-scale bridge with *in situ* foundations and because the results from previous laboratory tests have shown thrusts to be as predicted by linear static analysis they were not measured. Deflection measurements were carried out under increasing loads up to well above design loads.

Vibration tests were carried out using three independent, but similar, techniques all employing accelerometers/transducers to measure the response to a hammer blow, crowd walking over or jumping on the bridge or sandbags being dropped onto the deck. These results were compared with the results from a finite-element analysis.

### 4.1. Static loading

In the first series of tests, displacement transducers were used to measure the deformation profile of the bridge under applied static loadings. Two transducers were placed at 2·5 m centres along the span of the bridge (14 in total). All readings were automatically recorded using a data-logging system. As has been mentioned earlier, the bridge had been in place for three months before testing and the stressing bars had relaxed considerably. To create a further facet to the testing only half of the bars which were randomly selected were retensioned. This was to examine the difference it might make in the stiffness characteristics of the bridge.

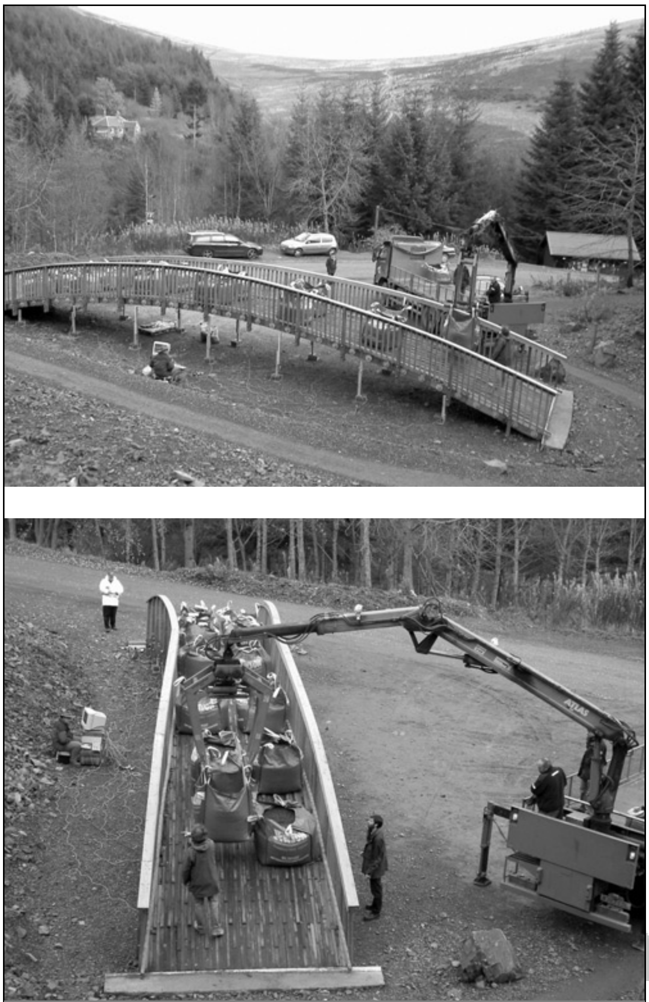

Fig. 3. Bridge at Glentress during testing

First, 50% of the loading was used to settle and bed down the bridge. The loads were then removed and transducers were adjusted and zeroed. The bridge was then loaded using 9 kN bags of sand placed by the hydraulic arm of the delivery lorry.

Fifteen bags were used to apply 135 kN as a uniformly distributed load (UDL) to simulate the 3·2 kN/m$^2$ design load which totals 128 kN over the 20 m span and 2 m wide deck (Figs 3 and 4), and deflections were recorded at each increment of loading. During unloading, readings were taken after each bag was removed. The results of these tests are illustrated in Fig. 5 and are compared with the stepwise linear analyses where at each step nodal coordinates of the bridge were updated to reflect the new geometry due to deformed profile.

The middle third of the bridge was then loaded with 14 bags (126 kN) and then unloaded using the same procedure. This represented approximately three times design load. The bridge sustained this load with no sign of any distress. On removal of the applied loads the bridge recovered over 80% of its maximum deflection.

This was then followed by applying up to eight bags of sand (72 kN) at the quarter point. In Fig. 6 details of the loading and the load–deformation behaviour of the bridge at the quarter-point loading up to 72 kN load are shown. The effects of uneven tension in the stressing bars are somewhat illustrated in Fig. 6(b) as the structure exhibits a characteristic unique to stress lamination (due to flexibility of the deck at various points along its length) as load is increased. The results of these tests are further illustrated in Fig. 7 and are compared with the stepwise linear analyses. With increase in load, it was noticed that due to extreme horizontal thrusts, the small strip concrete foundations started to slip/rotate. The magnitude of support slip at the loaded end was approximately 3 mm and at the unloaded end about 5 mm, both outwards (Fig. 8).

As a check on the most critical loading and to test the suitability of remote deflection measurement equipment for future tests, a further load test was carried out four months later, after the bridge had had time to readjust/recover from the extreme loadings. The quarter-point loading, as before, was used. The remote measurement was carried out using a SOKIA 4130R3 Total Station which measures any point in three dimensions to an

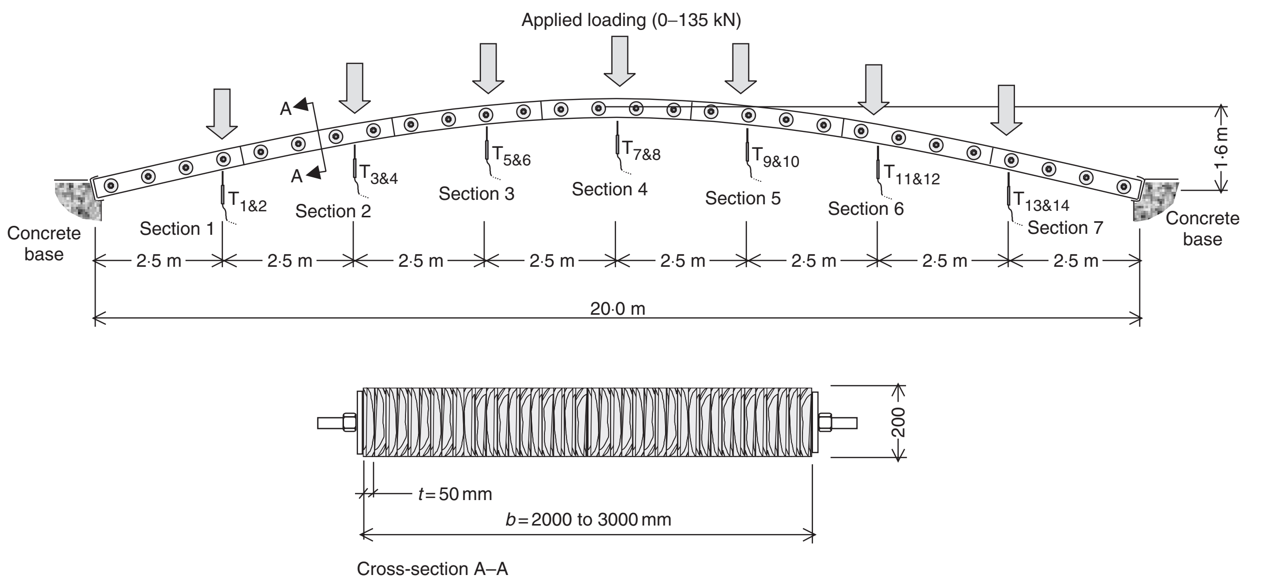


Fig. 4. Details of UDL test. Notes: Displacement transducer positions $T_{1\&2}$, $T_{3\&4}$, $T_{5\&6}$, etc. for vertical deformation. Each position represents two transducers—one on either side

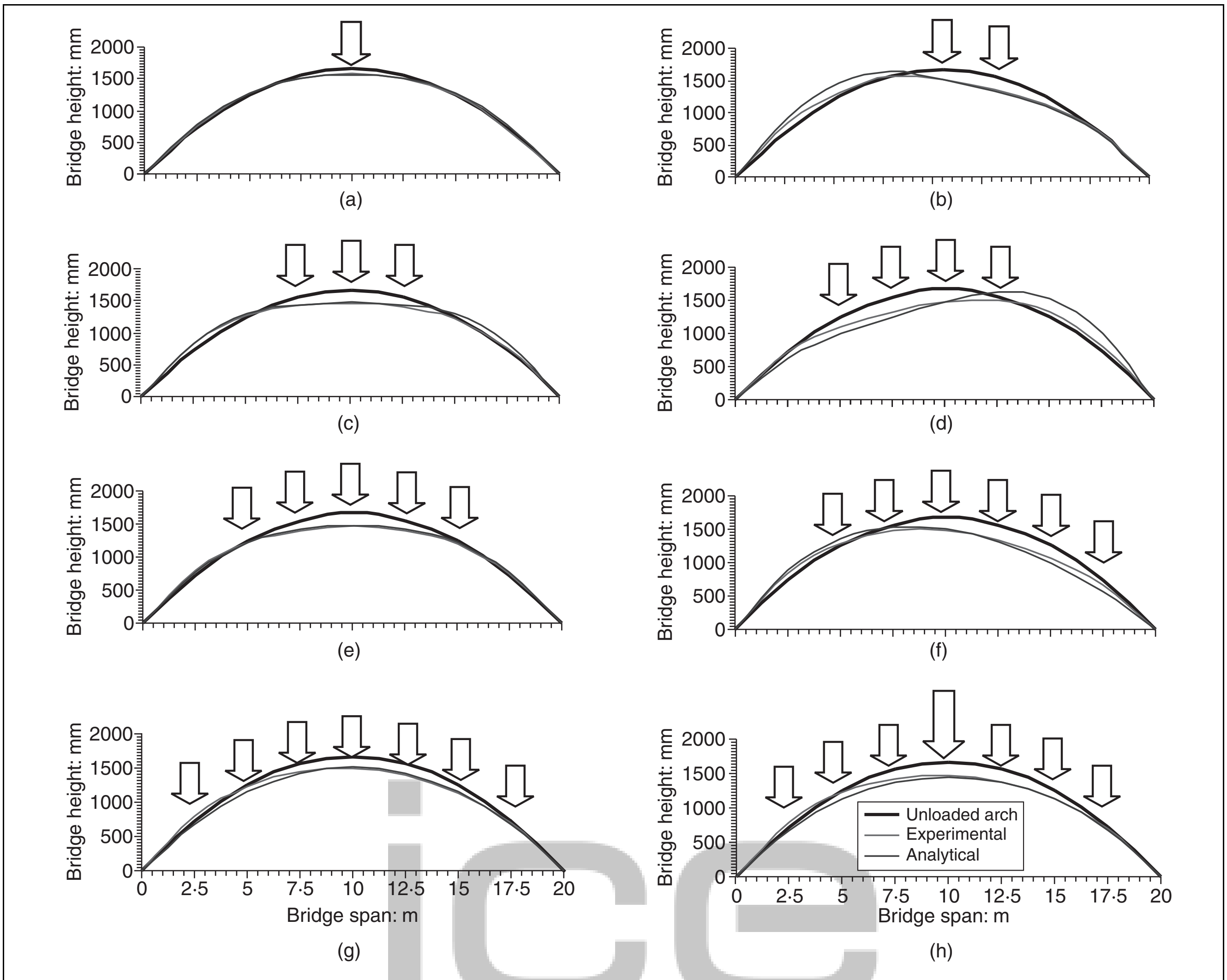


Fig. 5. Deformation profile under incremental UDL loading: comparison of analytical and experimental results—deflections are exaggerated for illustration: (a) load = 18 kN; (b) load = 36 kN; (c) load = 54 kN; (d) load = 72 kN; (e) load = 90 kN; (f) load = 108 kN; (g) load = 126 kN; (h) load = 135 kN

accuracy of 1 mm. For this test, all stressing bars were retensioned to 100 kN. The results are illustrated in Fig. 6(c) and in general gave a good comparison with those in the previous test (Fig. 6(b)) where only half the bars had been retensioned. The comparison of Fig. 6(b) and (c) further confirmed the previous findings of this programme that in an arch construction the level of stress in the bars is not as critical as those in flat bridges.[9]

### 4.2. Vibration tests

The first set of vibration tests were carried out using four vertical and two horizontal Pinocchio Vibraphones connected to an eight-channel TEAC LX10 data recorder in conjunction with ARTeMIS test planner and modal analyses software. The excitation required was provided by a crowd walking steadily over the bridge (Fig. 9). The measurements were made in the sequence shown in Fig. 10. Dashed arrows indicate free-moving sensors, and solid arrows indicate reference sensors.

In Table 1 the number of records used, the number of samples in each record for modal analysis and the Nyquist frequency are detailed.

In ARTeMIS Analyser the data were processed with a default signal processing configuration including a 1024-line spectral density estimation. Fig. 11 shows the singular values of the spectral densities of the third measurement. During the measurements modal analyses were made using the fast frequency domain decomposition peak picking technique. This was for quality checking of the data as well as verification of the sensors and their positions.

The first six modes were found and are shown in Table 2.

The second set of the dynamic tests involved the use of a dual spectrum analyser and impact excitation method. The impact hammer was used to excite the structure. The response was recorded using two accelerometers. Both the excitation force and the response signals were recorded using a multi-channel spectrum analyser. The results were analysed both in time and frequency domains. The fundamental frequency, using this method, was found to be 3·60 Hz.

The third set of tests was carried out four months later using a versatile handheld vibration analyser RT440 developed by

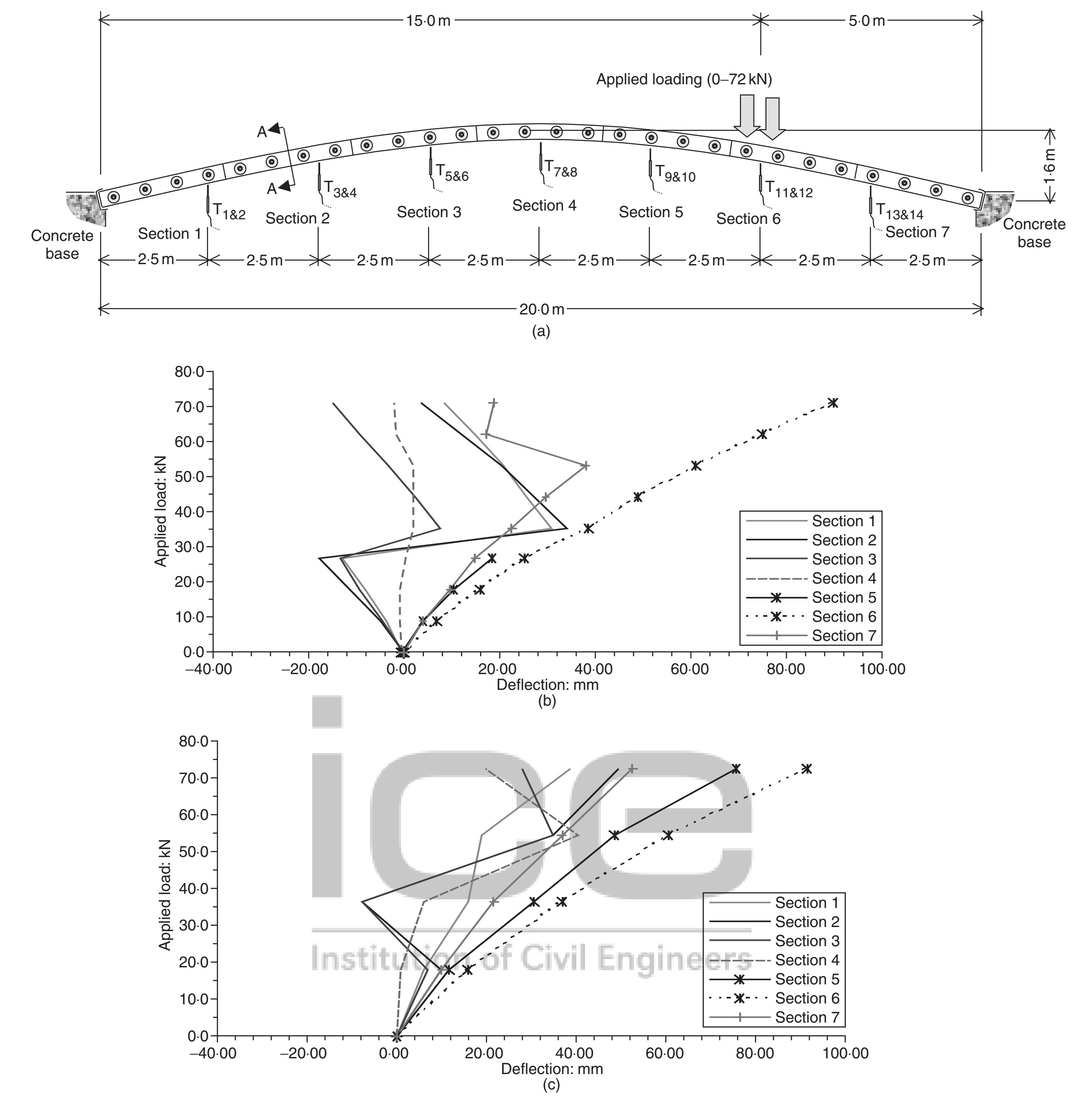


Fig. 6. Applied quarter-point loading of 0–72 kN using 9 kN sandbags: (a) bridge and loading details; (b) load–deformation when half of the stressing bars were randomly selected and tensioned; (c) load–deformation when all of the stressing bars were tensioned (four months later)

Reactec Ltd. The bridge was excited by impact (including a broadband frequency spectrum thus exciting any system natural frequency) using 50 kg bags of ballast and the response was measured utilising the RT440 'bump test' module. Several measurements were taken at both sides and the natural frequency of the bridge was again found to be approximately 3·5 Hz.

The experimental results obtained from the three different methods (including both input and instrumentation) compared very well with each other, indicating that the fundamental natural frequency of the bridge, without any topping of bitumen macadam and backfill at the abutments to reduce slope for access, is approximately 3·5 Hz. As mentioned earlier, a simplified finite-element analysis carried out to estimate the natural frequency of the bridge, assuming it as a single mass of homogeneous material and ignoring the laminate slip and the flexibility within the composite mass of the deck, was just above 4·0 Hz.

These values are close to the frequency possible by vandals (2·5 Hz) but resonance will not occur as ten people could never impart sufficient energy to an 8 t structure at 1 Hz over the optimum they can normally impart. At these values this parameter could be considered critical in the design of a footbridge but with a dense bitumen macadam topping as waterproofing the fundamental natural frequency (FNF) will increase beyond the critical zone.

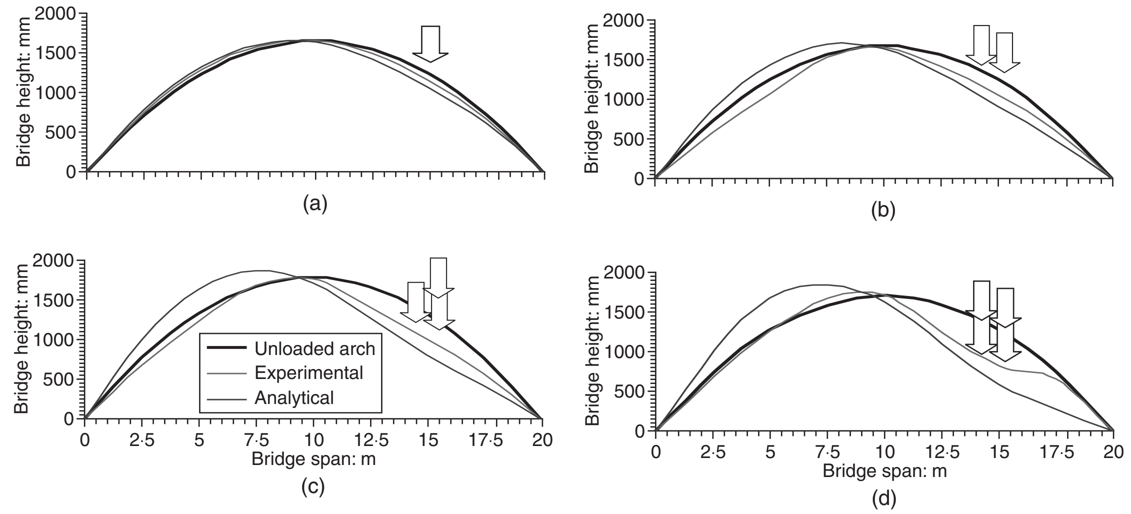


Fig. 7. Deformation profile under incremental quarter-point loading—comparison of analytical and experimental results—deflections are exaggerated for illustration: (a) load = 18 kN; (b) load = 36 kN; (c) load = 54 kN; (d) load = 72 kN

Fig. 8. Bridge at Glentress: effect of foundation slip (crack opening near a support) under the full quarter-point loading

Fig. 9. Bridge at Glentress during dynamic testing: excitation by crowd walking

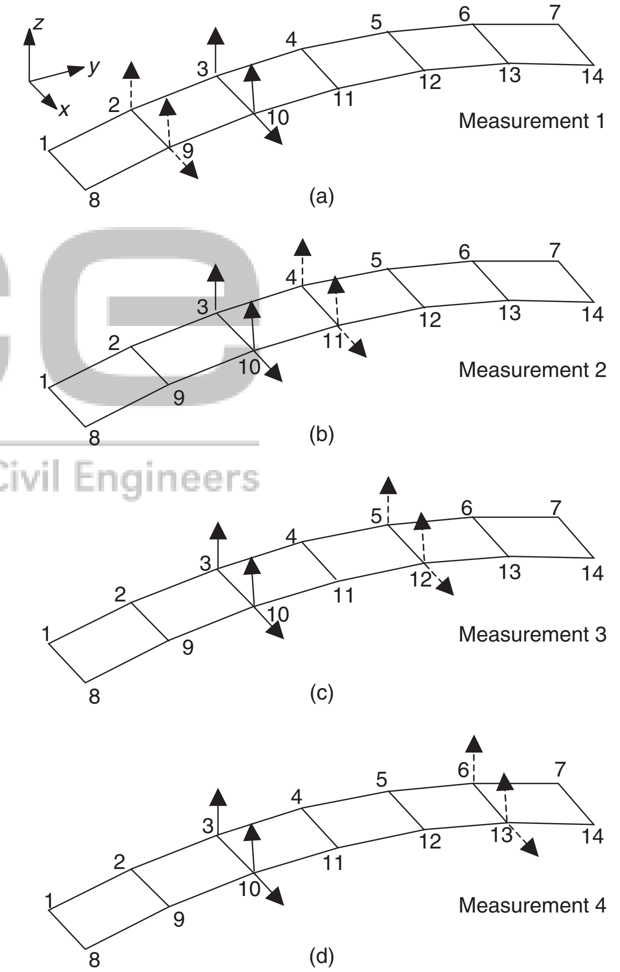


Fig. 10. Bridge geometry: data set measurements



## 5. PRESERVATIVE TREATMENT OF TIMBERS

Most external timbers are treated with preservatives to reduce their tendency to rot. Some species are more durable and others take treatment more easily so the permutations increase. There is great benefit from having the facility to pressure-treat each

| Description | Data |
|---|---|
| No. of records (degrees of freedom) | 6 |
| No. of samples in each record | 5000 |
| Duration of each record: s | 97·66 |
| Nyquist frequency: Hz | 25·6 |

Table 1. Acquired data

| Mode number | Frequency: Hz | Stand dev. freq.: Hz | Damping ratio: % | Stand dev. damp.: % |
|---|---|---|---|---|
| 1 | 3·54 | 0·07893 | 3·046 | 1·098 |
| 2 | 6·25 | 0·02062 | 2·115 | 0·4998 |
| 3 | 6·535 | 0·02934 | 2·111 | 1·013 |
| 4 | 8·103 | 0·05948 | 1·458 | 0·6083 |
| 5 | 10·1 | 0·07868 | 2·5 | 0·7061 |
| 6 | 11·29 | 0·0547 | 1·771 | 0·7613 |

Table 2. First six modes estimated using the UPC, stochastic subspace identification technique

laminate after sawing and drilling and before construction. The result is a solid mass of timber with treatment throughout which is not possible with large primary timbers. To ensure more reliable treatment and reduce stress loss through time the timbers are kiln dried to 18% before treatment. They are then allowed to dry again before being stressed in the structure. This can be done by kiln drying again or allowing to air dry for a number of weeks.

Copper chromium arsenic treatment was the most common until it was outlawed for general use in June 2004 to come in line with EEC legislation. A derogation in the Regulation permits its use, along with creosote, for timber bridge decks but establishments providing these treatments are difficult to find so copper chromium phosphate is normally used now. In a way this is convenient because this less toxic treatment would have to be used for the handrails whatever was used on the deck to ensure the public does not touch the timber and lick their hands. It is advisable to detail the structure so that the main timbers are kept dry during service even with preservative treatment.

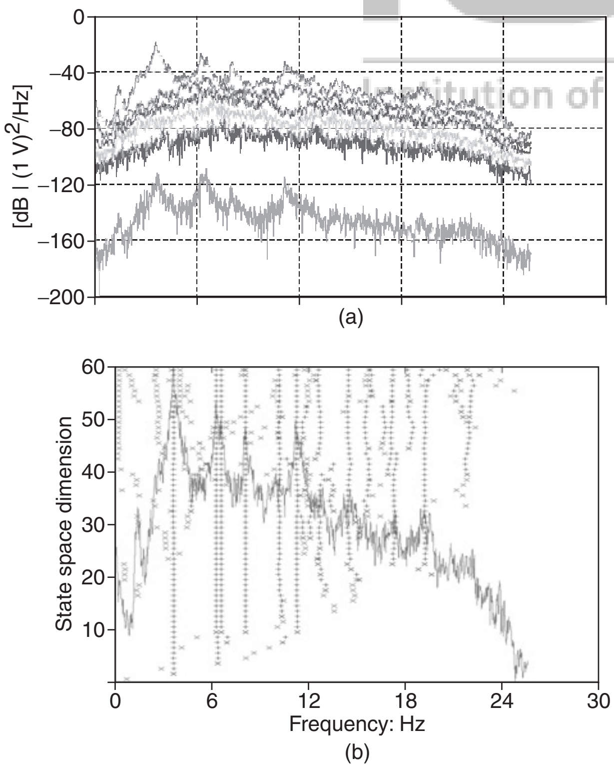


Fig. 11. Dynamic response: sample output for measurement 3: (a) singular values of the spectral densities; (b) stabilisation diagram (unweighted principal component, data driven)

### 5.1. Waterproofing of decks

Timber bridge structures are becoming more popular in many countries around the world. The reasons are a mixture of aesthetics, economics and sustainability. Timber bridges were common until 100 years ago when other more durable materials became available so, today, timber needs to last longer to compete. There are many timber bridges which have lasted 100 years and they all have one thing in common—the timber has been kept dry. Many of the surviving bridges have roofs to shed snow and, although not the original design concept, the roof has kept the structure dry. Today roofs cannot be built high enough for vehicles but there is an argument for roofs on footbridges. However there are simpler ways to waterproof the deck. This is done on these bridges by applying a sealer tack coat and finishing with a dense bitmac as a wearing course. This is thickened locally at the abutments to reduce the steep gradient at the end of the segmental arch. This also helps to increase the stiffness and thus the FNF.

### 5.2. Costs

Some 20 commercial bridges have been built in the field and eight scaled and full-size test bridges in the laboratory. As construction techniques are tried and tested costs are reducing. There will be large variations in costs depending on familiarity with the technique but the system holds the promise of very low-cost structures because the materials are not expensive, they are easily available and the skill level required is low.

The most recent 20 m span bridge over the River Forth cost about £500 per m$^2$. This would break down to

| | |
|---|---|
| timber, stressing bars and deck coating | £140 |
| foundations | £100 |
| scaffolding | £120 |
| labour | £140. |

In the future it is unlikely that the decking materials will be reduced in cost but there is scope for improving foundation design and there will always be a balance between *in situ* construction or workshop and crane. As footbridges they are competitive but it will be the development of SLTA vehicle bridges that will show the greatest savings.

## 6. CONCLUSION

The work reported in this paper further proves the value that SLTA structures can provide to the UK forest and recreation industries. The use of the arch utilises timber in compression and end bearing, which are some of its best properties. It permits long spans that have only been achieved elsewhere by creating more expensive composite constructions such as glulam or box beams which are either expensive or would harbour moisture in the UK thus promoting rot. Properly detailed SLTA bridges in the UK can have a design life of 50 years.

The results of this research have confirmed predictions that the strength and stiffness of this type of construction is well beyond the strength normally expected from a slender timber structure; as the bridge sustained loads well above its design load without sign of any distress.

The fundamental natural frequency of the bridge was approximately 3·5 Hz. This compared well with a finite-element analysis value of just above 4·0 Hz.

This study has aimed to create the foundation for a detailed understanding of the actions of stress-laminated arches. Current laboratory investigations at Napier University aim to explore the influence of several factors such as creep, moisture fluctuations, arch profile and stressing conditions on the stiffness, strength and dynamic response of the SLTA bridges.

## REFERENCES


1. Ritter M. A. *Timber Bridges: Design, Construction, Inspection and Maintenance*. US Department of Agriculture, Washington DC, 1990, EM 7700-8.
2. Taylor R. J., Batchelor B. and Van Dalen K. *Pre-stressed Wood Bridges*. Ministry of Transportation and Communication, Research and Development Branch, Downsview, Ontario, Canada, 1983.
3. Ritter M. A. and Lee P. D. H. Recommended construction practices for stress-laminated wood bridge decks. *Proceedings of the International Wood Engineering Conference*, New Orleans, USA, October 28–31 1996, Vol. 1, pp. 237–244.
4. American Association of State Highway and Transportation Officials. *Guide Specification for the Design of Stress-Laminated Wood Bridges*. AASHTO, Washington DC, 1991.
5. Crews K. L. Design procedures for stress laminated timber bridge decks in Australia. *Proceedings of the 3rd Pacific Timber Engineering Conference*, Gold Coast, Queensland, Australia, 1996, **2**, 131–137.
6. Apple D. A. and Woodward C. *National Conference on Wood Transportation Structures*. US Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, WI, 1996, Report FPL-GTR-94.
7. Wacker J. P. Field performance of stress-laminated highway bridges constructed with glued laminated timber. *Structures Congress 2004—Building On The Past: Securing The Future*, Nashville, TN, USA, 2004.
8. Freedman G. and Kermani A. Stress-laminated arches: a stronger case for timber bridges. *ICE Proceedings—Civil Engineering*, 2004, **157**, No. 4, 172–178.
9. Freedman G. and Kermani A. Development of long span stress-laminated timber arch bridges. *Proceedings of the 8th World Conference on Timber Engineering*, Lahti, Finland, 14–17 June 2004, **2**, 301—306.
10. British Standards Institution. *Steel, Concrete and Composite Bridges—Specifications for Loads*. BSI, London, 1978, BS 5400: Part 2.
11. Countryside Commission for Scotland. *Footbridges in the Countryside*. Scottish Natural Heritage, Edinburgh, 1981.
12. Fletcher M. S. and Parker J. S. Dynamics of the Hungerford Millennium footbridges, UK. *ICE Proceedings—Bridge Engineering*, 2003, **156**, No. BE2, 57–62.
13. Crews K. I., Bakoss S. L. and Carter D. J. Fundamental research and development of cellular stress laminated timber bridge decks for short to medium span bridges. *Australasian Structural Engineering Conference Proceedings*, Sydney, September 1994, pp. 59–64.
14. Batchelor B. de V., Van Dalen K., Hachborn A. and Lee E. *Structural Characteristics of Transversely Prestressed Laminated Wood Bridge Decks*. Queens University, Kingston, Ontario, 1983, Project Report 8305, Part 1, Chapters 1–5, Part 2.





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