RECOMMENDED PROCEDURES FOR DETERMINATION OF DISTRIBUTION WIDTHS IN THE DESIGN OF STRESS LAMINATED TIMBER PLATE DECKS

Professor Keith Crews
Deputy Director, Centre for Built Infrastructure Research
University of Technology, Sydney, Australia

SYNOPSIS

Stress laminated timber (SLT) decks are constructed by laminating individual pieces of timber placed side by side (on edge), until a solid deck of the desired width is achieved. The laminating is achieved by compressing individual timber members together by applying a prestress in the transverse direction, which "squeezes" the individual pieces of timber together, creating an orthotropic plate.

A number of approaches have been adopted for modelling the orthotropic behaviour of SLT plate decks. BS EN 1995-2: 2004 presents three of these as basic methods for design of SLT plate decks – orthotropic plate methods, grid (or grillage) modelling and the so called simplified method, which uses the concept of distribution width, to design the deck as a "wide beam."

This paper discusses the basis for modelling deck behaviour adopted in North America, Australia and Europe and compares the various predictions of distribution width, for a given material. Modification to some aspects of BS EN 1995-2: 2004 (Eurocode 5: Design of timber structures – Part 2: Bridges) are recommended that would lead to increased efficiencies in design using the "simplified method", based on the results of research in North America and Australia.

1 INTRODUCTION

Stress laminated timber decks are constructed by laminating individual pieces of timber placed side by side, until a solid deck of the desired width is achieved. The laminating is achieved by compressing individual timber members together by applying a prestress in the transverse direction as indicated in Figure 1, which "squeezes" the individual pieces of timber together, creating a solid timber deck.

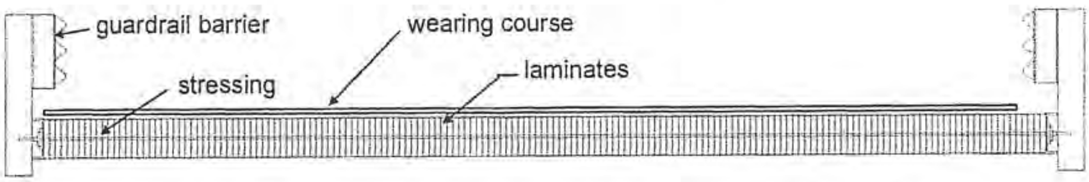

Figure 1 - Section through a Stress Laminated Timber Plate Bridge Deck

The system serves a dual purpose in providing a deck which is both the support for the wearing surface and also the structural system which resists the imposed loads and transfers resultant forces through to the substructure.

Once the prestress force is applied and maintained at or above the minimum design level, the stressed deck will behave as an orthotropic plate, effectively resisting loads, since these can be distributed laterally across some finite width of the deck (the distribution width) and then transferred longitudinally to the sub-structure. The most critical factor for design and maintenance of stress laminated timber deck systems, is to achieve and maintain adequate prestress force between the laminates so that the orthotropic plate action is maintained.

The compression force due to prestress allows transfer of vertical shear between the laminates through friction and also resists transverse bending which is induced by the loads, as shown in Figure 2. The initial compressive stresses commonly used in practice (in Australia) are of the order of 1200 kPa (1.2 N/mm^2) between the timber laminates.

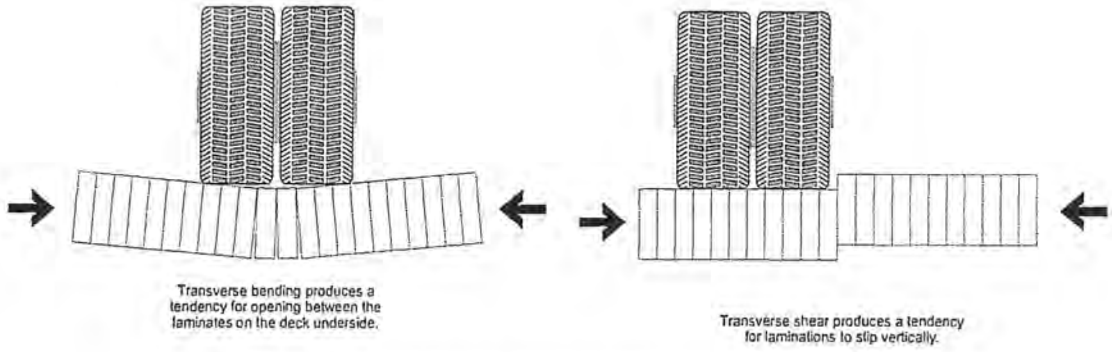

Figure 2 - Structural Actions of a Stress Laminated Timber Plate Bridge deck

2 DESIGN PROCESS FOR SLT PLATE DECKS

The critical phases in designing an SLT deck are:

- Designing the timber elements to have adequate stiffness and strength under flexural loading. This requires the use of a suitable model to represent the orthotropic plate behaviour and must also account for the effect of non continuous laminations (butt joints). Shear effects for longitudinal decks are not significant and are not considered.
- Selecting a suitable stressing system – bar sizes and spacings, such that the desired pressure between laminates is obtained, whilst minimising the effects of prestress losses
- Designing an anchorage and bearing system that transfers the post tensioning load into the deck
- Ensuring that the timber has adequate perpendicular to grain compression capacity to resist the prestress forces

3 MODELLING DECK BEHAVIOUR

A number of approaches have been adopted for modelling the behaviour of SLT plate decks. The basic analysis methods which can be used are as follows:

- Grillage Models
- Finite Element Models
- Orthotropic plate Models
- Modified Beam Models

BS EN1995-2: 2004 presents three of these as basic methods for design of SLT plate decks – orthotropic plate methods, grid (or grillage) modelling and the so called simplified method.

3.1 GRILLAGE MODELS

Grillage models are widely accepted for use in modelling and designing concrete decks. As such, some bridge designers adapt a grillage to model SLT decks and the use of grid or grillage is an acceptable means of analysis under Section 5.1 of BS EN1995-2: 2004.

In a study investigating the applicability of grid modelling, West [*1993*] demonstrated that calibrated grillage models can be used to accurately predict deflections and approximate moment distributions, particularly longitudinal moments, in stress laminated timber decks. However, calibration of material properties for lateral stiffness was necessary and the study concluded that the use of grillage models for design of SLT decks was considerably more complicated and time consuming than alternative methods, without providing any significant advantages for design.

3.2 FINITE ELEMENT MODELS

Finite element models can also be used in a similar way to orthotropic plate and grillage models for predicting plate performance with a reasonable degree of accuracy. However, once again the mesh density determines the sensitivity of the model and accurate material properties in all principal axes are necessary in order to develop a reasonable model which gives good predictive results for in-service performance of a stress laminated timber deck. In addition, modelling the effects of prestress levels on transverse stiffness can also be problematic.

Both grillage and FEM methods are numerically intensive to varying extents and are generally more sophisticated than is necessary in order for designers to get a reasonably accurate prediction of SLT deck performance.

3.3 ORTHOTROPIC PLATE MODELS

In order to undertake an orthotropic plate analysis, three essential material properties are required. These are the longitudinal elastic bending modulus of the material E_L (based on four point modulus of elasticity (MoE) testing), the transverse stiffness based upon the transverse modulus properties of a plate E_T (determined from transverse loading) and the shear modulus for the timber plate G_{LT} which is generally determined by twisting plate tests and other methods.

The first comprehensive research program involving determination of elastic properties for orthotropic SLT plates was undertaken at Queens University, Ontario. These tests were based on twisting "square" plates approximately 1200mm x 1200mm x 120mm thick, using a method proposed by Tsai [*1965*]. During the mid 1980's, extensive testing was also undertaken at the University of Wisconsin and the Forest Products Laboratory - Madison, to verify the Canadian test results, characterise the behaviour of plate decks and develop analytical modelling techniques for predicting behaviour of SLT decks.

This testing program involved simulated truck loadings on full scale single lane decks spanning up to 24ft (7.3m) and investigated the effect of different anchorage systems, stress bar spacings and levels of prestress on deck behaviour [*Oliva et al - 1990*]. The results quantified the effect of prestress on apparent longitudinal stiffness and load distribution (which the smaller Canadian test specimens had failed to identify) and characterised plate parameters by undertaking twisting plate tests similar to those completed at Queens University. However, the relationships determined from both programs for characterising the transverse stiffness and shear modulus plate parameters differed significantly, with the Queens University tests yielding values between 35% and 50% higher than the FPL results, for the serviceability range of prestress (350 to 700 kPa or 0.35 to 0.7 N/mm^2).

In the period 1990 to 1992, R&D undertaken at the University of Technology, Sydney investigated orthotropic plate properties for various locally used timber species, and these values are listed below in Table 1. Unlike the North American tests which were based on plates 1.2m square, the transverse stiffness and the shear modulus for Australian timbers were derived from testing plate decks 3.6 metres by 3.6 metres, with depths of up to 290mm.

Table 1 - Orthotropic Plate parameters (prestress of 700 kPa)

timber species:	*Average E_L (MPa or N/mm^2)*	E_T / E_L	G_{LT} / E_L
Seasoned Hardwood (planed)	18500	1.8 %	2.2 %
Australian Pine (planed)	12000	2.0 %	2.9 %
Douglas fir (sawn)	9500	1.5 %	2.5 %

A comparison of orthotropic plate parameters (based on a reference prestress level of 700 kPa) is presented in Table 2, which highlights the variability of results derived using different testing methods and different species. Whilst surface finish clearly has a significant influence on the values of E_T (as does the level of prestress) there is basic agreement on these values internally. However, there is much more variability inherent in the derived values for G_{LT}, which does not appear to be as sensitive to variation in the surface finish. Basically, the higher MoE timbers used in Canada, and timber species tested in the USA and Australia, all have ratio values about 3%, whilst the Canadian timbers with lower MoE's and the specified European values are twice these value at 6%.

Table 2 – International Comparison of Orthotropic Plate parameters

Region / timber	E_T / E_L	G_{LT} / E_L
Australian – sawn	1.5 %	2.5 %
Australian – planed	2.0 %	3.0 %
Canadian – high MoE	2.0 %	3.5 %
Canadian – low MoE	2.5 %	5.5 %
USA – sawn	1.3 %	3.0 %
BS EN1995-2: 2004 – sawn	1.5 %	6.0 %
BS EN1995-2: 2004 – planed	2.0 %	6.0 %

Whilst surface finish clearly has a significant influence on the values of E_T (as does the level of prestress) there is basic agreement on these values internationally, at about 2% of E_L, as noted in Table 2. However, there is much more variability inherent in the derived values for G_{LT}, which does not appear to be as sensitive to variation in the surface finish.

Orthotropic plate models have been widely used to validate plate behaviour observed in laboratory tests and on prototype bridges constructed in Australia and the United States. These models are very similar and have been calibrated using both laboratory data and load test data obtained from field load testing of prototype bridges (see Figure 3).

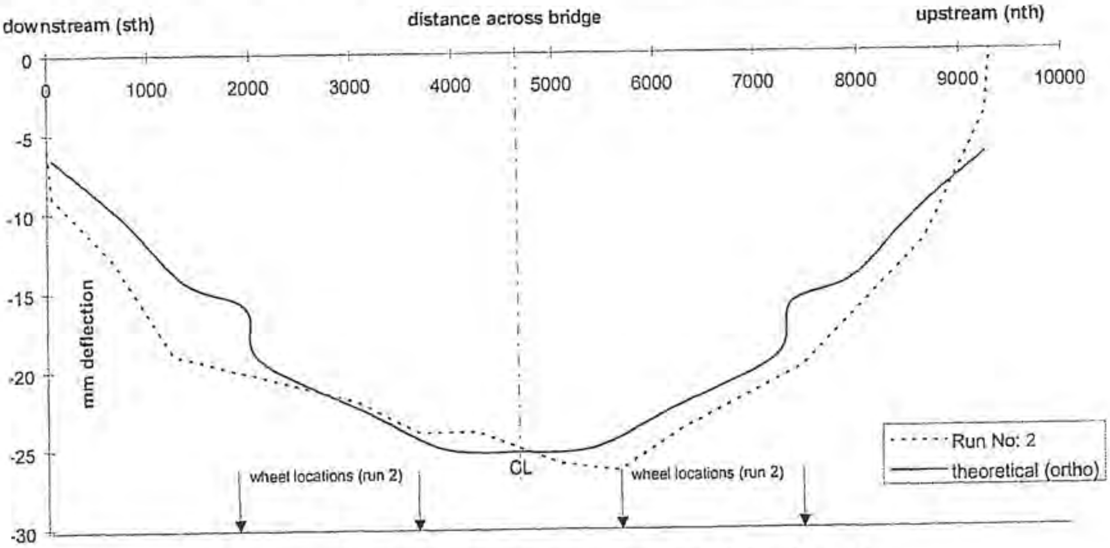

Figure 3 - Comparison of actual and predicted deflection plots

Whilst the orthotropic plate model provides a fairly accurate method of predicting deflections and moment distributions within a stress laminated timber deck (once the relevant plate properties are known), undertaking design using this analysis method is reasonably complex and can be computationally intensive. The method is also inherently sensitive to boundary conditions and as such it is best used as a research tool rather than as a routine design aid.

3.4 EQUIVALENT BEAM OR SIMPLIFIED ANALYSIS

The fourth method which is the most commonly used, is design the deck simply as a beam. The depth of this beam is equal to the deck thickness, whilst the width represents an idealised portion of the orthotropic plate deck over which the imposed loads (travelling along a wheel path) are distributed and resisted, as indicated in Figure 4.

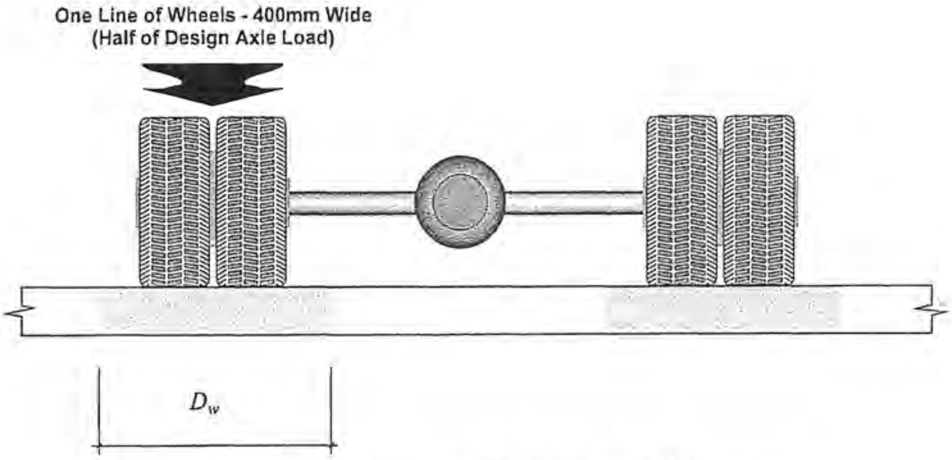

Figure 4 - Distribution widths for "normal" design vehicle

The width of this "equivalent beam" is known as the "distribution width" and it is used for predicting both the deflections (serviceability limit states) and longitudinal flexural capacity (ultimate strength limit state) of a stress laminated deck. As can be seen in Figure 3, the usual deflection profile for an SLT plate deck can be approximated by a parabolic distribution. However, this can be simplified by assuming that the peak deflection of the deck under load for the "equivalent beam" is constant across the distribution width, which is assumed to take all the load, whilst the remainder of the plate is assumed to carry no load and hence has deflections equal to zero. This is illustrated in Figure 5.

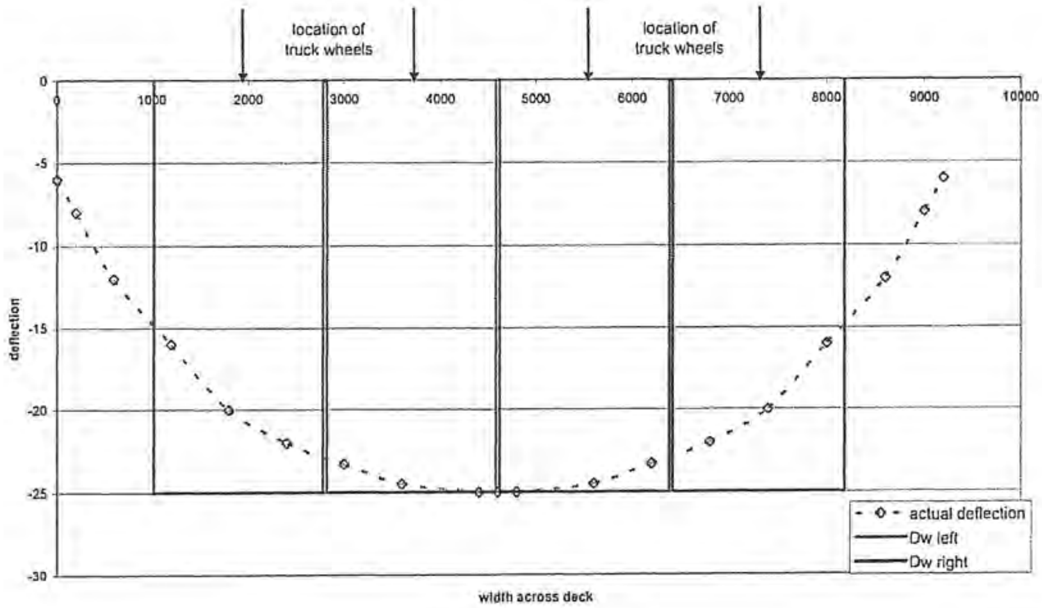

Figure 5 – Equivalent beams modelling the deflection of an SLT deck

Initial design procedures developed in the United States relied upon a series of nomographs for determining distribution widths, which were empirically derived as a function of the orthotropic plate properties and deck geometry. Most other design approaches involve use of a simple formula for estimating distribution widths. Either way, once the distribution width is determined, a stress laminated timber bridge deck can be analysed as either a simply supported or continuous beam.

The key element of these procedures and the extent of their accuracy is dependent upon the method used to determine the effective width of the beam. Provided the estimations for distribution width are representative of the true structural response of the bridge deck, modelling the decks as linear elastic beams is simple and the results obtained are accurate.

4 METHODS FOR DERIVING DISTRIBUTION WIDTH

Whilst the concept of a distribution width is familiar to most bridge engineers, the basis for derivation of such widths for SLT decks is not widely understood. Essentially, distribution widths specified in design codes for SLT decks have been derived in two ways; either by theoretical assumptions about load distribution in the deck, or on the basis of empirical data derived from testing.

4.1 SIMPLIFIED METHOD

The simplified method specified in Clause 5.1.3 of BS EN1995-2: 2004 is based upon a theoretical approach to the assumed dispersion of load through the timber laminations. For longitudinal laminations as used in most applications of SLT decks, the distribution beyond the edge of the actual wheel path is assumed to be 75 degrees to the horizontal. In other words, for a wheel path 400mm wide, and a deck depth of 300mm, the load dispersion is assumed to be approximately 561mm, and when combined with a distribution factor, the distribution width is 861mm, as indicated in Figure 6.

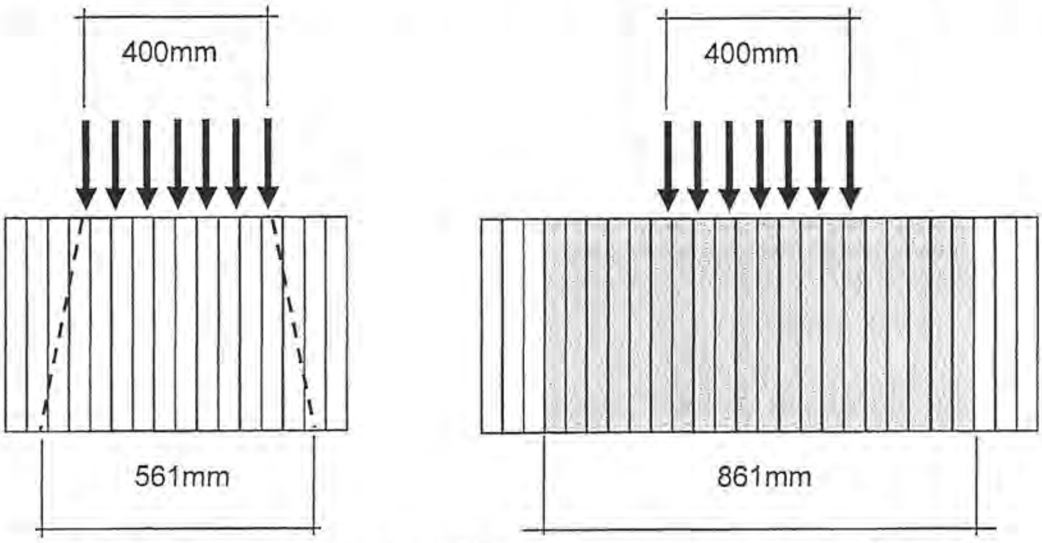

Figure 6 – (a) Dispersion of load, and (b) Distribution width – calculated using EN1995-2: 2004

4.2 DEFLECTION AREA METHOD

A relatively simple technique for predicting distribution widths from wheel path loads for plate decks is presented by Bakht [*1988*]. This involves using the deflections obtained from application of a wheel load and comparing the total area under the deflected shape of the orthotropic plate with the maximum or peak deflection, as indicated in equation 1:

$$D_t = \frac{A}{\Delta_{max}}$$ *equation: 1*

Where: D_t = the distribution width for a single wheel path load
A = the total area under the deflected shape (at the critical transverse section)
Δ_{max} = the peak value of deflection (at the critical transverse section)

Load testing of prototype bridges undertaken in Canada confirms that this method is acceptable, although perhaps slightly conservative. A similar technique has been used to derive the expressions for predicting distribution widths contained in the initial Australian limit states design procedures, using data obtained from extensive load testing on full scale bridge decks under both laboratory and field conditions. These expressions were based on a prestress of 1000 kPa (1.0 N/mm^2) (regardless of the timber species) which is the assumed serviceability prestress for Australian decks. For lower prestress levels the distribution widths calculated using equations 2 and 3 need to be reduced by 5% at 700 kPa (0.7 N/mm^2) and 10% at 550 kPa (0.55 N/mm^2).

The expressions for predicting distribution widths [*Crews - 1995*] are reproduced below:

Single lane bridges (normal vehicle – 400mm wheel width):

$$D_{wi} = \left(\frac{E}{10000}\right)^{0.5} + \frac{L}{25} + 0.45 \qquad \textit{equation: 2}$$

Two lane bridges (normal vehicle – 400mm wheel width):

$$D_{wi} = \left(\frac{E}{18000}\right)^{0.4} + \frac{L}{30} + 0.45 \qquad \textit{equation: 3}$$

Where:

D_{wi} = the effective equivalent beam distribution width (m) - based on a 1 in 4 butt joint pattern for normal vehicle loading

E = the modulus of elasticity (MPa or N/mm^2)

L = the effective design span (m)

4.3 DEVELOPMENT OF SIMPLE PREDICTION FORMULAS

Equations 2 and 3 were originally developed to take account of the significantly higher values for flexural modulus of elasticity that occur for native Australian hardwoods. However, subsequent testing indicated that variations in MoE did not greatly change the distribution width, whereas the effect of prestress is more significant. As such, the equations have been simplified (as indicated below), based on a lower serviceability prestress of 700 kPa (0.7 N/mm^2), to take account of creep related losses which gradually occur over time.

Single and Two lane bridges (one design vehicle – 400mm wheel width):

$$D_{wi} = 1.5 + \frac{L}{20} \qquad \textit{equation: 4}$$

Two lane bridges (two design vehicles – 400mm wheel width):

$$D_{wi} = \left(1.5 + \frac{L}{20}\right) \times 0.85 \qquad \textit{equation: 5}$$

The effectiveness of these equations is illustrated in Tables 3 and 4. In Table 3 the "observed" distribution width for each of three load tests on a prototype bridge indicated in Figure 7, is estimated using Equation 1 (column 4). Equations 4 & 5 have been used to calculate the predicted value of distribution width shown in column 5. It is noted that the Equation 4 can be slightly conservative for a single truck on a two lane bridge, although for narrower two lane bridges the ratio is generally about 95%.

Table 3 – Analysis of deflection data to predict Distribution Width

LOAD TEST:	*Maximum deflection (mm)*	*Approx Area (mm^2)*	*Estimated Dw*	*Predicted Dw*	*Predicted / Estimated*
Run 1 – One vehicle u/s	21.5	94460	2196	1980	90%
Run 2 – Two Vehicles	26	176600	1698	1683	99%
Run 3 – One vehicle d/s	20	91170	2170	1980	91%

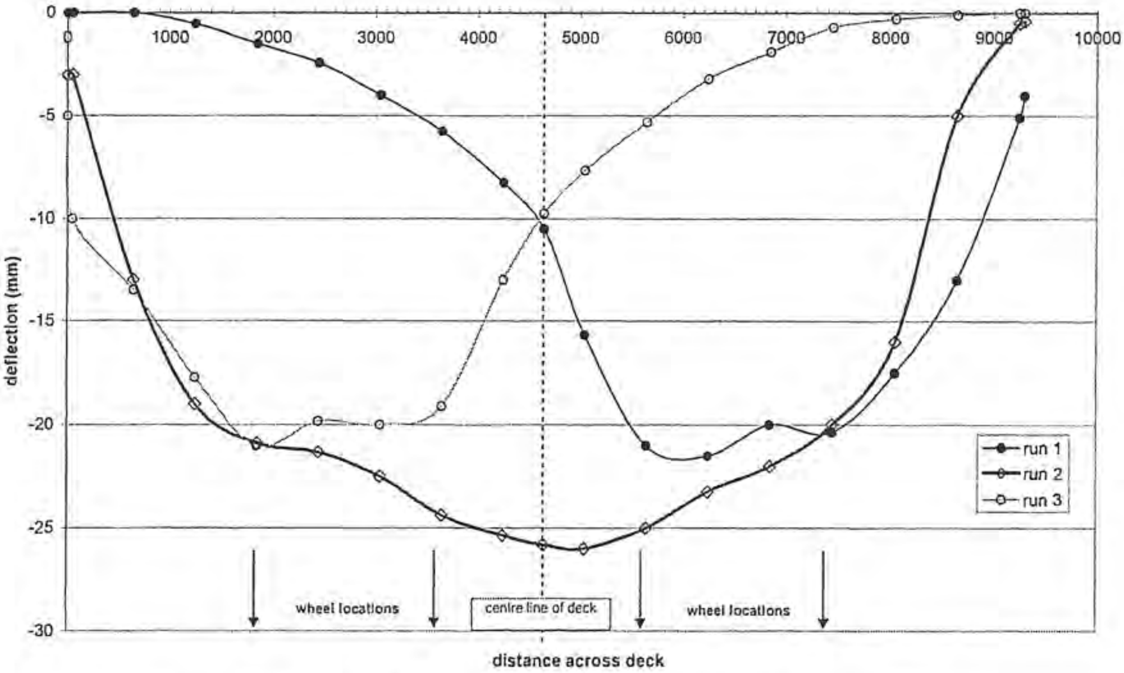

Figure 7 – Deflection data at midspan – Prototype bridge spanning 9.6m

Table 4 presents a summary of maximum deflection results obtained from testing 3 different plate bridges, where the distribution width has been estimated using equations 4 and 5.

Table 4 – Comparison actual and predicted maximum deflections, using predicted values of Distribution Width

Yarramundi Lagoon (800 kPa)	span (m)	depth (mm)	MOE (MPa)	Dw (m)	theoretical deflection	approx observed deflection	ratio (ob/th)
one lane loaded	9.3	290	18200	1.965	22.50	23	102%
two lanes loaded	9.3	290	18200	1.670	26.48	26	98%
McGraths Flat Lagoon (700 kPa)							
one lane loaded	9.6	340	12200	1.980	22.77	22	97%
two lanes loaded	9.6	340	12200	1.683	26.79	26	97%
McCarrs Creek (700 kPa)							
one lane loaded	6	290	12800	1.800	9.07	9	99%
two lanes loaded	6	290	12800	1.530	10.67	10.5	98%

4.4 COMPARISON OF INTERNATIONAL PRACTICES

A comparison of the various methods that have been used in North America, Australia and Europe for prediction of distribution width in SLT decks is presented in Figures 8 and 9. The Figures assume that the bridges have all been designed using softwood timber with an MoE of 10500 MPa (N/mm^2), with a live load deflection limitation of span / 400. The relevant equations from the OHBDC [*1991*] are reproduced below and it can be seen that the Canadian and Australian equations are quite similar.

Single lane bridges (one design vehicle – 400mm wheel width):

$$D_{wi} = 1.55 + \frac{L}{25}$$ *equation: 6*

Two lane bridges (two design vehicles – 400mm wheel width):

$$D_{wi} = 1.30 + \frac{L}{30}$$ *equation: 7*

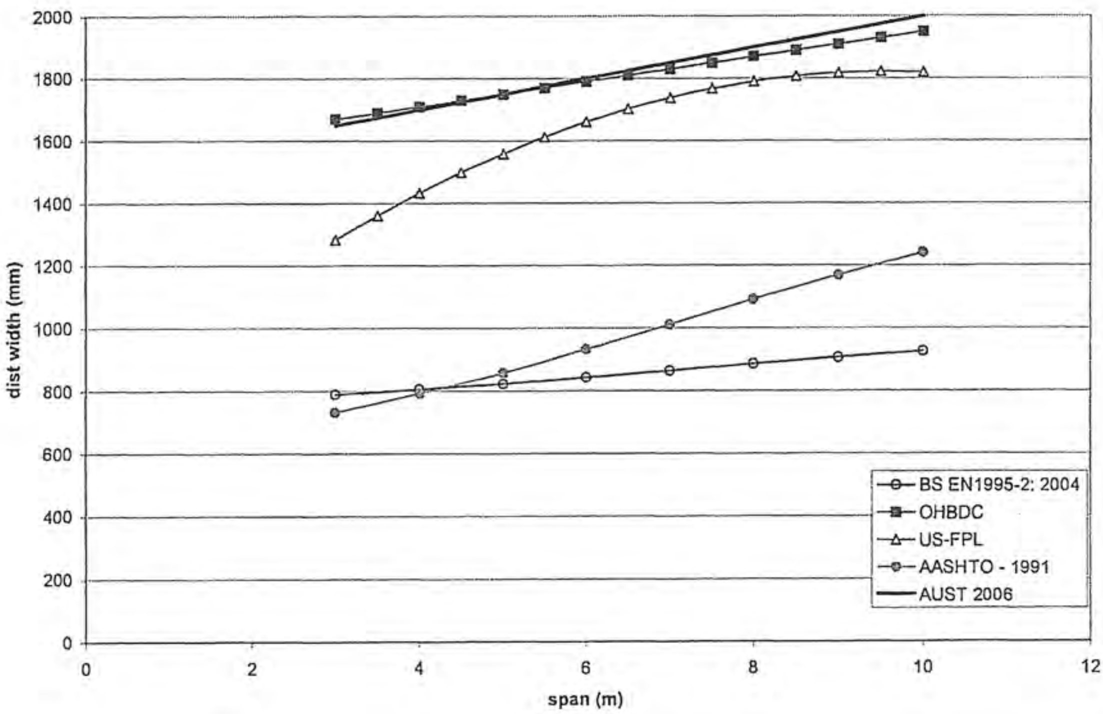

Figure 8 – Comparison of distribution width predictions – single lane bridges

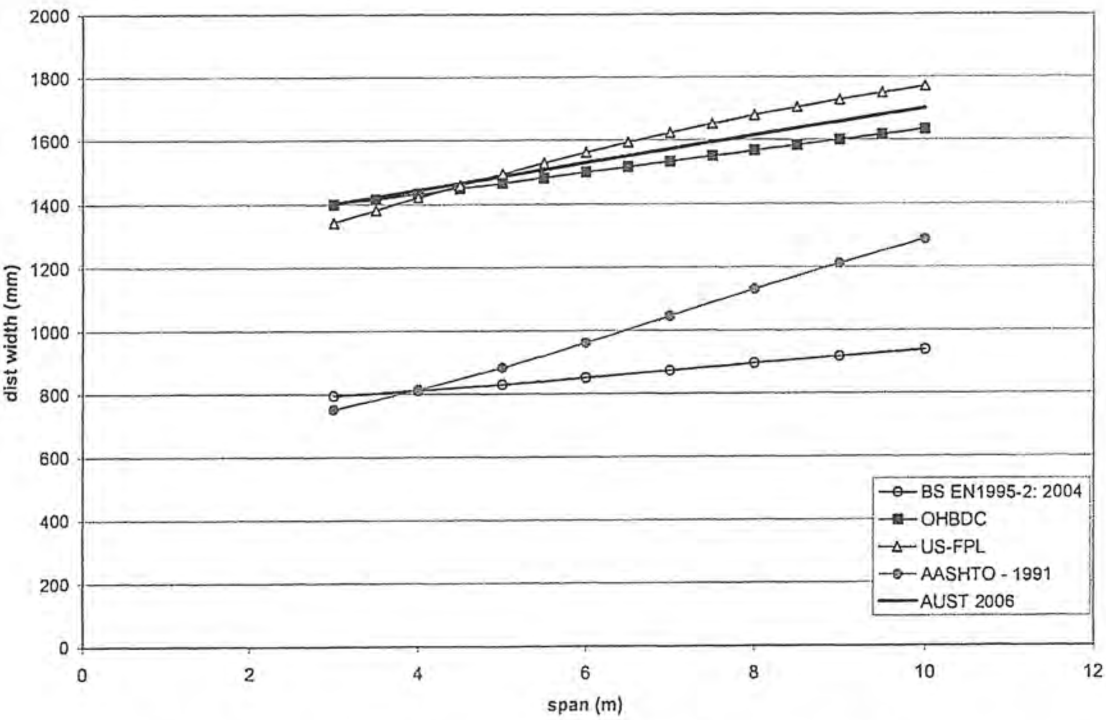

Figure 9 – Comparison of distribution width predictions – two lane bridges

The AASHTO [*1991*] method was first published in the interim design guidelines for use in the USA, but was later superseded by the "FPL" method, which was developed by Ritter [*1990, 1995*] following a widespread load testing program of prototype bridges. The OHBDC method was also developed from load testing and is now incorporated into the Canadian Bridge Design Code.

Distribution width predictions using equations 4 and 5 have also been compared with load test results from field monitoring programs undertaken in both Australia and North America and close correlations have been observed. The equations consistently predict maximum deflection for design load effects to within ± 5%; for example, maximum deflection of the bridge deck indicated in Figure 5, are 2% higher than those predicted using equation 5.

5 CONCLUSIONS AND RECOMMENDATIONS

Accurate prediction of the distribution width for design of an SLT plate deck has a major bearing on the economics of the structure, as well as being important for providing realistic assumptions for modelling the structural behaviour of the deck system. From Figures 8 and 9 it can be concluded that the current simplified method for determining distribution width specified in BS EN1995-2: 2004, is conservative when compared with other international methods, which have been derived from load testing of prototype stress laminated timber bridge decks.

Both the Canadian and Australian methods for predicting the distribution width are based on simple equations that have been found to have acceptable levels of accuracy and reliability when compared with the results of full scale load testing. It is therefore recommended that these same equations be assessed for applicability for modelling the load responses of European stress laminated timber plate bridge decks. This assessment should ideally be undertaken by analysis (using the equations presented in Section 4 of this paper) of data obtained from load testing of suitable bridges constructed from European timber species. The applicability of the equations would then be determined and if necessary adjustments made to produce a simplified set of equations that replace the current provisions of Clause 5.1.3 of BS EN1995-2: 2004 for longitudinal SLT decks.

6 REFERENCES

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