Development of Limit States Design Procedures for Timber Bridges

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Abstract

Recent work in North America, Europe, and Australia has resulted in the development of Limit States or Load and Resistance Factor Design (LRFD) procedures for timber bridges. This new approach makes significant departures from the current allowable (working) stress design procedures, which have often been based on the design of domestic building structures. This paper discusses the development of limit states design procedures for timber bridges in Australia and the United States, with a particular focus on the design of stress-laminated timber bridge decks and girder bridges. Issues such as characterisation of material properties, reliability, strength, and serviceability are also presented.

Keywords: Timber, Bridges, Design, Limit States, Load and resistance factor design, procedures.

Introduction

With any structural material, successful design is a function of the adherence to realistic and safe design criteria. In most situations, these criteria are set in design standards. Historically, design standards have focused solely on strength considerations based on allowable stress design criteria which limit the stresses induced by loads to values less than or equal to permissible stresses defined in the standards.

The current edition of the Australian Timber Structures Design Code (AS 1720.1-1988) is written in an allowable stress format and is one of the few structural design codes used by designers in Australia which is not yet in a limit states format (Standards Australia 1988). A soft conversion to limit states format was published in July 1994 as DR 94276, a draft document for public comment. This document has undergone several significant revisions and it is anticipated that the final version will be published in late 1996. A similar development process has occurred for timber bridge design in the United States where a load and resistance factor design (LRFD) code, *LRFD Bridge Design Specifications,* was recently published by the American Association of State Highway and Transportation Officials (AASHTO).

Timber Design Codes

Historical Developments

Most timber codes that were used throughout the world until the 1980s were in an allowable stress format. The permissible stresses for these codes were based on extensive research into the properties of clear wood samples tested in the first half of this century. Clear wood samples are small pieces of wood that are free from any sign of obvious strength-reducing characteristics. The absence of strength-reducing characteristics means that the properties are essentially those of the wood fibres aligned in the most favourable orientation. In general, the clear wood samples provide an indication of the upper limit of the expected performance of realistic timber pieces.

Derivation of Strength Properties

Design standards correct the clear wood sample data by reducing the basic species strength data to allow for the expected weakening caused by such characteristics as slope of grain, knots, checks, and splits. Over the years, these adjustments varied to ensure that various grades of some species showed satisfactory performance in service. However for most species, no data was available to relate the performance of the small clear samples on which the design properties were based to the performance of in-grade timber. In some respects, this approach represents the opposite that is required for design purposes, because the critical design criteria is not how strong a piece of timber is, but rather how weak it could be.

Part of the move towards reliability-based design was the need to have a clear understanding of the actual properties of the timber used in service. This led to the implementation of in-grade testing programs which assign the properties of timber samples randomly obtained from stocks of commercially available timber. Because commercially available timber has characteristics that affect the strength of the timber, and these characteristics can be randomly positioned in service spans, the in-grade test requirements in Australia call for random positioning of the test pieces within the test span. In-grade testing in other countries biases the location of defects, which generally results in lower characteristic strengths for pieces of small end section timber (less than 150 mm (6 in.) deep), but results in similar strength results for larger sizes as a result of the larger test span required for deeper sections.

Data from in-grade testing show considerable variation, which represents the true variability in properties of commercially available timber. These data can be used to determine either allowable stresses or characteristic strengths of the tested grade of timber; however, the sample size required is very large if the data are to be statistically significant.

Reliability of Material Properties

In-grade testing gives a measure of both the properties and the variance of the properties of commercially available timber. This more accurately represents actual component properties compared to the small, clear wood sample data. The current Australian Timber Structures code (AS 1720.1) recognises this in engineering applications through the use of a strength modification factor known as the k_2 factor (Amendment 1 - AS 1720.1)(Standards Australia 1993). This factor reduces strength data for timber to be used in places with severe consequences of failure, but strength properties derived from in-grade data receive a lower penalty than the properties derived from small, clear sample data. This effect is also reflected in the modification factors specified in the forthcoming limit states version of AS 1720.1-1996.

One of the driving forces for the incorporation of such a "reliability" strength modification factor in an allowable stress code was the fact that in Australia, there are no design procedures available for timber bridges other than the general structural provisions of AS 1720.1. The one exception to this is for the design of stress-laminated timber bridge decks, which is discussed later in this paper. The lack of suitable design code provisions for timber bridges has presented bridge designers with a number of problems, not the least of which is a general ignorance and lack of understanding of timber engineering among the design professions.

Limit States Design Procedures

Why "Limit States"?

Limit states design reflects a global trend. Plans are in place for the use of limit states design codes for all engineering materials in Europe, North America, Australia, New Zealand, and more recently in Asia. This global trend has a sound rational basis, as variations in capacity reduction (ø) factors can be used to account more accurately for variability in properties or capacity models. This results in a more uniform and quantifiable level of reliability than can be achieved with allowable stress design.

Limit states codes separate loads expected for the serviceability limit state from those expected for the strength limit state. This is the reality of design. The parts of any timber design code that pertain to deflection will change very little in a limit states code, and so will produce similar results to those of the current allowable stress code at similar loads to the current working loads. This is significant, as many structural elements are controlled by the serviceability limit state.

A number of limit states design codes for timber structures are currently in use throughout the world. Of special note is the Canadian code which has made substantial use of Canadian in-grade data to produce a reliability-based limit states code (Canadian Standards Association 1984). The draft European code, *EuroCode 5* (1992), makes use of a wealth of data on the behaviour of European softwood timber and connections but differs markedly from the Canadian standard in format. This code has no official status as yet, but will serve as a model for national European codes to be developed in the future. New Zealand's timber design code (Standards New Zealand 1993) has also used in-grade data for their principal structural species, radiate pine, to produce a soft conversion of a permissible stress format code to a limit states code. The USA has developed and implemented a limit states timber structures code which is also quite different to other existing limit states codes and drafts (Gramala et al. 1994; ASCE 1995).

Limit States Philosophy

Traditionally, allowable stress design codes have focused on providing adequate strength and achieving a level of safety acceptable to the general community. However, limit states philosophy recognises that the community in general, and clients in particular, are also interested in other design considerations such as serviceability, safety, and stability. If performance by any of these criteria is unsatisfactory, the "limits" in one or more of the limit states are exceeded. There are others as well. The fire limit state is a special case, where the loads that must be carried by a structure that has been partially damaged by fire are of primary consideration. Another is the fatigue limit state in which a structure or component that is loaded and unloaded repeatedly throughout its life must perform satisfactorily with acceptable fatigue damage.

Each of the performance limit states is different, representing a different loading scenario in the life of a structure. Violation of the limits produces different consequences for each of the performance limits mentioned. Design for a limit state aims to produce satisfactory performance to the limit prescribed in each of the performance states (Boughton and Crews 1996). In the design of timber structures, it is primarily the serviceability and strength limit states that will be the first consideration, with checking required to ensure adequate performance in other limit states.

Serviceability Limit States

Under prescribed loads, there may be limits on deflection (all materials), cracking (concrete, composite structures), and vibration (all materials). If performance is beyond these limits, the client may find the performance of the structure unacceptable. In many cases, these limits are not prescribed in codes, but are left to the discretion of the designer and client. Where the serviceability limits are violated, there may be inconvenience in the operation of the structure or some damage to non-structural elements, such as the asphalt wearing surface on timber bridges. In general, the damage can be repaired as part of routine maintenance, but rectifying the fault that caused the damage is often very difficult as it involves modification of the basic structure.

Violations of the serviceability limit states are not catastrophes but produce annoyances requiring extra work and expense for the client. If the violations occur less frequently than typical routine maintenance, they are generally considered as a separate maintenance issue. However, if the violations occur more frequently than typical routine maintenance, the required repair work becomes an additional burden and the client will have justifiable cause for complaint.

The serviceability load combinations specified in most limit states design codes are reasonable estimates of combinations of loads likely to cause maintenance problems or affect the performance of the structure. Under most codes, the probability of exceeding the serviceability load combinations is approximately 5% per year.

Strength Limit States

Violations of the strength limit state imply structural failure. Thus, it is important that there be a very low probability that the strength limit state will be exceeded over the life of the structure. Failure may be due to overloading and/or under-performance. Overloading is addressed by the use of appropriate load factors and under-performance is addressed by both a capacity reduction factor and the use of conservative strength capacities derived from the lower 5th percentile of test data for timber.

Figure 1 shows a classic probability distribution model for loads. Load effects can be approximated by normal distributions, log-normal distributions, or Gumbel distributions. In viewing the probability distribution, the probability of exceeding a given load (z) is given by the area to the right of the load (shaded in Figure 1) as a fractile of the entire area under the distribution curve. The AUSTROADS bridge design code defines the limit states probabilities as shown in Table 1.

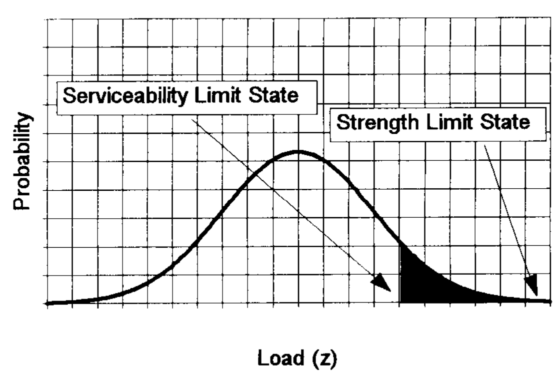

Figure 1— Probability distribution model for typical loads on a structure.

Table 1—Definition of Limit States probabilities (AUSTROADS).

Limit State	Return interval of load	Probability of exceedance in any 1 year
Servicability	20 years	0.05
Ultimate (strength)	2,000 years	0.0005

In the same way that there is a probability distribution for loads, there is also a probability distribution model for strength resulting from material variation, failure mechanics, and variation in workmanship. The probability distribution model for strength is illustrated in Figure 2 and is similar to the curve for loading. The principle concern in this diagram is the chance of not attaining a given strength. The area under the curve to the left of a value expressed as a fractile of the total area under the curve gives the probability of not attaining that strength.

By overlaying the load distribution curve and the strength distribution curve, it can be seen that the strength curve lies to the right of the load curve (Figure 3). As the two curves come closer together, the probability of failure increases. Therefore, the distributions must be separated as necessary to reflect an acceptable low probability of failure. Factors in design codes are chosen to give a probability of failure that is calibrated to acceptable design performance and practice. In designing or checking for the strength limit state, designers ensure that the probability that the factored capacity is less than the factored loads is very small.

Capacity Design Equations

The design equation for the strength limit state is:

$$R_d \geq S^* \quad \textbf{(1)}$$

where R_d is the design ultimate capacity of the member and S* is the design action effect due to the factored design loads imposed on the particular element. The design ultimate capacity is derived from the nominal capacity of the section multiplied by the material capacity reduction factor as shown in the following:

$$R_d = \phi R_u \quad (2)$$

In this case, R_u is the nominal capacity and ø is the material capacity reduction factor, defined by the code, to give an appropriate level of safety for the type of structural element under consideration. The nominal capacity is stated in terms of a tabulated characteristic value, multiplied by the relevant strength modification factors, which are designated as "k" factors in the Australian code. This is defined in the following:

$$R_u = k_{\mathbf{mod}} R_k \quad (3)$$

where K_{mod} is the total effect of the strength modification factors that are relevant to the design action (e.g., duration of load, load sharing, size, stability), and R_k is the basic characteristic strength of the structural member. Details of the AASHTO LRFD equations have a similar format and are discussed in detail elsewhere (Ritter and Nowak 1994).

The principal differences between an allowable stress design code and a limit states code are in the design philosophy, the loads, and the basic design stresses. By comparison with Equations (1) to (3), allowable stress codes generally present the same design equations in the

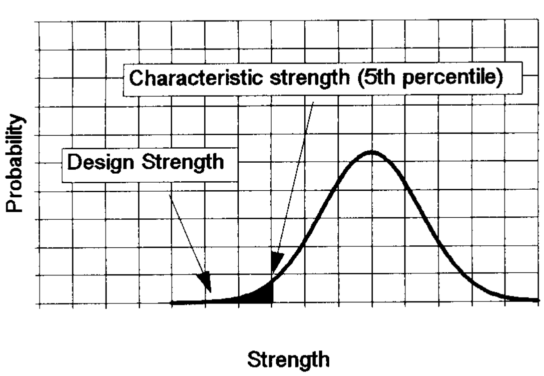

Figure 2—Probability distribution model for strength.

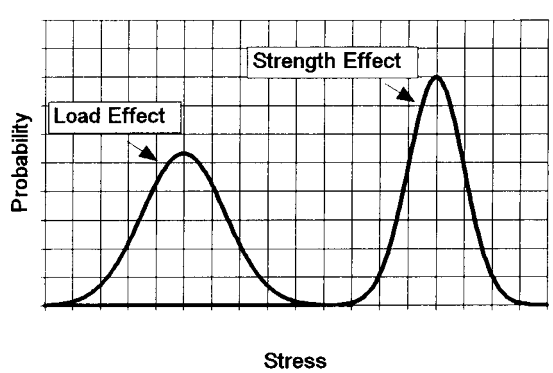

Figure 3—Relative positions of load and strength probability distributions.

form given in the following:

$$f'_k \leq k_{\mathbf{mod}} \, F'_k \qquad (4)$$

where f'_k is the actual stress F'_k is the basic allowable stress for the design action under consideration. In limit states design, the design effects for live load, dead load, and combinations of loads must be compared with the factored design capacities obtained from the previous equations. In all cases the design capacity must be greater than or equal to the load effect if the strength limit states are to be satisfied. As the design loads are factored to give those for the very rarely occurring limit state "event", the ultimate loads are substantially larger than the actual working loads. The strength side of the equation must also correspond to the capacity of the member under the type of loading anticipated in the strength limit state event. The characteristic stress given in the limit states code must therefore be a stress which could reasonably be expected to be achieved just prior to the failure of the member. These characteristic stresses are substantially greater than the basic allowable stresses, recognizing that in limit states format, the factor of safety is principally applied to the loads, rather than the stresses.

Interim Methods for Improving Reliability

Most of the timber structures which formed the experience base for development of AS 1720.1 and similar allowable stress codes were essentially "domestic structures" which contain many parallel elements. If failure of one member occurred, it would normally lead to load redistribution to adjacent elements. The magnitude and nature of loads in these essentially low rise residential structures are also such that in many cases, a "failure" may go unnoticed. However, many modern timber structures, such as glulam or structural composite lumber (SCL) portal frames and bridges, have little redundancy so that redistribution of load is difficult or not possible. Failure in these circumstances may prove catastrophic. It was only as in-grade data started to become available that the code committees could properly evaluate the level of reliability of timber structures.

It was clear that where parallel systems were available and where the cost and risk of failure were low (e.g., in domestic structures), the level of reliability of the entire structure was adequate. However, where such load sharing was not available or where the loads were of greater magnitude and much more concentrated, the level of reliability in timber structures needed to be increased, particularly for timber bridges. The "material and application" factor, k_2, was introduced in an amendment to AS 1720.1 in 1993. The factor was intended to correct the overall reliability of timber structures, which were still being designed using a allowable stress code, to bring it into line with steel and concrete structures which were already being designed using reliability-based, limit states design codes. The k_2 factor as it currently stands affects only those structural elements where load redistribution is not possible and/or where failure of the element may prove catastrophic; i.e., it is concerned with the effect of failure of an element on the "system" behaviour. Because an in-grade evaluation of structural properties gives a higher level of reliability in the properties than those inferred from the testing of small clear samples, the k_2 factor for material with in-grade properties is greater than that with properties found by other methods.

Application of Limit States to Timber Bridges

Historical Developments

In Australia, design of all bridges is defined by the AUSTROADS Bridge Design Code, which was published in a Limit States format in 1992. This code has a reliability basis for determining loads for strength and serviceability limit states and the design of all aspects of bridges constructed from concrete and/or steel. Although the AUSTROADS code is specific to road and pedestrian bridges, the format and technical provisions are similar to the Standards Australia Limit States structural design codes, which have been in use since the late 1980s. However, there are no AUSTROADS provisions for timber bridges and engineers assessing or designing such structures have been forced to use the existing allowable stress version of AS 1720.1 - 1988. For most engineers, this is difficult since they are unfamiliar with allowable stress timber design criteria. Additionally, timber design is outside of the experience of most practicing engineers.

For bridge designers in the United States, many of these problems have been essentially addressed by the AASHTO LRFD provisions for design of timber bridges. The development of similar procedures in Australia is still underway, although most issues will be addressed with the publication of the Limit States version of AS 1720.1 at the end of 1996.

The first Australian version of a limit states design code for timber structures was published in April 1995 (Crews 1995). Although this code is specific to stress-laminated timber (SLT) bridge decks, it contains principles which are generally applicable to the design of engineered timber structures. This design specification references the loading provisions of Section 2 of the AUSTROADS code.

Defining Appropriate Reliability Levels

One of the problems facing development of the limit states format for timber bridges is the fact that most material properties used throughout the world have been derived on the basis of building loads rather than the greater or more concentrated loads associated with bridge structures. As noted previously, this necessitates higher levels of safety and reliability. This has been partially addressed in Australia for both the existing allowable stress design timber structures code and the limit states design code for SLT bridges, by use of the k_2 strength modification factor.

Bridge structures also require a high factor of safety which must be addressed not only for loads but also in the characteristic material strength properties. The effect of this in a limit states format is that the material properties or characteristic strengths specified by such codes may in fact have a lower than desirable β safety index. For example, the derivation of fifth percentile modulus of rupture (MOR) values, which form the basis of the characteristic strength properties of timber, has been based on a 75’% tolerance level which is acceptable for normal building applications. However, in order to achieve the higher degree of reliability which is necessitated by a bridge structure, the tolerance level for prediction of fifth percentile values should possibly be as high as a 95% confidence level. This would effectively lower the fifth percentile estimate of MOR or the characteristic strengths in order to obtain suitable design properties for bridges. Similarly, a first percentile rather than a fifth percentile value could be used, using the existing statistical methods for characterising material properties, with much the same effect.

Modification Factors

Possibly the simplest and most desirable approach would be to adjust the current industry values by a “bridge” or “consequence of failure” adjustment factor applied by the designer. This is the method which has been adopted in most design codes through selection of appropriate values for the capacity reduction factor. This is an alternative to independently developing a specific set of characteristic strength properties for heavily loaded, engineered timber structures such as bridges.

In the current allowable stress edition of AS1720.1, it is not possible to incorporate a material capacity reduction factor (ø), so an increase to the β safety index has been effected for engineered timber structures by including the k_2 material and application factor. For the purpose of the first edition of the limit states design procedure for SLT bridges, the k_2 factor has been used as the modifier for the material properties rather than changing the basic characteristic strength properties or ø.

The current values of k_2 in AS 1720.1 are 0.9 and 0.7 for characteristic properties derived by in-grade and small clear samples, respectively. Initial research by Crews indicated that these values should be reduced to 0.85 and 0.65 respectively, for bridges (Crews 1994). Although this approach appears to increase the safety index to an acceptable level for bridge structures, further research needs to be done in this area. Similar work is being undertaken in the United States and has been previously presented (Ritter and Williamson 1991).

For the design of SLT bridge decks, k_2 will equal 0.85 if the material properties have been obtained using an in-grade testing program, or will equal 0.65 if the material properties have been derived by any other method (e.g., from testing of small clear samples).

Development of a “True” Reliability

The intention of the k_2 factor in the Australian timber codes has been to ensure that the level of reliability of timber in higher risk/consequence of failure “engineered” structures is consistent with that for similar structures designed using other engineering materials, such as steel or reinforced concrete. Although the k_2 factor is a responsible and convenient way to increase reliability for an allowable stress design code, it is not really appropriate for use in a limit states code where the reliability of material properties or the capacity of the structural element is generally modified by a capacity reduction (ø) factor.

Recent work by Crews (1966) proposed that k_2 be removed from the limit states code (draft) and that it be replaced by a table of appropriate derived ø factors. This will simplify use of the code by eliminating a “k” factor and “doubling up” on reliability modifiers (i.e., currently a designer must use both k_2 and a ø factor). It will also permit the use of ø factors which have a logical connection with a specific safety index and level of reliability for the timber product being designed. The research underlying this proposal, which has been accepted for inclusion in AS 1720.1-1996, focuses on a detailed probabilistic analysis of load effects and material variability in the determination of safety indices and levels of reliability for timber structures, which are appropriate to their end use and the consequence of failure (Leicester 1985; Foschi et al. 1989). The outcome of this research program is a table of ø factors, which are reproduced in Table 2.

These capacity reduction factors effectively decrease the probability of failure, due to a combination of excessive overload with grossly substandard timber strength, to levels appropriate for the type of structure being designed.

Table 2—Proposed capacity reduction (ø) factors for AS 1720.1.

Properties group		Consequence of failure Category 3	Consequence of failure Category 2	Consequence of failure Category 1
Group	**Typical examples**	**Minor significance**	**Normal significance**	**High significance**
A	SCL & plywood	0.9	0.85	0.80
B	MGP pine & glulam (ingrade with quality control and ongoing verification)	0.9	0.8	0.75
C	F graded timber with some verification	0.85	0.7	0.65
D	F graded timber	0.8	0.65	0.55

Note: Consequence of failure Category 3 corresponds to $\beta \approx 3.0$, Category 2 corresponds to $\beta \approx 3.5\text{-}4.0$, Category 1 corresponds to $\beta \approx 4.0 - 4.5$

For bridges, secondary components would be classed under Category 2, while all primary members and critical elements would be classed under Category 1. Webs and laminates in SLT decks would be classified under Category 1, noting that the Australian limit states design procedures for plate decks already permit significant increases (up to 50% for sawn timber) in the basic characteristic strength as a result of load sharing distribution effects.

The original calibrations were done for $\beta \cong 3.0$, 3.5, and 4.0, respectively, but due to rounding of the ø factors, a range of β values occurs in the table. The ø factors for connection systems are still being addressed, but it is considered that a value of β=5 to 6 is appropriate for connection systems in bridges. Depending on the timber material in the connection, this will produce values of ø, ranging from 0.75 to 0.4. Combining this approach with the existing limit states provisions for ultimate limit state (ULS) design loads means that the probability of failure during a 100 year design life is about 10^{-4} for Category 2 members and about 10^{-5} for Category 1 members.

The AASHTO LFRD specification is similar and combines a factor which is relevant to the type of load effect, with a table of "base resistance" characteristic strengths for species and grades of timber commonly used in bridge design. For species and grades not included in the LRFD tables, a series of conversion factors is provided to determine values for the base resistance appropriate for bridge design. The nominal resistance is then determined using modification factors in a form similar to that shown in Equation (3).

Defining Failure at the Strength Limit State

An important consideration that also needs to be addressed in determining appropriate levels of reliability for bridge design is the definition of what constitutes failure. For example, in a single lane girder bridge, the failure of one girder as a result of excessive load combined with weak material is likely to be catastrophic for the entire bridge. However, a single girder failure in a wider bridge with 7 or 8 girders may not be quite as disastrous, depending on the location of the girder in the bridge.

For SLT decks constructed as either plates, box, or T-beam sections, the deck system has exceptionally high "post critical" load capacity, even after failure at the strength limit state has occurred. It should be noted that full-scale laboratory testing of SLT deck systems has confirmed that the ultimate load capacity of such decks is generally considerably in excess of the ULS requirements specified in the AASHTO LFRD and AUSTROADS limit states loading codes. Not only are such deck systems remarkably ductile, but they inherently redistribute loads around any localised area of member failure (due to rupture), and behave in a linear elastic manner in the serviceability load range even after failure has occurred in the critical web members of cellular/box decks. Unless prestress has decreased to less than 50% of the initial level, these deck systems do not “fail” catastrophically in a manner which would be observed in steel or concrete deck systems. T-beam SLT bridges have enhanced load sharing capacity when compared with girder bridges, but still tend to be limited by the capacity of the webs, particularly on the tension side of the beams. Details of ultimate and post critical behaviour of SLT decks systems are presented elsewhere (Crews and Bakoss 1993; Crews and Bakoss 1996).

Defining Load Events

The preceding discussion has focused mainly on the problems associated with defining reliability with respect to the material capacity or resistance side of Equation (l). Considerable research work has also been undertaken with respect to the loading allowances for limit states design codes. Both the AASHTO and AUSTROADS codes specify numerous load events based on the effects of vehicular and environmental loads (predominately due to earthquake, wind, and water) on the bridge structure.

Generally, three vehicular load events are defined for consideration in a limit states code. These are based on “standard” design vehicles, with specified axle loads and geometry. The loads are factored and load effect envelopes for flexure, shear, and reactions are determined. The three vehicular load events normally considered are (1) strength (2) serviceability for a “normal” design truck, and (3) strength for an “abnormal” heavy load design vehicle. For example, AUSTROADS specifies that the strength limit state be determined for the load effects of

- dead load with a ULS load factor of 1.2,
- a T44 vehicle with a ULS load factor of 2.0, and
- a Heavy Load Platform (HLP) vehicle with a ULS load factor of 1.5.

The serviceability limit state (SLS) is determined for load factors of 1.0 for all load events. In all cases for vehicle loads, the dynamic load allowance (DLA) must be applied as detailed in the next section.

Dynamic Load Allowance

In the past, the effects of dynamic loading have usually been ignored for timber bridges. The AASHTO LRFD specification requires that the load effects from the static design vehicle loads be increased by 75% for deck joints and 33% for all other components, while the AUSTROADS code requires a 20 to 40% increase for the T44 vehicle loads (depending on the first natural flexural frequency of the superstructure) and 10% for heavy load platform (HLP) loads. It is generally recognised that timber bridges have excellent inherent damping under dynamic load and because of this fact, both the AASHTO and Australian SLT provisions permit a reduction in the dynamic load allowance, of 50% and 30%, respectively.

Serviceability Limit States

Serviceability design for timber bridges is mainly concerned with deflection limits. Neither the previous U.S. nor Australian allowable stress design codes contain provisions for checking deflection criteria. However, this has now been addressed in the AASHTO LFRD specification and the Australian SLT Code. Deflection limits are generally based on the unfactored vehicle live load, including the DLA. Timber bridges tend to be much more tolerant of deflection than those of concrete or steel and this is reflected in the deflection limits specified in codes. The AASHTO LFRD specification presents a deflection limit of bridge span divided by 425, which is left to the discretion of the designer to apply. The Australian SLT provisions specify span divided by 380. This latter limit is applicable only to SLT decks with a flexible bituminous seal wearing course, and a limit of span divided by 450 is more appropriate for other bridge types. These limits are based mainly on the need to avoid rotation at the supports, and more stringent limits may be appropriate if a rigid asphaltic concrete wearing course is used on the deck to avoid cracking over the supports.

Concluding Remarks

The AASHTO LRFD specifications provide structural engineers with a suitable code for limit states design of timber bridges. The reliability approach incorporated in the forthcoming limit states timber structures code (AS 1720.1-1996) in Australia will provide the technical basis for development of a specific timber bridge design code. Preliminary work on an Australian code for timber bridges has commenced for Standards Australia, which has now taken over the codification role of AUSTROADS. The current SLT code is being revised in the light of work currently being done by the authors, and it is envisaged that this updated design specification will be incorporated into this general timber bridge code. The end result is that engineers in both the United States and Australia can have the same level of confidence in the design and reliability of timber bridges as they currently enjoy for similar structures designed in concrete and steel.

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