Structural Reliability of Plank Decks

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Abstract

The structural reliability of plank decks designed by the AASHTO codes are determined. Inadequacies in load distribution and plank resistance in the current Specifications are identified.

Introduction

The objective of this study is to evaluate the adequacy of the current AASHTO Specifications' design criteria for plank decks for highway bridges. This evaluation will be based on the reliability index, β, the probabilistic measure of the structural performance. It has been observed that the current AASHTO code (1996) provisions for load distribution for plank decks are not realistic. The problem was identified by the AASHTO Committee on Timber Bridges as a priority item requiring an urgent solution.

Plank decks are classified into one of two categories, depending on the direction of planks relative to the direction of traffic; transverse decks and longitudinal decks.

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For transverse plank decks, the span of stringers is usually 5-6m, while in older structures it can be up to 11m. Stringers are spaced between 300-450mm center-to-center, and not more than 600mm. They are made of sawn lumber. Typical Southern Pine size is 150x450mm, and Douglas-Fir can have larger sizes. Planks are typically 100x250mm or 100x300mm, with a length of 3.5-11m, and are nailed to stringers. Longitudinal plank decks are similar. For longitudinal decks, the major design parameter determined by the designer is the spacing between stringers.

In this paper, it is assumed that stringers have an adequate load carrying capacity and that they provide a sufficient support for planks. The design of stringers is not considered.

The study is focused on distribution of the truck load to plank decks and plank resistance. Material properties, in particular modulus of rupture (MOR), are based on actual test data. The load model is based on available weigh-in-motion measurements data, and the contact pressure between truck tire and road surface is modeled using the available literature.

Load and Resistance Models

The live load model is developed on the basis of the actual truck measurements. Extensive weigh-in-motion (WIM) measurements were carried out by researchers at the University of Michigan (Nowak et al. 1994). The study provided statistical data on gross vehicle weights (GVW), axle weights and axle spacing. The mean maximum weight for a two-tire wheel is 200 kN, or 100 kN for a single tire, and the coefficient of variation is 0.25. Actual tire contact area is based on work by Pezo et al. (1989), and is taken as 200x250mm for a single tire, where 250mm is in the direction of traffic. For a two-tire wheel unit, the area is taken as 500x250mm, with a 100mm space between tires.

The wheel load is applied as a uniform pressure on the planks. For transverse planks, if the plank width is larger than the length of contact area (250mm), then it is assumed that the load is distributed over the whole plank width. If the plank width is less than 250mm, then the plank takes only a portion of the wheel load proportional to the ratio of plank width and 250mm. Longitudinal planks are treated similarly. If the plank width is larger than the contract area width (200mm), then the load is distributed over the whole plank width. If the plank width is less than 200mm, then the load is reduced proportionally. Planks are modeled as continuous beams on elastic supports.

The major parameter which determines the structural performance of wood components is the modulus of rupture (MOR). The statistical model of MOR is based on actual in-grade tests carried out by researchers in Canada (Madsen and Nielsen 1978) and the test data was processed by Nowak (1983). The flatwise use factor, the ratio of MOR for edge-wise and flat-wise loading, is based on work by Stankiewicz and Nowak (1997).

The reliability analysis is carried our using the procedure developed for calibration of the AASHTO LRFD (Nowak 1995). Load and resistance are treated as lognormal random variables. Reliability is measured in terms of the reliability index, β. The analysis is performed for plank decks designed using AASHTO (1996) and AASHTO LRFD (1994). The reliability index is calculated using the following equation:

$$\beta = [\ln (m_R) - 0.5 \ln (V_R^2 + 1) - \ln (m_Q) + 0.5 \ln (V_Q^2 + 1)] / [\ln (V_R^2 + 1) + \ln (V_Q^2 + 1)]^{1/2} \qquad (1)$$

Where m_R = mean resistance, m_Q = mean load, V_R = coefficient of variation of resistance, V_Q = coefficient of variation of load.

The corresponding β's are representative of current design practice. Calculation results are presented in Table 1 for AASHTO (1996) and AASHTO LRFD (1998).

Table 1. Reliability Indices for Douglas-Fir Planks, AASHTO LRFD (1998).

	Transverse Planks				Longitudinal Planks			
	Select		Grade 1&2		Select		Grade 1&2	
Size	1996	1998	1996	1998	1996	1998	1996	1998
100 x 150	5.3	5.3	6.1	6.1	*	6.4	*	*
100 x 200	4.6	4.6	5.3	5.3	*	5.6	*	6.6
100 x 250	3.6	3.6	4.9	4.9	7.6	5.2	*	6.5
100 x 300	4.2	4.2	6.1	5.4	7.9	5.9	*	7.0

The results indicate that there are considerable differences in the reliability indices. The lowest values of β are obtained for 100x150 and 100x200. On the other hand, from the system reliability point of view, 100x150 planks are better because they allow for a better load sharing between the planks.

Conclusions

Given that the LRFD code target reliability index is 3.5, it is clear that in most cases the codes are overly conservative. This is primarily the result of two factors: an unrealistic load distribution model, which assumes the entire wheel load is carried by a single plank, regardless of plank width, and flat-use factors which do not adequately predict plank capacity (current values in the code were significantly lower than those found in testing). Simple changes in the Specifications, such as adopting a more reasonable load distribution and analysis model, and more accurate flat-use factors, may dramatically improve results and establish code reliability index consistency.

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April 18-21, 1999
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