Stress-Laminated Timber Bridge Decks: Non-linear Effects in Ultimate and Serviceability Limit States

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Summary 10 lines max

Stress-laminated timber bridge decks made of glulam beams are advantageous when it comes to their strength, production and construction costs. The thickness of a deck with a specified span and width is typically determined by considering requirements relating to strength and stiffness stipulated in a design code and specifically in Europe the Eurocode 5. Linear stress calculations do not have inherent potential to simulate slip or gaps between beams, but experience from finite element (FE) simulations and full-scale tests shows that these nonlinear effects from slip and gaps between glulam beams exist. In this paper, comparisons between linear and non-linear FE results for two bridge decks with different spans, widths and thicknesses are made. Separate comparisons are made in the ultimate limit state (ULS) and the serviceability limit state (SLS). It is shown that non-linear effects may be important and should be considered, especially in the case of thin decks.

Keywords: prestressed, stress laminated, timber bridge deck, finite element, FEM

1. Introduction

Stress laminated timber bridge decks in Sweden are now days designed according to the rules in Eurocode 5 [1] in the serviceability limit state (SLS) and the ultimate limit state (ULS). Limits for strength of a bridge deck are determined by specified design values for load and wood material strength in ULS. Limits for deformations in SLS may vary depending on the specific application in question. Design values for loads and material properties to be applied are different for SLS and ULS and normally taken as mean values for SLS and 5-percent percentile values for ULS, i.e. in general higher design loads for ULS.

Eurocode does not explicitly specify calculation methods which mean that linear manual calculation methods or linear FE (finite element) methods may be used. Traditionally early developed US effective width beam methods with correction factors for plate effects have been used, see [2].

Recent full-scale tests and FE-simulations with nonlinear models of frictional slip included have shown that slip between beams in stress laminated bridge decks start to appear at relatively low load values, [3], [4], [5]. At low loads frictional slip is mainly horizontal due to twisting moments and appears in the corners of the bridge deck but later at higher load levels also vertical slip appear, normally close to the load application points. The slip does not lead to failure or instability but is visible as a deviation from linearity of the load vs. displacement curve.

A number of calculations with different calculation methods have been conducted recently in a project for 5 different bridge deck geometries and for 3 different load cases. The methods are a linear FE shell method, a nonlinear 3D (three-dimensional) FE contact method and a nonlinear 3D FE elastic-plastic method. In this paper results for the elastic-plastic FE method is shown for two of these bridge decks. The purpose of this paper is to show differences between linear and nonlinear FE results and to show for which bridge deck geometries and for which of ULS and SLS these differences are large and thus needs to be taken into consideration. Also shown are the consequences of nonlinearity such as the size of frictional slip.

2. Materials

Two different single span bridge deck geometries were chosen for this paper, one short and one long. Dimensions were 5x4 m with thickness 315 mm and 25x12 m with thickness 1125 mm, see Table 1. The deck thicknesses were beforehand chosen fulfil approximately acceptable load-bearing capacity in ULS and this was then checked afterwards.

Table 1. Bridge deck geometries.

Span *l* (m)	Width *b* (m)	Thickness *t* (mm)	*b/l*	*t/l*	*t/b*	
5	4	315	0.80	0.16	0.079	short
25	12	1125	0.48	0.045	0.094	long

The decks were built up from Norway spruce (*Picea abies*) glulam beams having a width of 90 mm and being produced from lengthwise finger jointed boards with thicknesses 45 mm. Prestress value was set to 0.35 MPa which was the lowest acceptable prestress value.

3. Methods

Finite element (FE) modelling of the deck geometries were made with the commercial FE code ABAQUS, [6] and a specially written user subroutine for treating orthotropic plasticity [7]. The method to handle frictional slip was described in [3]. The deck is treated as a continuum with no borders between beams and frictional slip is modeled applying an elastic plastic material effect. Frictional slip appear in horizontal and vertical directions with friction coefficients 0.29 and 0.34, respectively. Gaps that may appear due to transverse bending were modeled as local reductions of all elastic constants to near zero when local transverse stresses exceeded zero. Density of the glulam beams was 400 kg/m^3 and moduli of elasticity 12000 and 240 MPa in the fiber and cross directions, respectively. Shear moduli were 720 MPa for bending shear in two shear planes and 72 MPa for rolling shear modulus. Poissons ratios were all set to zero, all of this according to Eurocode standard. Design values for strength were taken as 22.18, 1.94, 2.52, 0.58 MPa for bending stress, transverse compressive stress, bending shear stress and rolling shear stress, respectively. Supports were modeled as very stiff and prestress 0.35 MPa was applied as a constant pressure on the side surfaces of the bridge decks. Half models were used due to symmetry conditions see Fig.1.

The only load case that was studied here was a selected combination of a number of loads. Gravity load of wood, pavement load 2.5 kN/m^2, a distributed load of 9 kN/m^2 on the whole top surface of the deck and finally 4 wheel loads each of 225 kN on 0.4x0.4m surfaces on the top surface of the deck, i.e. a double axel load. The wheel loads center was positioned in the middle section of the span but laterally close to the edge see, Fig.1. These levels of loads combined together defined the ULS design load in this paper and its relative load value was designated 1.0. For reasons of simplicity and to be able to compare between SLS and ULS and also in order to find out what happens for even greater loads, the combined loads were increased proportionally from zero to the relative value 2.3. This was made in order to find out where nonlinear effects and/or instabilities starts to occur. This is not the exact way load cases are to be treated in Eurocode 5 since design load combinations differ between ULS and SLS and also the relative proportion of each load varies between ULS and SLS. Also, this is only one of many load cases that have to be treated when performing a full examination according to Eurocode. However, as a rough estimate used here to compare linear and nonlinear results, relative load levels 0.436 and 1.0 were found to approximately represent the SLS load case and the ULS load case in Eurocode 5 for the 5x4m deck, respectively. In the same way relative load levels 0.222 and 1.0 were found to approximately represent the SLS load case and the ULS load case in Eurocode 5 for the 25x12 m deck, respectively. Limits for stresses in ULS according to Eurocode were checked and points where limits were reached were marked on the load vs. displacement curves. Vertical slip values were calculated as plastic vertical shear strain times the beam width 90 mm in order to get a specific value for vertical slip between beams. Horizontal slip values were generally much lower.

4. Results

4.1 Bridge deck 5x4 m

Figs.2a and 2b show linear and nonlinear load-displacement curves for bridge deck 5x4 m for loads up to 2.3 times the ULS-load. The maximum deformation in the nonlinear solution was about 22% and 12% higher than the linear solution at the ULS load and SLS load, respectively. The displacement was measured at a point on the very edge of the bridge deck in the midsection. This point was the point of maximum displacement for relative load levels below 1.19 but for higher load levels a point in the midsection between the wheels close to the side achieved larger displacement due to slip between beams, see Fig.3. No instabilities occurred for relative load levels up to 2.3 but maximum slip between two beams increased up to 20 mm. ULS limits for bending stresses in fiber direction were reached at relative load levels 0.97 and 0.84 for the linear and nonlinear case, respectively. A limit of span/400=12.5 mm was used as the SLS limit for deformation. Horizontal slip started early at about 0.1 relative load and over large areas close to the corners of the deck at the top and bottom of the deck. At load level 2.3 the horizontal slip was at the most 1.70 mm. Load vs. displacement curve started to deviate from linearity at load level 0.1. Vertical slip started at about load level 0.4 close to the wheel load application areas and in the middle of the height of the deck and it reached values shown in Fig.2. The maximum gap between two beams is shown in Fig.2a.

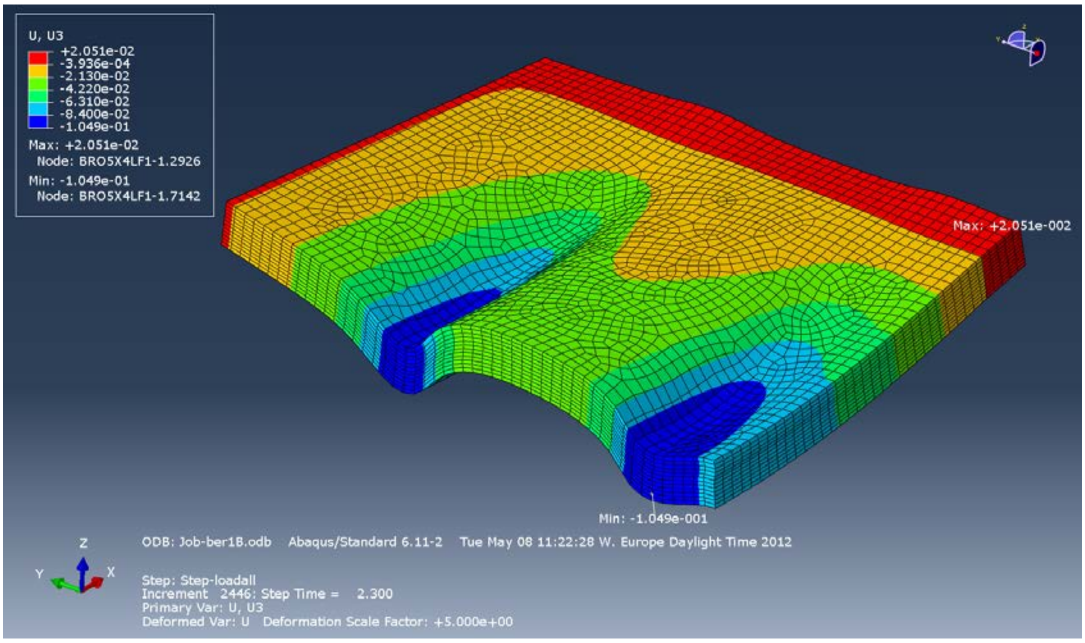


Fig.1. FE half-model of bridge deck 5x4 m. Vertical displacements (m).

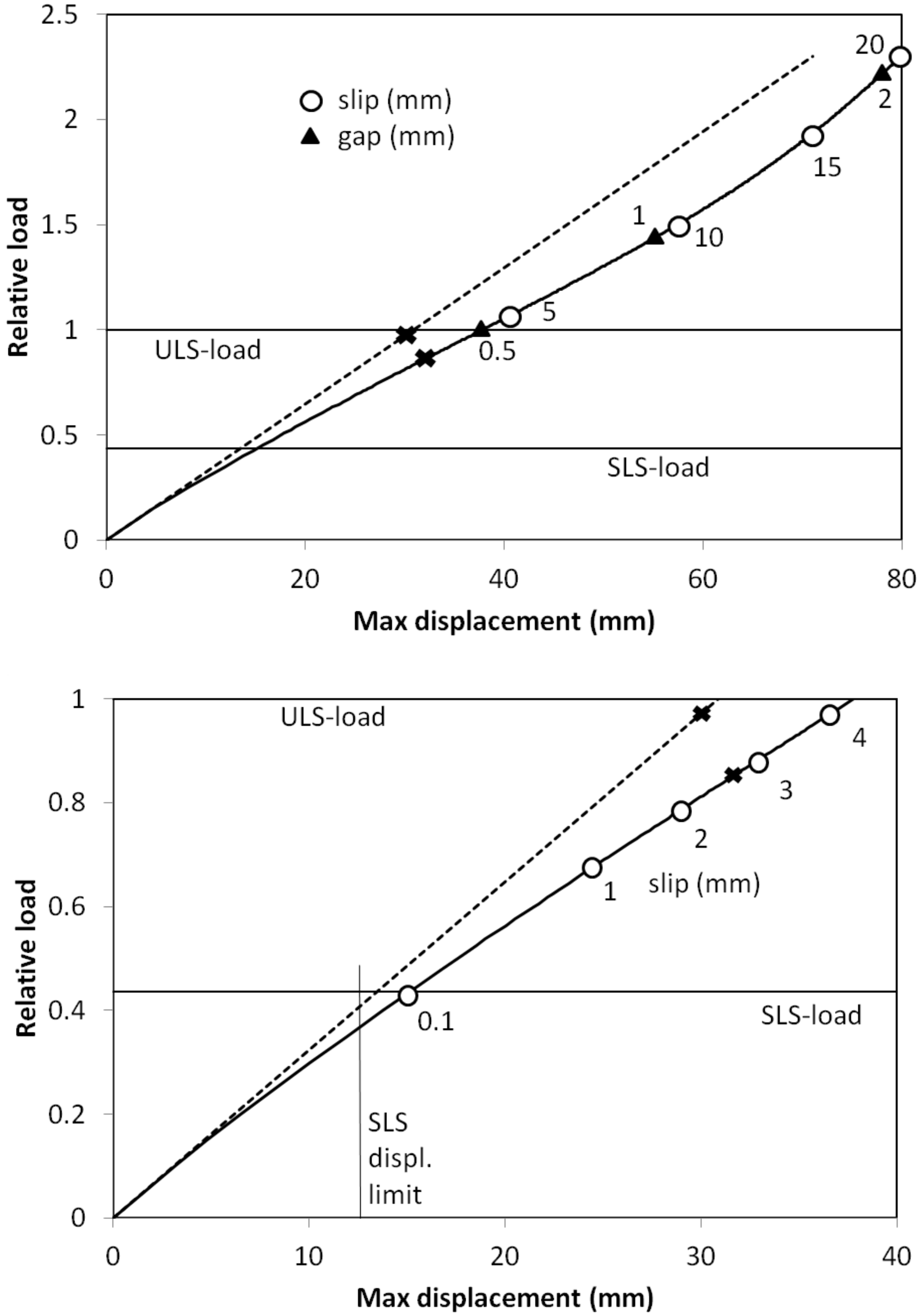


Fig.2a and b. Load vs. displacement curve for bridge deck 5x4 m (span x width), thickness 315 mm. Slip is maximum vertical slip between two beams and gap is maximum local gap between two beams. Crosses denotes where maximum ULS-design stresses are reached locally. Dotted lines are results from the linear model and solid lines are results from the nonlinear model.

Upper a) load level 0 to 2.3

Lower b) load level 0 to 1

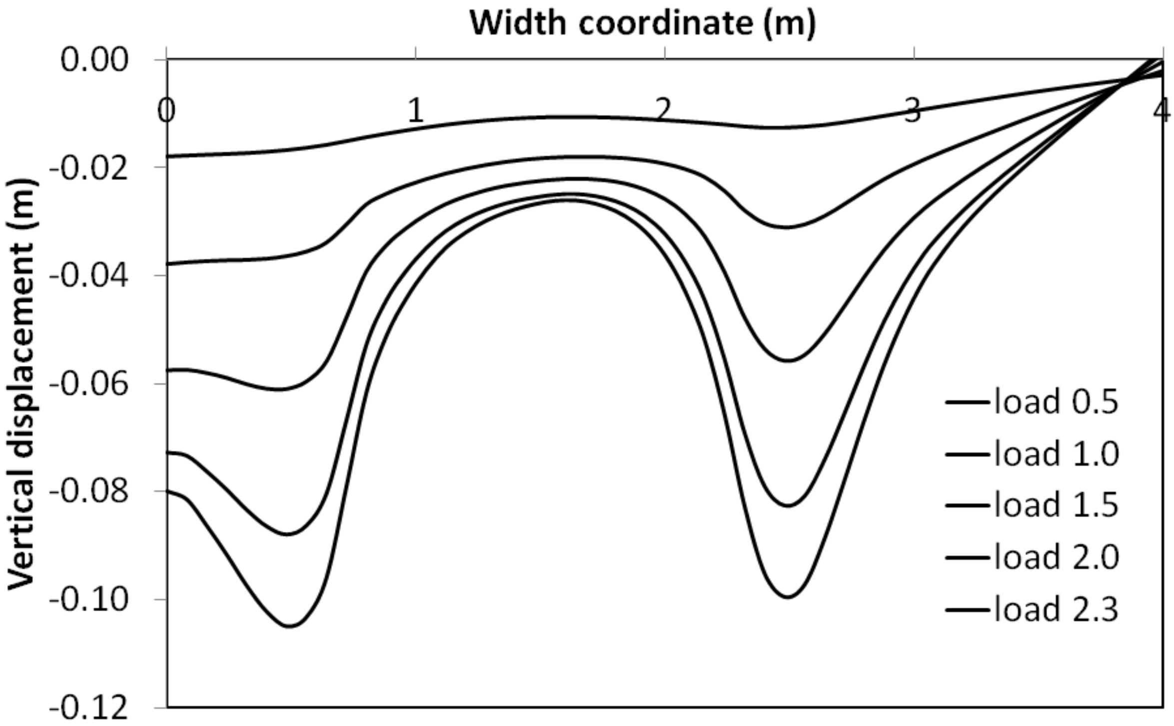
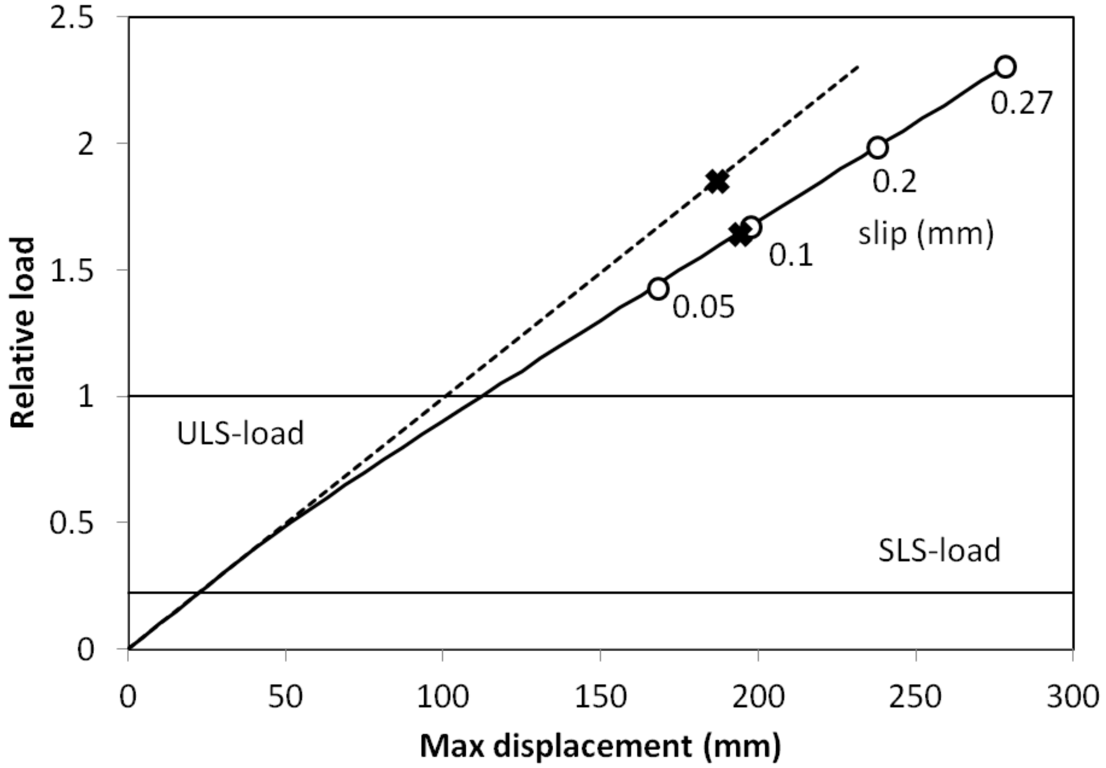


Fig.3. Bridge deck 5x4 m. Midsection displacement for different relative load levels from the nonlinear model.

4.2 Bridge deck 25x12 m

Figs.4a and 4b show linear and nonlinear load-displacement curves for bridge deck 25x12 m for loads up to 2.3 times the ULS-load. The maximum deformation in the nonlinear solution was about 11% higher than the linear solution at the ULS load and did not differ at all at the SLS load. The displacement was measured at a point on the very edge of the bridge deck in the midsection and this did not change for loads up to 2.3. No instabilities occurred for relative load levels up to 2.3 and maximum vertical slip was at the most 0.27 mm at relative load 2.3. ULS limits for bending stresses in fiber direction were reached at relative load levels 1.89 and 1.67 for the linear and nonlinear case, respectively. A limit of span/400=62.5 mm was used as the SLS limit for deformation. At load level 2.3 the horizontal slip was at the most 0.21 mm. Load vs. displacement curve started to deviate from linearity at load level 0.3. A gap started to form at load 2.2 and became at the most 0.01 mm in the midsection in the middle of the width on the upper deck surface.



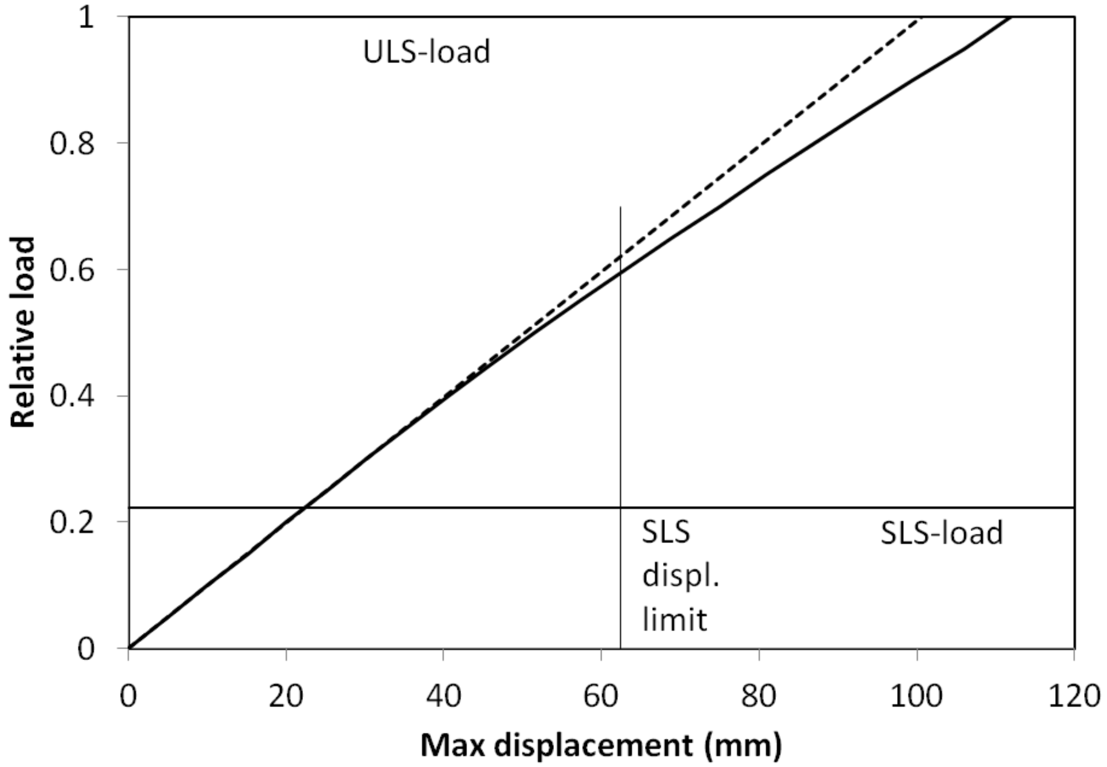


Fig.4a and b. Load vs. displacement curve for bridge deck 25x12 m (span x width), thickness 1125 mm. Slip is maximum vertical slip between two beams and gap is maximum local gap between two beams. Crosses denotes where maximum ULS-design stresses are reached locally. Dotted lines are results from the linear model and solid lines are results from the nonlinear model.

Upper a) load level 0 to 2.3

Lower b) load level 0 to 1

5. Analysis and discussion

The thickness 315 mm that was beforehand chosen for the 5x4 m deck was well suited and almost exactly fulfills the requirements according to ULS limits in the linear calculation (load 0.97, Fig.2a). However the nonlinear analysis showed that the linear calculation was non-conservative (ULS nonlinear limit load 0.84, see Fig.2a). Also shown from the nonlinear calculation results was that at the ULS limit the vertical slip was about 4 to 5 mm (Fig.2a and 2b) which may lead to failure of the deck surface sealing layer and/or paving on the deck. The maximum deformation in the nonlinear solution was higher than the linear solution due to slip. Vertical slip at SLS was much less than 1 mm which perhaps could be acceptable from viewpoint of integrity of sealing layer and paving. Gaps started to evolve just before reaching the relative load 1.0 and became 0.5 mm at load 1.0 but then increased up to 2 mm at load 2.3. Nonlinear effects in the midsection displacement curves make the point of maximum deformation change from a point on the edge to a point under a wheel load due to local vertical slip between beams at high load (see Fig.3).

The thickness 1125 mm for the 25x12 m deck resulted in lower than maximum allowable design stresses in ULS (Fig.4a), thus this deck thickness could have been reduced. Also for this deck the linear calculation in ULS gave allowable loads that were higher than the allowable loads from the nonlinear calculation (load 1.89 cf.1.67 allowable load in ULS). SLS load (0.222) was lower for this deck than for the 5x4 m deck due to the larger relative contribution from gravity load of wood compared to wheel load for this deck. SLS limits for deformation were easily disposed of with a large margin (Fig. 4b). There was no vertical slip at load 1.0 and only 0.27 mm vertical slip at load 2.3. Gaps were negligible and did only appear above relative load 2.2.

For the studied deck geometries nonlinear effects were only significant for the 5x4 m deck since the 25x12 m deck did cope with all restrictions (limits) both in the linear and nonlinear calculation. The nonlinear calculation of the 5x4 m deck showed that two different potential problems arouse. The first was that the ULS design stresses were too high (only 0.84 load was allowed) and the second was that the vertical slip in ULS was 4 to 5 mm. The first problem could be a non-relevant problem if it is that the Eurocode design stress limits are set in a way to cope with the difference between linear and nonlinear stress results. Regarding the other potential problem related to vertical slip then it is not yet known whether this is a real problem or not or if this can be solved by the bridge producers.

6. Acknowledgement

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7. References

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