Sensitivity Study for Computational Assessment of Prestressed Concrete Bridges

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Summary

A current challenge for civil engineers is the conservation of existing infrastructures which includes maintenance, assessment and, if necessary, strengthening according to new code requirements. For the evaluation of the actual load carrying capacity and the remaining service life, detailed knowledge of the material properties and a realistic structural model is required. If necessary, latter has to be adjusted for existing structures by in-situ measurements and inspections. In this paper, the structural behaviour of a concrete hollow box girder bridge is investigated by nonlinear analysis with a 3D finite element model. Based on these results a sensitivity study is performed by variation of material properties.

In a previous case study it was shown how eccentrically arranged traffic loads are inducted into the bridge-system and in which way the maximum stressing of a cross section could be calculated. For this, a 3D finite element model with the assumption of nonlinear material behaviour was used.

In a first step the redistribution of section forces in the transverse and longitudinal direction of the structural system due to nonlinear material behaviour is discussed. It will be shown in which way the force flow is influenced by nonlinear material behaviour in comparison to a linear elastic analysis. Second, the ultimate limit state (ULS) is discussed by means of a sensitivity analysis. Finally, the influence of the used material assumption and the statical system in nonlinear calculations to the load bearing capacity is discussed. This might help in the judgement whether the service life of a bridge can be reached or a strengthening is necessary.

Keywords: concrete structures; nonlinear analysis; finite element; hollow box girder bridge

1. Introduction

The construction of prestressed concrete bridges started in the 1940s and reached its peak in the 1960s. By that time, the need for new bridges was very large and it was extensively taken advantage of the prestressing technology. Today, higher traffic loads and serious durability problems require regular inspections of the bridges. Since the planned service life of most of these bridges is not reached, it is often more economic to maintain and strengthen them, rather than replacing.

First problems with such constructions occurred in the 1970s. These bridges were usually built in sections i.e. a new section was coupled to an older one whose concrete has already hardened. Because in these coupling joints the concrete tensile strength is lower than in other areas, they were located in regions which are almost not stressed in tension due to permanent loads. Typically, the joints were located at a distance of approximately 0,2L from an intermediate support, where L is the length of one span. At these joints the prestressing tendons are coupled too, which causes an additional weakening of the cross section (Fig. 1). In the 1970s, cracks in the coupling joints in many bridges of this construction type were observed and after the collapse of one bridge a special regulation was published in 1977 [1]. This regulation specified the dimensioning and the construction of such coupling joints. The main causes for the defects at the coupling joints were found in an oversimplified statical system, insufficient consideration of temperature effects and neglecting of additional section forces due to the deformation of the cross section.

In [2] by means of an idealized bridge model it was shown that additional forces due to the deformation of the cross section occur if one web of a hollow box is directly loaded with a line load or a single load in the middle of the span. In this case the additional forces can be determined analytically as e.g. demonstrated in [3] and [4]. Using a 3D finite element model it was further demonstrated that if the loads are located close to the support and not directly above the web no significant additional forces arise. The differences between detailed and simplified calculations result from varying levels of detail regarding the application and distribution of loads in the structural system [6].

In this paper it will be discussed how the load bearing capacity of hollow box girder bridges can be determined, if these bridges are loaded with dead and traffic loads [7]. For this purpose a 3D finite element model (FEM) with shell elements considering nonlinear material properties of reinforced concrete is used. Finally, with the help of sensitivity analysis it will be discussed which influence different material parameters have to the load bearing capacity. Depending on the variation of these material parameters a variation of load bearing capacities is a result out of it.

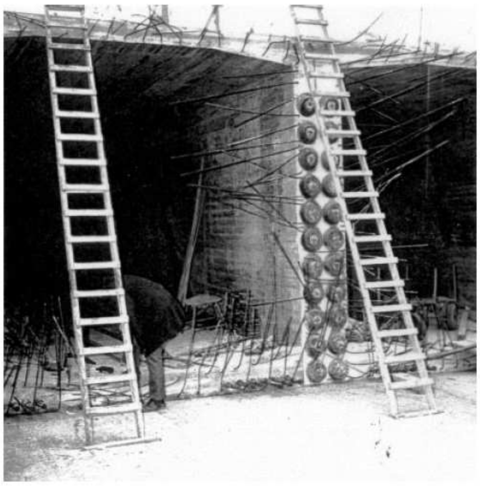

Fig. 1: Coupling joint of a prestressed concrete bridge [5]

2. Computational Examination of Existing Bridges

2.1 Introduction

In 2009 a study was published [5] in which it is shown that almost 90% of sequentially built bridges of the years 1958 to 1976 had cracks or damaged coupling joins and strengthening was necessary. 78 out of 118 bridges built until 1979 have a hollow box cross section, so that this study concentrates on that type of bridges (Fig. 2). The quality of the bridges and of the coupling joints, respectively, increases with the year of construction. The question arises whether global or local strengthening of such bridges is really necessary or a corrosion protection might be sufficient. For an assessment of a strengthening and the remaining service life and the global safety factor to the actual code loads have to be determined.

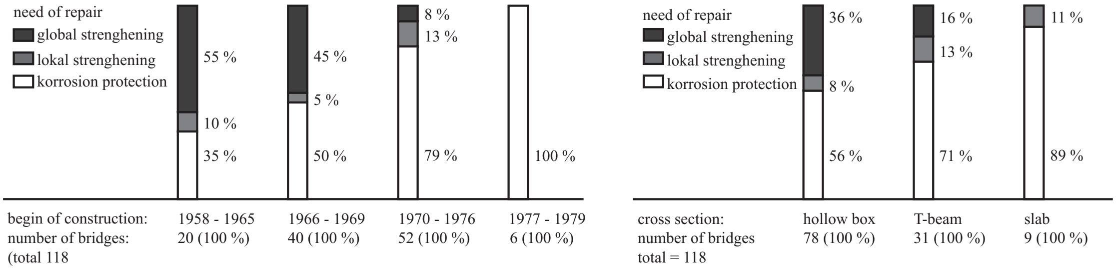

Fig. 2: Need of repair of existing bridges [5]

2.2 Exemplary Analysis on a Hollow Box Girder Bridge

2.2.1 Shear Forces in a Two-Cell Cross Section due to Code Loads

The effect of a group of loads movable in the bridge's transverse direction on the sectional forces is shown in [6]. Here, the entire load case according to the German design code [7] is used; the movable loads are located with the maximum eccentricity in transverse direction and in a distance of $2{,}0H$ from the intermediate support to perform a maximum shear load to the outer web near this support [6].

In this paper a two span bridge is studied with a span of 60 m. The focus is on the distribution of shear forces at a distance of $1{,}0H$ from the support. The Cross section and loads are depicted in Fig. 3; the dimensions are listed in Table 1.

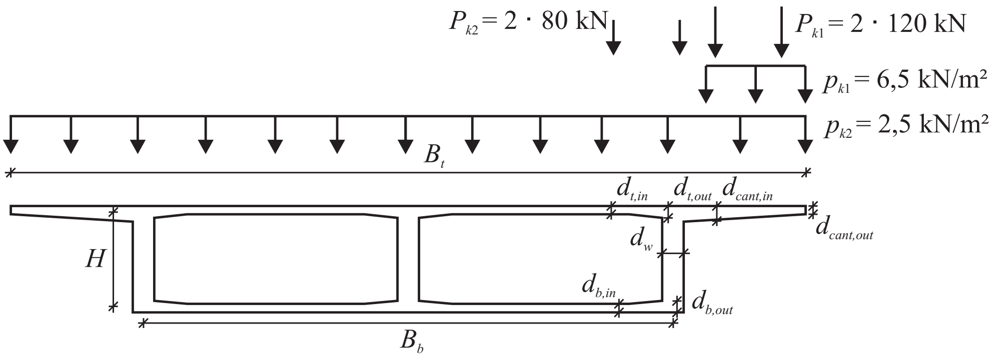

Fig. 3: Cross sections and loads

Table 1: Dimensions of the cross section

B_t	B_b	H	$d_{t,in}$	$d_{t,out}$	$d_{cant,in}$	$d_{cant,out}$	d_w	$d_{b,in}$	$d_{b,out}$
16 m	8 m	3 m	0,25 m	0,35 m	0,45 m	0,25 m	0,65 m	0,25 m	0,35 m

2.2.2 Material Parameters for Sensitivity Analysis

To perform an analysis of an existing structure the geometry and the used materials have to be known. The geometry of the bridge can be taken from construction plans or by an in-situ measurement. In this study the variation of the geometry is neglected. Defining the service life of the structure the knowledge of the used material and there actual properties is deciding. For the here performed analysis standard values for the concrete and reinforcement steel is used. Due to different in house testing the mean values (σ) and the coefficient of variations (μ) of these materials could be determined: it will be used a concrete with a mean compression strength of 40 MPa and a reinforcing steel S500B; the material parameter are listed below. In accordance to JCSS [10] a normal distribution for concrete and a lognormal distribution for reinforcing steel are adopted.

Table 2: Material parameter: mean values and coefficient of variation

Parameter	σ [MPa]	μ [%]
f_{cm}	40	1,8
$f_{ct,m}$	2	-
E_{cm}	28500	2,5
$f_{y,m}$	490	2,4

3. 3D nonlinear FEM-Model

3.1 Finite Element Model

For the nonlinear calculations the FE-model is discretized by shell elements (Fig. 4). In the shell elements smeared reinforcement is considered. A nonlinear constitutive law for concrete in compression is used. The tensile behaviour is assumed linear elastic until reaching the maximum tensile strength followed by a softening branch that continuously descends to zero at a predefined strain [8]. The variation of the tensile strength will be neglected. The reinforcing steel is approximated with a bilinear stress-strain relationship. By using this FEM material models bending and tensile tests published in [9] have been simulated in a good agreement.

To take into account the concrete cracking it is necessary to know the amount of reinforcement before running the calculation. Therefore, in a preliminary step the reinforcement was pre-dimensioned. Prestressing is determined on the basis of the load balancing method. A compensation of 100% of the dead loads and 30% of the uniformly distributed live loads and a parabolic tendon profile in the webs are assumed. The prestressing effects are modelled as externally applied loads.

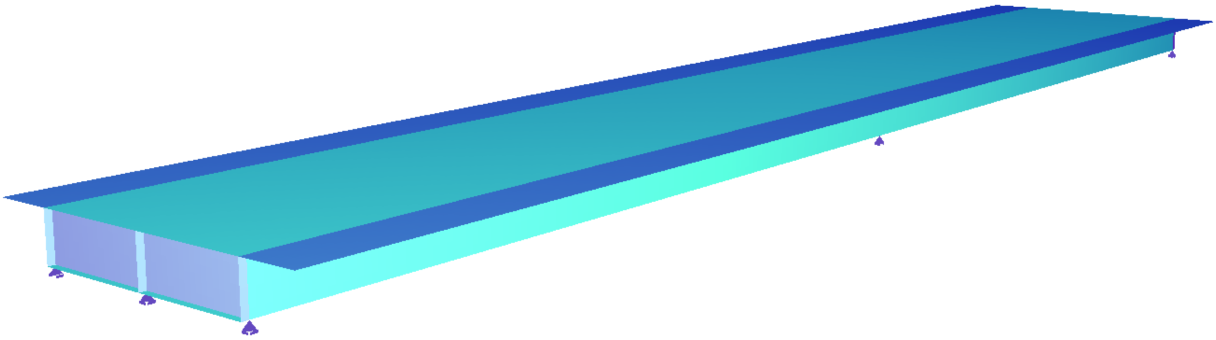

Fig. 4: FE-Model

Using nonlinear calculations the principle of superposition is not applicable. This means that results of ultimate loads or global safety factors are coupled to a specific load case. In this paper only shear forces due to movable loads (axle, main and adjacent track loads) are considered. Therefore, only the live loads will be increased to determine the ultimate load (Fig. 3). The axle loads are located at a distance of 2,0*H* to the intermediate support with a maximum eccentricity on the bridge deck; where *H* is the height of the cross section; the dead load is constant on the design level. For more details on the load position causing maximum shear forces in the respective cross section it is referred to [2].

3.2 Nonlinear Calculations of Load Bearing Capacity

Fig. 5 shows a comparison of the shear forces only in the webs at the distance of 1,0*H* to the intermediate and end support calculated with the assumption of a linear elastic and nonlinear material behaviour, respectively. In this example the dead load and the prestressing are at the design level. The adjacent and main track load acts on the complete bridge deck on both spans (2 x 60m), the axle loads are located at a distance of 2,0*H* to the intermediate support with a maximum eccentricity on the bridge deck. This traffic load is increased incrementally until the maximum load bearing capacity with the factor f_P is reached. For this calculation mean values of the material properties are used (Table 2). The load bearing capacity is limited by the biaxial concrete compression strength due to bending near to the intermediate support.

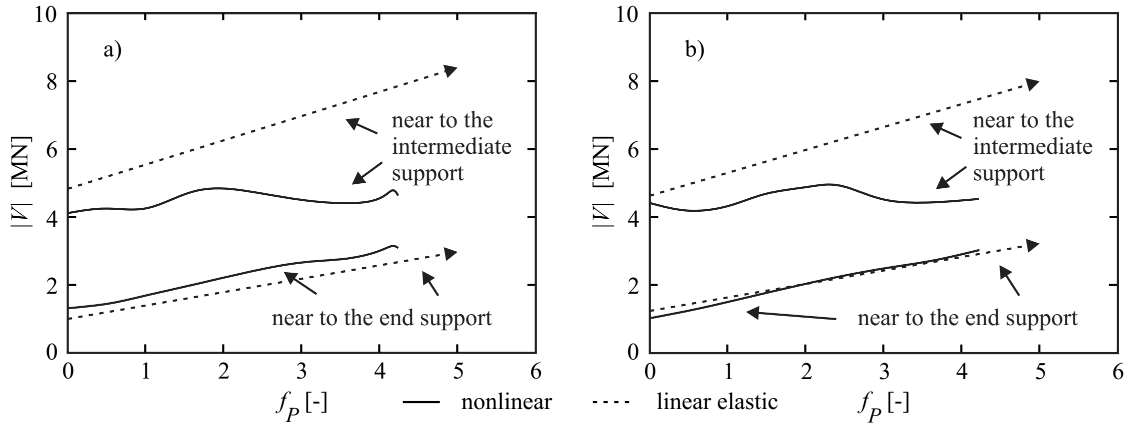

Fig. 5: Shear forces at 1,0H from the support due to the adjacent load (dotted lines for linear elastic and full lines for nonlinear calculations): a) outer webs, b) inner web

Fig. 5 shows the shear forces in the webs at the distance of 1,0*H* from the end support. The shear forces near to the intermediate support are smaller taken into account nonlinear material behaviour than for the linear elastic approach and greater at the end supports. In that stage, the FE-model implies cracking due to bending in longitudinal and transversal direction. The differences between the linear and nonlinear simulation cannot be only explained by the redistribution of the forces in longitudinal direction because the reaction forces for both simulations are nearly equal. Therefore the redistribution of the shear forces is caused by nonlinearities within the cross section and the slabs close to the web (Fig 6).

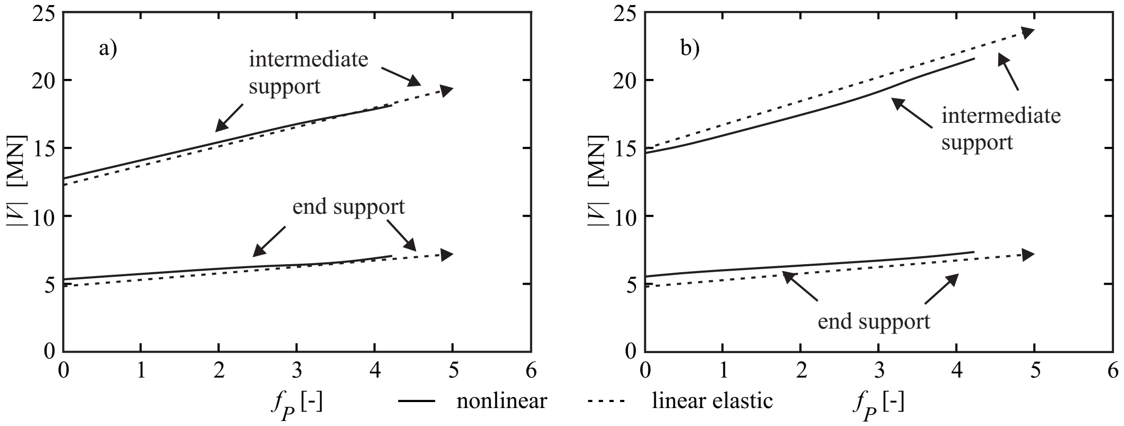

Fig. 6: Reaction forces (dotted lines for linear elastic and full lines for nonlinear calculations): a) outer webs, b) inner web

Due to cracking, parts of the slabs close to the web contribute to the shear transfer in longitudinal direction (Fig. 7). In Fig. 7 shear forces in longitudinal direction in the cantilever plate and the deck slab at the distance of 1,0d_i of the regarded cross section (see Table 1for d_i) to the web. This is one of the reasons why the forces at the intermediate support are smaller for the nonlinear simulation. Additionally, the flow of forces in the slab changes close to the intermediate support where cracks occurred due to bending in transverse and longitudinal direction. Therefore, the load is transferred more directly to the transverse stiffening wall.

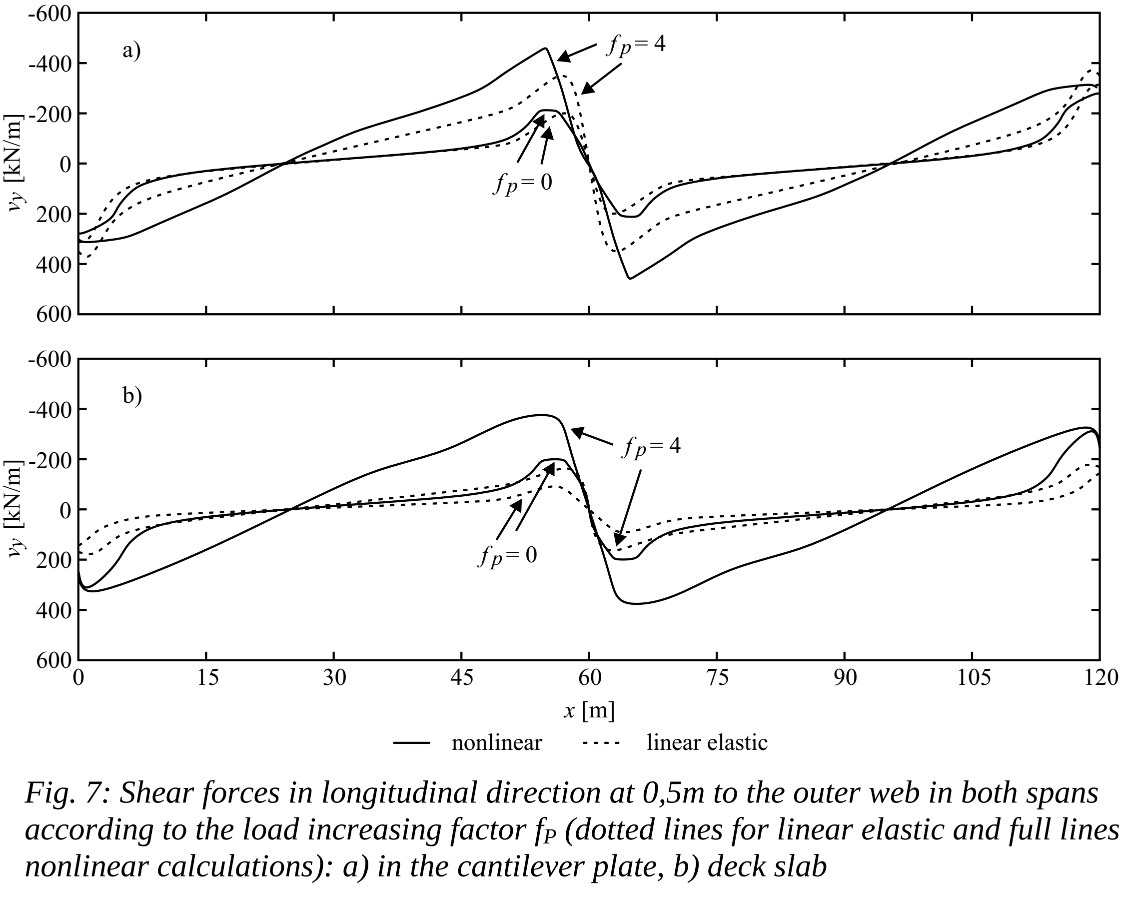

Fig. 7: Shear forces in longitudinal direction at 0,5m to the outer web in both spans according to the load increasing factor f_P (dotted lines for linear elastic and full lines for nonlinear calculations): a) in the cantilever plate, b) deck slab

Due to transverse cracks in the cantilever slab near to the end support, uniformly distributed loads are transferred more directly to the web as in a linear elastic structure. This results in an increase of shear forces in the webs at the distance of 1,0*H*.

4. Sensitivity Study

The presented results indicate that the structural behaviour and the load bearing capacity can be simulated realistic. In a computational assessment of existing bridges the knowledge of the used materials is necessary. It is mostly difficult to get the information about them because the structural plans often do not exist anymore. In lack of these information the engineer have to assess them from his experience (reinforcing and prestressing steel) or use values from retrieved material samples. These values show a scattering of their mechanical properties.

On the basis of a statistical evaluation it is possible to define a mean value and a standard deviation for concrete and reinforcing steel (Table 2); comparable values can be found in [10]. These values are used to perform a sample of simulations on the presented bridge to assess the load bearing capacities. In this study 20 nonlinear simulations were performed. The influence of the bridge geometry is neglected and could be taken into account in a following, more detailed investigation.

The results of nonlinear calculations with all material combination of Table 2 are depicted in Fig. 8. In Fig. 8 a) shear forces in the outer web near to the intermediate support in dependency of the load increasing factor f_P are shown. The main outcome is that the variation of material properties has a minor influence to the shear forces in the ultimate limit state. Similar to the results in Fig. 5, the shear forces do not overstep a maximum value. Due to the transversal redistribution of the forces in the cross section the shear forces in the web could not increase constantly. Due to cracking and first local yielding of the bending reinforcement in longitudinal and transversal direction the decrease of the stiffness of the cross section resulting in change of the force flow in both directions. Parts of the slabs near the web contribute to the shear transfer in longitudinal direction. As mentioned before, the load bearing capacity is limited with the maximum biaxial stress in compression of concrete near to the intermediate support (Fig. 8 b)).

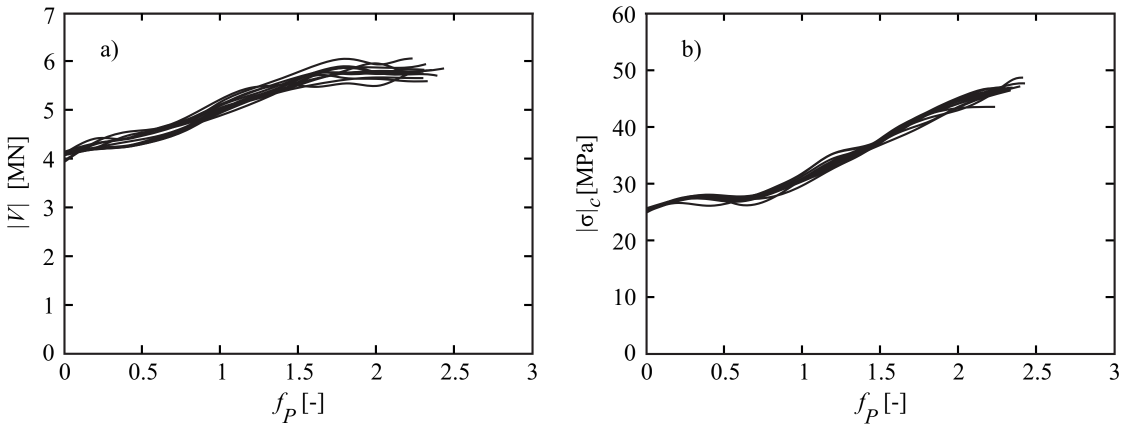

Fig. 8:Values due to increasing traffic load factor f_P: a) Shear forces at 1,0H from the intermediate support in the outer web, b) concrete compression stress due to bending near the intermediate support in the outer web

The evaluation of the maximum load factor f_P show a mean value of the bearing load capacity of $f_{P,m}$ = 2,32 (Table 3) and a coefficient of variation of 6%. Latter indicates that the variation of material properties has a minor influence. The load bearing capacity depends mainly on the statical system of the regarded bridge, respectively the statically indeterminate of the structure. Local weakening like gravel pockets or damaged reinforcing rebar due to fatigue, for example, might have an influence; this should be investigated in detail. General statements to their influence can not be done at this position.

Table 3: Bearing load factor

Parameter	$f_{P,m}$ [-]	μ [%]
Bearing load factor	2,32	6,0

5. Discussion and Conclusions

This paper deals with the computational assessment of a prestressed concrete bridge. For the assessment of the load bearing capacity due to nonlinear material properties a 3D finite element bridge model was discretized using shell elements with smeared reinforcement within the element formulation.

It was shown how shear forces were redistributed in the bridge when concrete cracking and local steel yielding occurred. The shear forces in the control section near to the support did not overstep a maximum value although if the traffic loads were increased. This can be explained with the redistribution of forces in the bridge. Due to concrete cracking, parts of the slabs close to the web contribute to the shear transfer in longitudinal direction.

In a second step a sensitivity analysis was done by means of varying the material parameters of concrete and reinforcing steel. The latter has a minor effect and shear forces are mainly influenced by the statical system of the bridge. Based on these first results from a sensitivity analysis a more detailed probabilistic approach will be done as next step, so that a remaining service life can be determined more exactly.

6. Acknowledgements

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