Computational Assessment of Prestressed Concrete Bridges

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Summary

A current challenge for engineers is the conservation of existing structures which includes maintenance, assessment and, if necessary, strengthening. For the evaluation of the load carrying capacity and the remaining service life, detailed knowledge of material properties is required and a structural model should be used, with which the stress state can reliably be analysed. In this paper, concrete hollow box girder bridges (single and multi-cell cross sections) are examined.

First, on the basis of a case study it is shown how eccentrically arranged traffic loads are introduced into the system. For this, a 3d finite element model with the assumption of linear elastic material behaviour is used. The results are compared to those of a beam based calculation where, regarding the shear forces, considerable differences are found.

Second, the influence of the reduction of the stiffness due to cracking of the concrete is discussed. Finally, the effect of the decrease of prestress as a result of damaged coupling joints is outlined.

Keywords: concrete bridges, coupling joints; prestressed concrete; structural assessment.

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 intended service life of most of these bridges is not reached, rather than replacing them it is often more economic to maintain and, if necessary, strengthen them.

First problems with the new construction method occurred in the 1970s. The bridges were usually built in sections i.e. a new section was attached to an older one whose concrete has already hardened. As 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 of young bridges were detected and after the collapse of a bridge a special regulation was published in 1977 [1], in which the dimensioning and construction of the coupling joints is specified. The main causes for the defects at the coupling joints were seen in oversimplified statical systems, insufficient consideration of temperature effects and neglect of additional 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

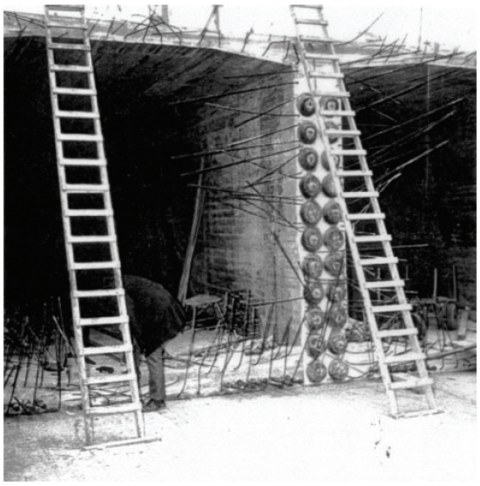

Fig. 1: Coupling joint of a prestressed concrete bridge [5]

significant additional forces arise. The differences between detailed and simplified calculations result from varying levels of detail regarding the introduction and distribution of loads in the structural components.

In this contribution the transfer of loads in bridge girders is discussed in detail. For comparison purposes the shear forces in the webs determined at a distance of 1,0*H* from the support (*H* denoting the depth of the cross section) are looked at. On the one hand, the shear forces are calculated with the beam based method (BBM) and on the other hand with a 3d finite element model (FEM) using shell elements. In a first step, a linear elastic material model is used for the shell elements. Thereafter, nonlinear material behaviour is implemented to show differences in the results when cracking of the concrete and yielding of the steel as accounted for. Finally, the influence of damaged coupling joints on the load carrying capacity is discussed.

2. Computational Examination of Existing Bridges

2.1 Introduction

In 2009 a study was published [5] in which it is shown that almost 2/3 of sequentially built bridges of the years 1958 to 1965 had damaged coupling joins and strengthening was necessary. 43 out of 122 bridges built until 1979 had crack widths in the range of 0,2 to 0,4 mm in the coupling joints (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. On the basis of a case study it is shown in which cases a detailed calculation is necessary and how influences like concrete cracking and steel yielding can be taken into account.

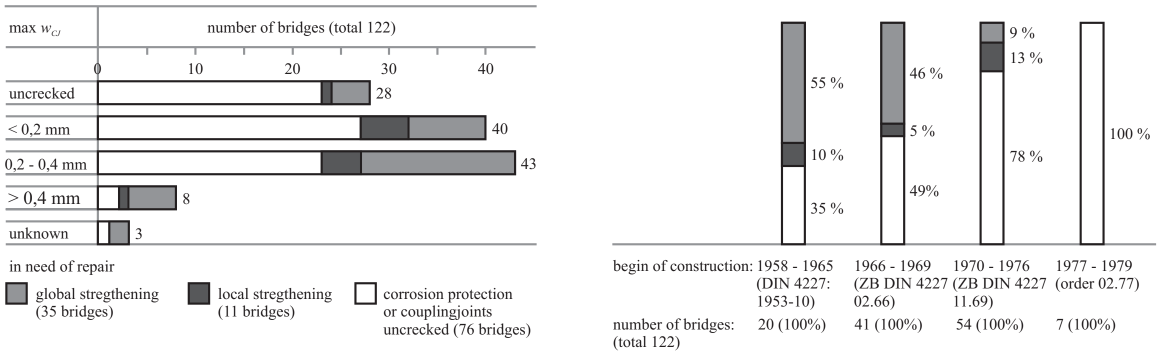

Fig. 2: Number of bridges with cracks in coupling joints [5]

2.2 Case Study on Hollow Box Girder Bridge

2.2.1 Single- and Multi-Cell Cross Sections

The effect of a group of loads movable in the bridge's transverse direction on the sectional forces is studied. Here, the entire load case according to the German design code [6] is used; in the figures only the results due to axle loads are depicted (results due to loads in the traffic lanes can be found in [2]). Constant distributed loads (dead load and side lane loads) are not considered here because they produce evenly distributed sectional forces. The results calculated with the beam based method (BBM) are compared to those determined with the finite element method (FEM). The focus is on the shear forces at a distance of 1,0*H* from the support. The study not only includes single-cell but also multi-cell cross sections; two span bridges are studied with a span of 60 m. Cross sections and loads are shown in Fig. 3; the dimensions are listed in Table 1; the ratio B_b/H is the same for all cross sections.

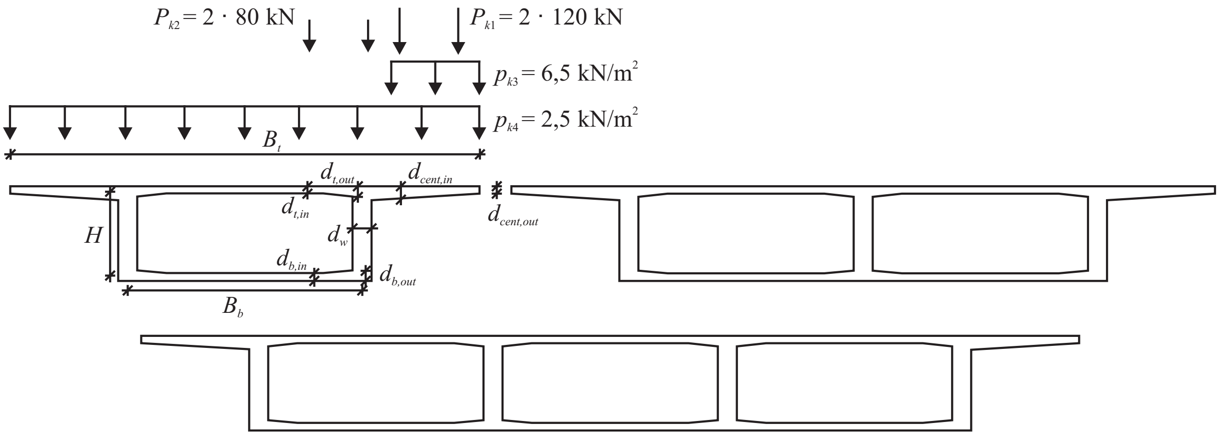

Fig. 3: Cross sections and loads

Table 1: Dimensions of the cross section

B_t	B_b	H	$d_{t,in}$	$d_{t,out}$	$d_{cent,in}$	$d_{cent,out}$	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

According to the BBM the shear force in the web is calculated with $V_{tot,web}=V_{Q,web}+V_{T,web}$ (1), where $V_{Q,web}$ is due to bending and $V_{T,web}$ due to torsion. The shear force $V_{Q,web}$ is equally distributed on the webs, i.e. $V_{Q,web}=V_{tot}/n_{web}$ (2), V_{tot} denoting the total shear force and n_{web} the number of webs. By contrast, $V_{T,web}$ is taken by the outer webs only, i.e. $V_{T,web}=T_{tot}/(2B)$ (3); T_{tot} is the total torsional moment and B the distance between the outer webs. These shear forces are compared to the FEM results. In the latter case, only the shear force contributions from the FEM-elements representing the web are considered; shear forces from the remaining parts of the cross section (like the slabs) are neglected.

For these calculations the notation of Fig. 4 is used. The value a represents the distance between the centre axis of the cross section and the geometric centre of the load; b is half the width of the hollow box and independent of the number of the cells.

Fig. 5 shows the comparison of the BBM and the FEM results in one web. The shear forces are plotted as a function of the a/b ratio which is a measure of the eccentricity of the load. As mentioned above, only shear forces due to the group of axle loads are looked at. To maximize the forces at the distance of 1,0H from the support they have to be placed at a distance of 2,0H [2] which is due to the spatial load introduction. Hence, for the three left diagrams of Fig. 5 the loads are close to the end support and for the other three close to the intermediate support. For comparison purposes, in the right hand sided diagrams forces are shown as negative values. Fig. 5 illustrates that the forces calculated with the FEM do not mach with the results based on the BBM.

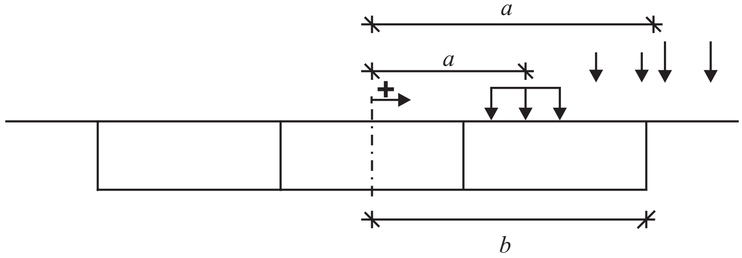

Fig. 4: Position of loads

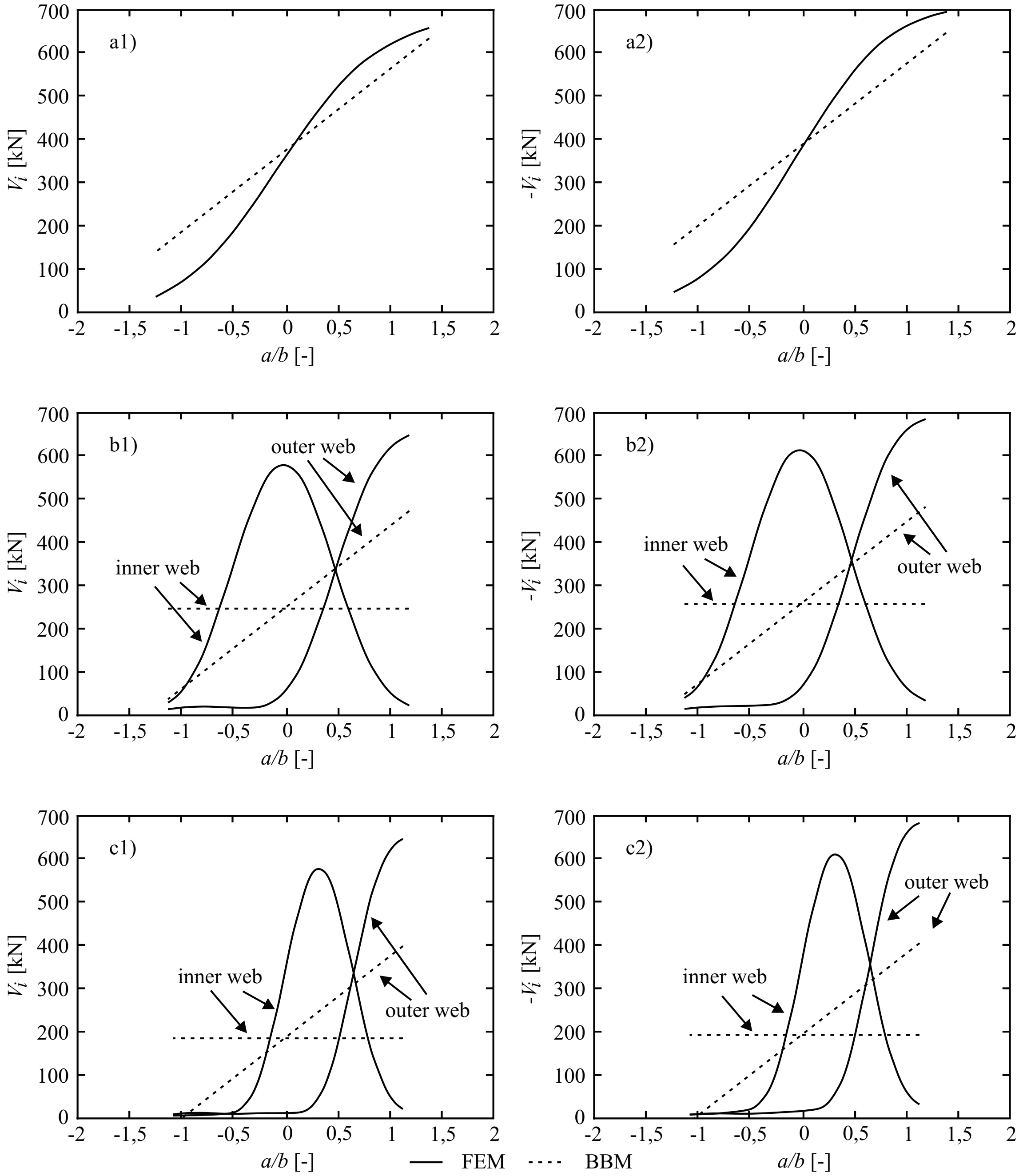

Fig. 5: Shear force at 1,0H from the support due to horizontally movable axle loads (dotted lines for beam based method and full lines for 3D FEM); a) one cell, b) two cells, c) three cells; 1) end support, 2) intermediate support

The distribution of the shear forces and the shear flow depend on the transverse stiffness of the cross section. For the equations (2) and (3) it is assumed that the cross section has an infinitely high stiffness in the transverse direction which in reality is not given. The closer the loads approach a web, the more force remains in this web. For the two-cell and three-cell cross sections, see Fig. 5 b2) and c2), the difference between BBM and FEM increases if the cell number increases. Thereby, the total shear force is independent of the number of the cells. In the diagrams of Fig. 5 it can also be seen that, if loaded directly, the inner web of a multi-cell cross section is always less stressed than the outer webs.

The reason for this is the tendency of the forces to flow to the nearby webs. In general, an increased stiffness of the slabs in transverse direction augments and an increased stiffness of the webs in longitudinal direction reduces this tendency [2]. For Fig. 5, only shear forces due to axle loads are considered; the findings from this case study can be transferred to loads on the main traffic lanes without restriction [2].

2.2.2 Simplified Calculation

The above considerations indicate that the shear forces due to code loads and the relevant locations for dimensioning are almost independent of the cell number, but the differences between FEM and BBM increase with the number of cells. This is mainly because the BBM assumes all webs to act equivalently in bending and shear (equation (2)). As illustrated in Fig. 5, for multi-cell hollow boxes and regions in the vicinity of the applied single loads this assumption is not very accurate.

For a preliminary design or a first check of a bridge a simplified structural system for calculating the shear forces due to eccentric loads can be used. Figure 6 shows a possible configuration to determine the maximum shear force close to the support with the BBM. In this model, the axle loads are placed at a distance of 1,0H from the support. Only the torsion creating part of the main traffic lane load has to be included and only the shear force due to bending is to calculate. By doing so, the characteristic shear force due to the axle loads is approximately 730 kN. A detailed calculation with the FEM yields 650 kN. For the outer webs this is a good approximation; the inner webs are slightly overestimated.

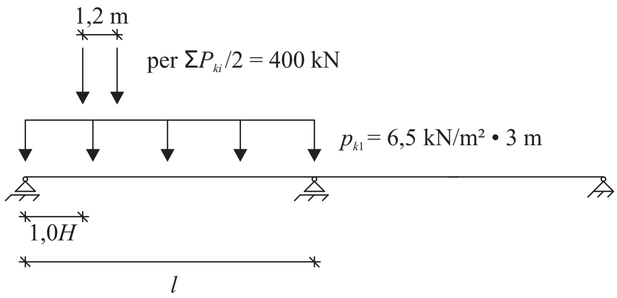

Fig. 6: Equivalent static system for eccentric loads

2.3 3d Models considering Material Nonlinearities

2.3.1 Finite Element Model

For the nonlinear calculations the same FE-models are used as for the linear elastic calculation. In the shell elements smeared reinforcement is considered. A nonlinear behaviour of concrete in compression with a constant maximum value at failure is used. The tensile behaviour of concrete is assumed as linear elastic until reaching tensile strength followed by a softening branch that continuously descends to zero at a predefined strain [7]. The behaviour of the reinforcing steel is approximated with a bilinear stress-strain relationship. By using this FEM material model bending and tensile tests published in [8] have been recalculated and good agreement was found.

To include the cracking behaviour in the calculations it is necessary to know the reinforcement. Therefore, in a preliminary working step the reinforcement is dimensioned with help of the BBM.. 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.

2.3.2 Comparison of Linear and Nonlinear Calculations

With the above assumptions it is possible to make a case study analogue to the one of section 2.2.1. Fig. 7 shows a comparison of the shear forces calculated on the basis of a linear elastic and nonlinear material models, respectively. Different to Fig. 5, the design values of the loads are used here and a completely loaded system is taken into consideration. Results from separate loading configurations cannot be used because in a nonlinear calculation the principle of superposition is not applicable. With regard to the a/b ratio the axle loads and the eccentric part of the main traffic lane load are moved simultaneously.

Fig. 7 shows that shear forces close to the intermediate support are smaller for the nonlinear material behaviour than for the linear elastic approach and bigger at the end supports. In that stage, the FE-model comprises cracking due to bending at mid-span and above the intermediate support. The differences between the two calculation methods cannot be explained by the redistribution of the forces in longitudinal direction only; they are caused by a different flow of the forces within the structure. The reaction forces are equal for both calculation methods.

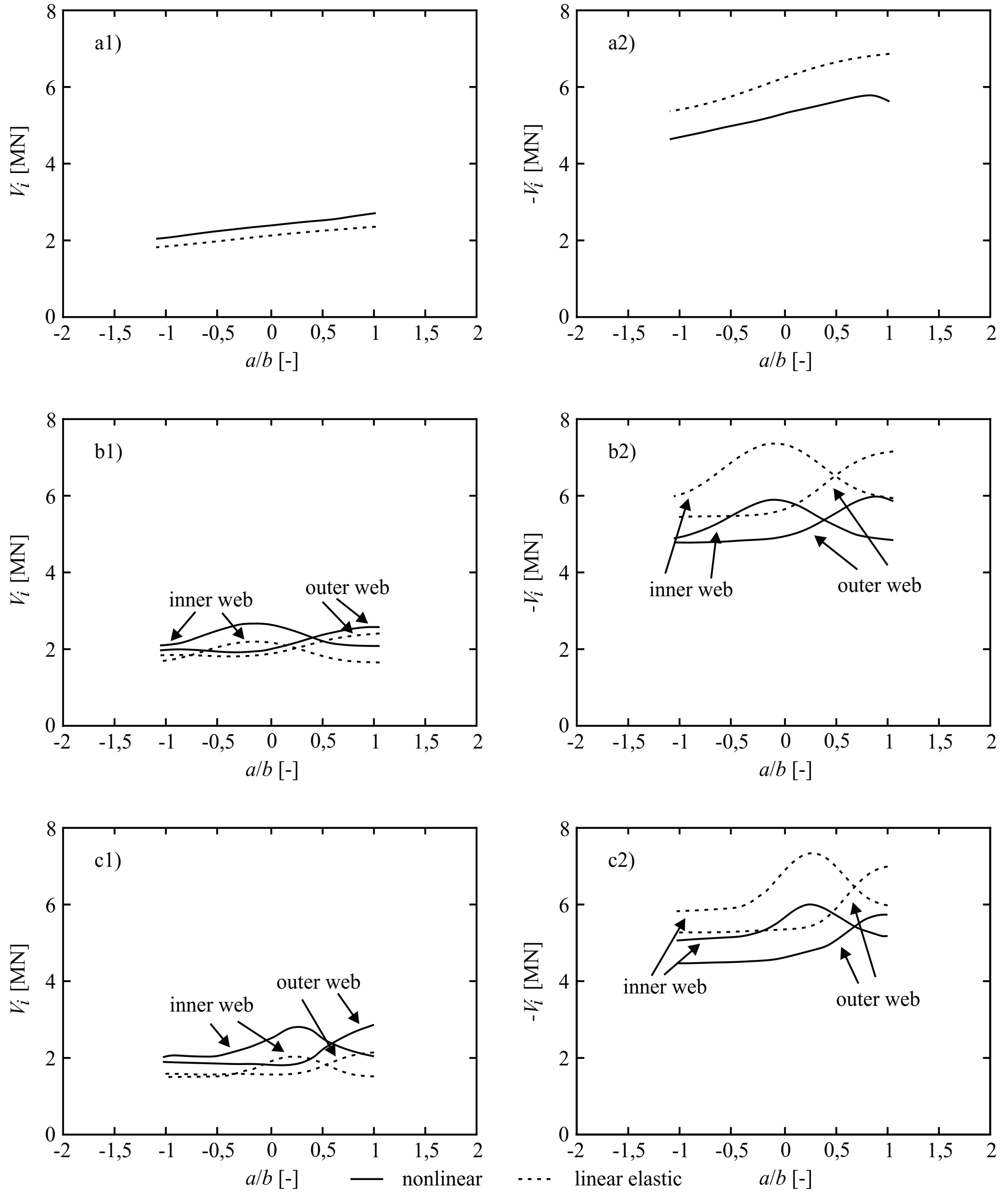

Fig. 7: Shear force at 1,0H from the support due to horizontally movable axle loads (dotted lines for linear elastic and full lines for nonlinear calculations): a) one cell, b) two cells, c) three cells; 1) end support, 2) intermediate support

Due to cracking, parts of the slabs close to the web contribute to the shear transfer in longitudinal direction. This is one of the reasons why the forces at the intermediate support are smaller for the nonlinear calculation. Additionally, the flow of forces in the slab changes in the vicinity of the intermediate support where cracks due to bending in transverse and longitudinal direction occur; here, the load is transferred more directly to the transverse stiffening wall.

Due to transverse cracks in the cantilever slab at the end support, uniformly distributed loads are transferred more directly to the web as in an uncracked structure; therefore, the shear force at the distance of 1,0H increases.

3. Influence of Damaged Coupling Joints

3.1 Introduction

Fig. 2 shows that existing bridges exhibit considerable cracking at the coupling joints and hence, there is a necessity to repair and/or strengthen the structures due to increasing traffic loads. The following study on bridge structures with single- and multi-cell cross sections (Fig. 3) shows on the one hand the possibility of determining a potential damage of the prestressing tendons in the coupling joints; on the other hand it is demonstrated that local strengthening can be sufficient if the global load bearing capacity is guaranteed.

For the case study the bridges are loaded with design values of loads. The damaged coupling joint is located at 0,2*L* from the intermediate support. The traffic lane loads are placed only in the span with the coupling joint, the main traffic lane and the axle loads are arranged with their maximum eccentricity to the outer web. The axle loads are assumed to be directly above the coupling joint. Based on section 2 the coupling joint is stressed at its maximum with this load configuration.

In the case study the prestress (along the bond length) in the coupling joint is reduced incrementally and the influence on the crack widths and the stresses in the mild steel reinforcement is looked at. As an example, one damaged coupling join in the outer web is investigated.

3.2 Consequences of Damaged Coupling Joints

Fig. 8 shows the shear forces in the web for a decreasing prestressing force in the coupling joint of the outer web. Due to the bond between concrete and prestressing tendons the loss of prestressing refers only to a small area around the coupling join. Fig. 8 shows that close to the end support the shear forces remain constant whereas at the intermediate support the forces decrease; however, the reductions are small. The reason for the reduction is the participation of the slabs in carrying the shear forces in longitudinal direction. Additionally, Fig. 8 shows that a local loss of prestress has almost no influence on the global shear forces. This implies that for the strengthening of a coupling joint a local intervention is sufficient as long as the global load bearing capacity is ensured.

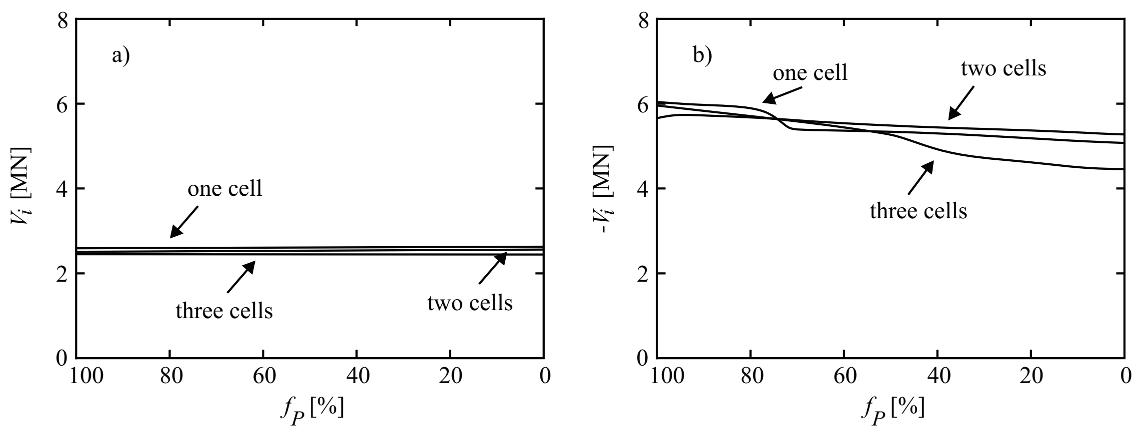

Fig. 8: Shear force at 1,0H from the support as a function of the prestressing-factor (f_P); a) close to the end support, b) close to the intermediate support

By looking at the stresses in the mild steel reinforcement at the bottom fibre of the web (Fig. 9 a)) it can be seen that at a level of presstress of approximately 70% in a single-cell cross section and of 55% in a three-cells cross section the mild steel reinforcement yields. Due to the ductility of the steel the bridge does not collapse, if a coupling joint is damaged, however, serviceability can not be guaranteed and the structure might be much more sensitive to fatigue.

Fig 9 b) shows the crack opening in the coupling joint as a function of the reduced prestress. By using this approach a required prestressing level can be defined and an optical measuring system could be installed (Fig. 2); the other way round, a prestressing level might be estimated on the basis of admissible crack widths.

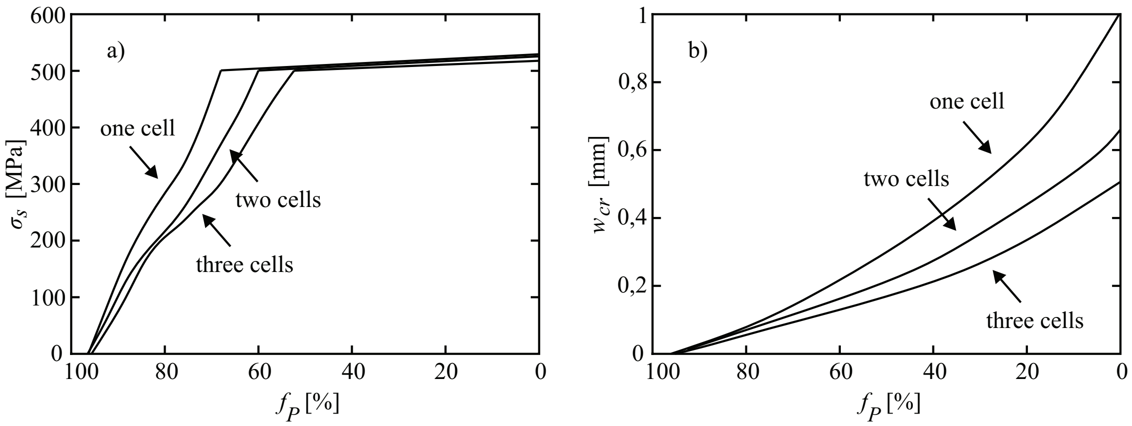

Fig. 9: a) Stresses in the mild steel reinforcement at the bottom fibre of the web in the coupling joint, b) crack widths in the coupling joint.

4. Discussion, Conclusions and Acknowledgements

This paper deals with the computational assessment of prestressed concrete bridges. The calculated shear forces with the beam based method (BBM) are compared to those determined with a 3d finite element model (FEM); on the one hand linear elastic material behaviour and on the other hand nonlinear material behaviour is used. In the first part it is shown, that shear forces calculated with the BBM are smaller than according to the FEM. In the following, FEM calculations with the assumptions of nonlinear and linear elastic material behaviour are compared. It is found that the shear forces from the nonlinear FEM are smaller at the intermediate support; because of the cracking of the webs the slabs contribute in carrying shear forces in longitudinal direction. Close to the end support the shear forces from the nonlinear FEM are bigger; this is due to the fact that loads are transferred more directly to the web as a result of the cracking in the cantilever slab in transverse direction. Finally, the loss of prestress in coupling joints is investigated. It becomes quite clear that with help of detailed calculations it is possible to define intervention criteria.

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