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NONLINEAR FINITE ELEMENT ANALYSIS OF TRANSVERSELY PRESTRESSED CONCRETE DECK SLABS

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This paper describes the modeling and analysis procedure of a 3D, solid, nonlinear finite element (FE) model of a bridge developed in the finite element analysis software package TNO DIANA to study the structural behavior in punching shear of transversely prestressed concrete deck slabs cast between flanges of long, pretensioned girders, and compressive membrane action. The numerical research was part of a broad project involving laboratory experiments carried out on a 1:2 scale model of such a bridge in Delft University of Technology. Both the experimental and numerical results showed much higher capacities than expected and this was attributed to the development of compressive membrane action in the plane of the slab. The numerical results were then compared with the experimentally found ultimate loads of eight basic test cases and it was discovered that the nonlinear FE models can predict the load carrying capacity quite accurately with a coefficient of variation of only 11%. It was concluded that punching shear failures can be reasonably modeled with non-linear finite element analysis of 3D solid models. Furthermore, using composed elements can lead to the determination of compressive membrane forces developed in a laterally restrained slab, which was previously difficult to determine using analytical techniques.

Keywords: Bridge, Bearing capacity, Punching shear, Compressive membrane action, Numerical modeling, DIANA, Composed elements.

1 INTRODUCTION

1.1 Background

Safety of existing structures is a pressing matter being investigated by designers all over the world. A lot of research has been going on in the Netherlands on this subject as there are a large number of bridges built back in the 60s and 70s of the last century. One particular type of bridge, whose safety was questionable, consists of a thin, transversely prestressed bridge deck slab cast between flanges of long, pretensioned girders. There are 69 bridges in the Netherlands of this type and it was found out that the shear capacity of the deck slabs as prescribed by the codes is more conservative in the recently implemented EN 1992-1-1:2005 (CEN 2005) than in the former Dutch NEN 6720:1995 (1995). Also, modern traffic loads are higher than assumed in the original design. Therefore, it was decided by Rijkswaterstaat (Ministry of Infrastructure and the Environment, Netherlands) to carry out a comprehensive research program to investigate the bearing capacity of these bridge decks. The first part of the research involved nineteen experiments being carried out in the Stevin II laboratory, Faculty of Civil Engineering and Geosciences, Delft University of Technology on the scaled model of such a bridge. A 1:2 scaled model of the bridge was constructed consisting of a thin transversely prestressed concrete deck slab cast between precast concrete girders. The slab panels were subjected to concentrated loads simulating Eurocode Load Model 1 (CEN 2005) and the bearing (punching shear) capacity was observed. This paper describes the numerical research that followed the experimental program to investigate the bearing capacity of the model bridge deck. A comparison of the nonlinear finite element analysis and the experimental analysis of eight typical test cases is made in this paper.

1.2 Compressive Membrane Action

The conventional methods of bridge design were based on conservative flexural theories but it has been discovered that under concentrated wheel loads, the laterally restrained deck slabs mostly fail in punching shear rather than in flexure (Batchelor 1990, Fang *et al.* 1994). The reason behind this is the development of compressive membrane forces in the deck slab. When a load is applied on a laterally restrained slab, its edges tend to move outside and the restraint of the boundary elements produce a compressive membrane force in the plane of the slab enhancing the bearing capacity in both flexure and punching shear. This phenomenon is called compressive membrane action (CMA).

Currently, codes like Eurocode 2 (CEN 2005) and ACI 318 (2005) do not consider CMA in their capacity formulae. However, there are some codes that do consider CMA like CSA: CHBDC (2005), the Transit New Zealand (2003) code and UK HA, BD81/02 (2002), but these are only applied for reinforced concrete slabs. It is hypothesized, in this research, that the in-plane forces arising from the combined action of prestressing and membrane forces will increase the bearing capacity to a large extent allowing thinner deck slabs to be applied with no problems of serviceability and structural safety.

2 EXPERIMENTAL PROGRAM

The 1:2 scale model bridge deck consisted of three transversely posttensioned deck slab panels cast in-situ between precast, pretensioned girder flanges. The interface between the deck slab and the girder flanges was indented to generate sufficient shear capacity and had an inclination of 1:20. The transverse prestressing bars (15 mm ϕ bars in 400 mm c/c ducts) in the deck slab were unbonded so that the prestressing level could be varied during the experiments. Two transverse cross-beams were also provided and were posttensioned to the same level as the deck slab. The test setup is shown in Figure 1.

For the deck slab and the transverse cross-beams, the concrete compressive cylinder strength was 65 MPa, the tensile strength was 5.41 MPa and the modulus of elasticity was calculated as 39 GPa (as per Eurocode 2). For the girders, the concrete compressive cylinder strength was 75 MPa, the tensile strength was 6.30 MPa and the modulus of elasticity was 41 GPa. The steel reinforcement had a yield strength of 525 MPa and the prestressing steel bars had a characteristic tensile strength of 1100 MPa.

Figure 2 shows the model of the bridge deck in the laboratory and the test loading positions in the plan view of the deck slab. In all the tests, a concentrated load (wheel print load) was applied through a 200×200 mm, 8 mm thick rubber bonded to two 200×200×20 mm steel plates. The concentrated load was according to Eurocode 1 Load model 1, NEN-EN 1991-2:2003 (CEN 2005) scaled down according to 1:2.

Two levels of transverse prestressing were mainly investigated: 1.25 MPa and 2.5 MPa. Four types of tests were performed: a) Single point load acting at mid span of deck slab panel (P1M);

b) Single point load acting close to the girder flange-deck slab interface/joint (P1J); c) Double point loads at 600 mm c/c acting at mid span of deck slab panel (P2M); d) Double point loads at 600 mm c/c acting close to the girder flange-deck slab interface/joint (P2J). In the tests performed close to the girder flange-deck slab interface, load was placed at 200 mm c/c from the joint except in two tests, BB3 and BB4 where it was placed at 110 mm c/c. For details of the real bridge, the prototype and the experimental setup, reference is made to Amir (2014) and Amir *et al*. (2016).

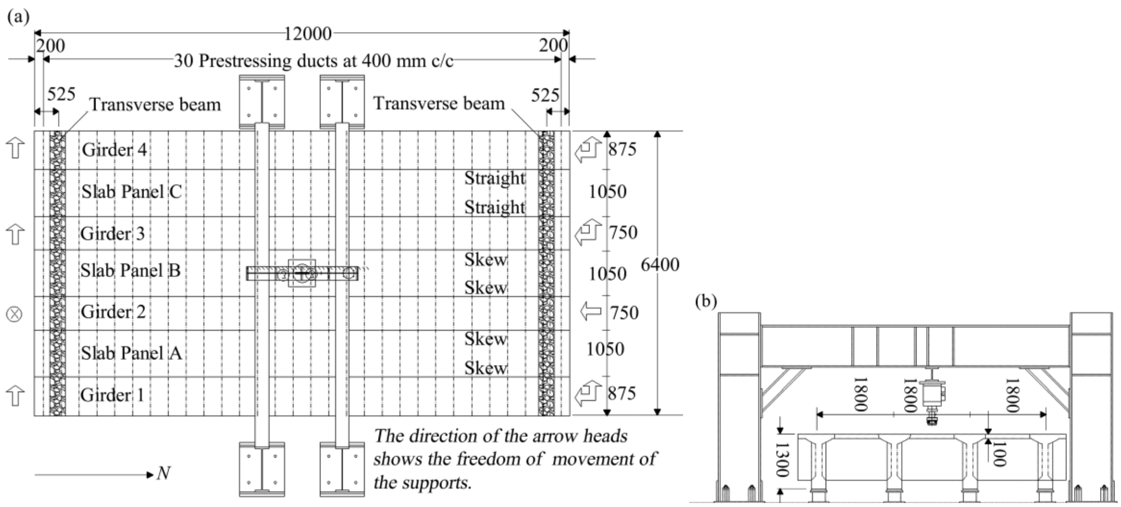

Figure 1. Test setup: a) Plan view b) Side view (transverse direction). All dimensions are in mm.

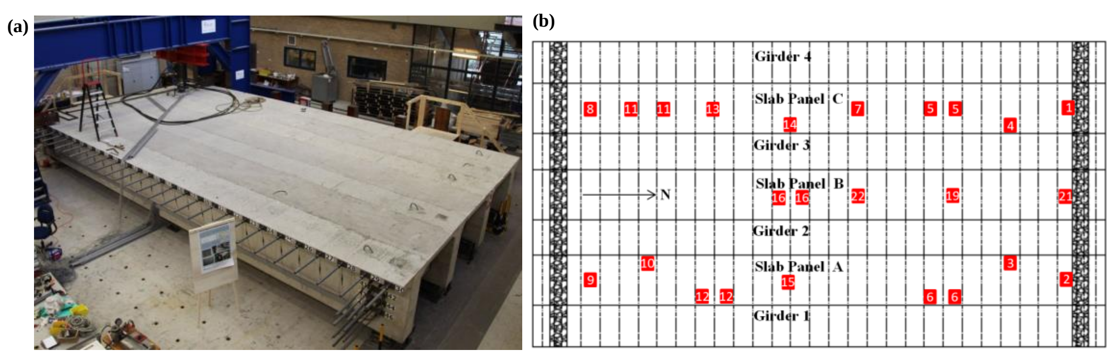

Figure 2. a) Bridge model; b) Deck slab test positions (BB1-BB22). Duct positions are also labeled.

3 NUMERICAL ANALYSIS

For the numerical analysis, a 3D solid finite element model of the prototype bridge deck (Figure 3) was made in the FEA software package DIANA (FX+ 9.4.4) (2012). The model consisted of 3D solid elements (CHX60 and CTP45) with a fine mesh around the loading area and a course mesh away from the loading to reduce the time for computation. A layer of composed elements (CQ8CM) was provided in the fine mesh area to calculate compressive membrane forces. Ducts at 400 mm c/c were provided only in the fine mesh area around the loading. Prestressing pressure was applied according to the required level of transverse prestressing in the deck slab and the transverse cross-beams.

For most cases the deck slab was analyzed nonlinearly while the girders and the transverse cross-beams remained in the linear range. The only exceptions to this were the tests BB3 & 4. The flange of the adjoining girder was analyzed as nonlinear since the experimental load was too close to the interface (110 mm c/c) and linearity of the flange would have induced a much higher capacity than in reality. An embedded reinforcement grid based on the actual steel reinforcement ratio was provided in the deck slab panels at the top and bottom in the horizontal as well as the vertical direction.

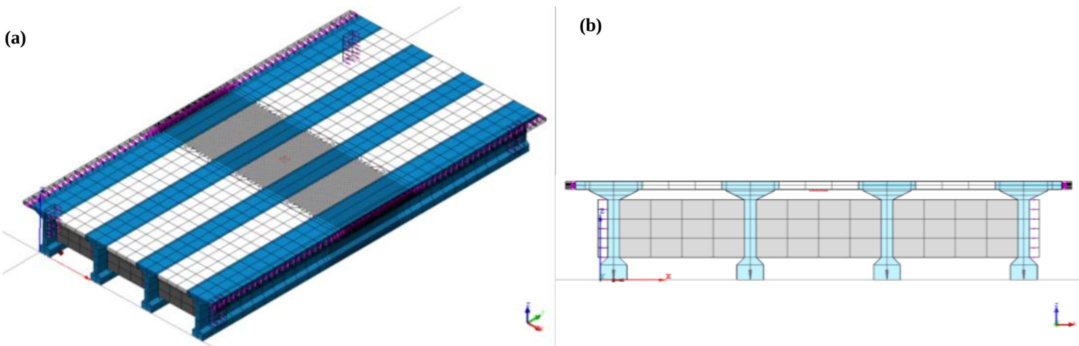

Figure 3. a) 3D solid finite element model developed in DIANA (2012); b) Cross-section of the model.

3.1 Material Models and Additional FEA Nonlinear Material Properties

For the nonlinear analysis of the deck slab, a smeared cracking "Total strain crack rotating model" was selected. An elastic-perfectly plastic model, CONSTA, was used for the concrete behavior in compression, whereas, an exponential softening curve, HORDIJK, (Hordijk 1991) was used for the concrete behavior in tension. A fracture energy (G_f) of 0.15 N/mm was assumed for the deck slab concrete (for a maximum aggregate size of 20 mm, MC90 (1990) gives a value of 0.135 N/mm for the fracture energy, whereas MC2010 (2012) gives a value of 0.21 N/mm). The Poisson ratio, ν, for all the concrete components, was taken as 0.2. For the embedded grid reinforcement, the von Mises plasticity criterion was used with a Poisson ratio of 0.3.

3.2 Iteration Method and Convergence Criteria

Both physical and geometrical nonlinearities were applied to the system. Composed elements were generated while giving the analysis commands. An incremental-iterative procedure was used for the nonlinear analysis and modified Newton Raphson method was used for the solution. The prestressing load was applied to the bridge deck in a single step. After that a displacement-controlled load was applied with a step size of 0.1 mm unless the solution diverged, in which case the displacement increment was reduced to 0.05 mm. Since the applied load was displacement-controlled, the default force and energy-based convergence criterion was employed.

4 COMPARISON OF EXPERIMENTAL AND NUMERICAL RESULTS

Results for ultimate load and failure mode of the typical test cases are described in Table 1. It was observed that for all test cases, failure always occurred by punching shear. Flexural punching (large rotations but final failure was in punching shear) was observed for double loads applied in the midspan which is consistent with findings of Zheng *et al.* (2010), and brittle punching was observed for the rest. It was also observed that the ultimate bearing capacity increased with the increasing prestressing level. Both the experimental and numerical results show comparable ultimate loads and similar failure modes, see Table 1.

Table 1. Comparison of finite element analyses and experimental ultimate loads.

Test BB.	TPL (MPa)	Designation	P_T (kN)	P_{FEA} (mm)	Test and FEA Failure Mode	P_T/P_{FEA}
1.	2.5	C-P1M	348.7	302.3	Brittle punching	1.15
2.	2.5	A-P1M	321.4	302.3	Brittle punching	1.06
3.	2.5	A-P1J	441.6	429.9	Brittle punching	1.03
4.	2.5	C-P1J	472.3	429.9	Brittle punching	1.10
5.	2.5	C-P2M	490.4	529.9	Flexural punching	0.93
6.	2.5	A-P2J	576.8	537.0	Brittle punching	1.07
7.	2.5	C-P1M	345.9	302.3	Brittle punching	1.14
8.	1.25	C-P1M	284.5	271.4	Brittle punching	1.05
9	1.25	A-P1M	258.2	271.4	Brittle punching	0.95
10.	1.25	A-P1J	340.3	300.7	Brittle punching	1.13
11.	1.25	C-P2M	377.9	453.4	Flexural punching	0.83
12.	1.25	A-P2J	373.7	454.9	Brittle punching	0.82
					Mean	1.02
					Standard deviation	0.11
					Coefficient of variation	0.11

Note: TPL: Transverse prestressing level, P_T: Test ultimate load, P_{FEA}: Finite element analysis ultimate load.

5 RELATIONSHIP BETWEEN THE IN-PLANE FORCES AND FAILURE LOADS

It is evident from the experimental and finite element analysis results that sufficient membrane action had developed in the plane of the deck slab and combined with the transverse prestressing, it positively enhanced the ultimate bearing capacity of the deck slab. It was observed both experimentally and numerically that for each type of the load, the deck slab showed horizontal or lateral displacements only after the initial cracking (Amir 2014) which correlates well with the findings of Liebenberg (1966), He (1992) and Fang (1985).

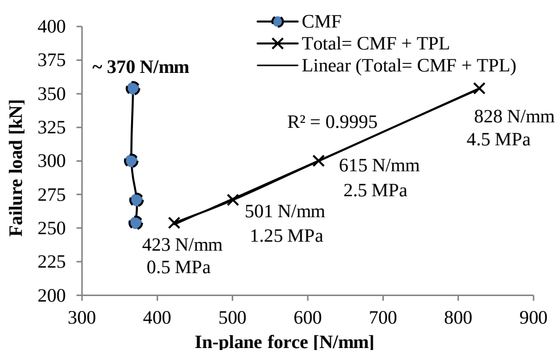

Figure 4. Relationship between the failure load and the in-plane forces for various TPLs.

Figure 4 shows the relationship between the distributed in-plane force developed (N_{xx} obtained from composed elements in DIANA (2012)) and the failure load for various levels of transverse prestressing level (0.5, 1.25, 2.5 and 4.5 MPa). It can be observed that for all cases, the overall in-plane force increases with the increasing prestress force and the relationship is almost linear. However, subtracting the initial prestressing from the overall in-plane force corresponding to that particular TPL gives a constant value of the compressive membrane force (CMF ~ 370 N/mm) for the deck slab showing that CMA is independent of the transverse prestress level. This would mean that for a particular deck slab having a certain lateral stiffness, the membrane action developed remains constant if all other parameters remain the same.

6 SUMMARY AND CONCLUSIONS

A 3D, solid, 1:2 scaled model of a real bridge was developed in DIANA (2012) and non-linear analyses were performed to simulate the experiments done in the laboratory on the same prototype. A basic analysis comprising of eight test cases was performed to study the ultimate bearing capacity of the bridge deck and compressive membrane action. It was observed that substantial CMA develops in the deck slab and transverse prestressing affects the bearing capacity positively. Furthermore, failure always occurred by punching in the deck slab span, regardless of the position of the load. The interface proved to have sufficient strength and was never governing. It can be concluded that punching shear failures can be reasonably modeled with nonlinear finite element analysis of 3D solid models. Sufficient cost savings can be made if numerical studies are performed over expensive experiments, but it is recommended that the results of applied computer models are first calibrated with experimental results.

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