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EXPERIMENTAL AND NUMERICAL INVESTIGATION OF THE EFFECT OF SIZE IN POST-TENSIONED CONCRETE DECK SLABS

SANA AMIR[1], COR VAN DER VEEN[2], and ANE DE BOER[2]

[1]*Faculty of Engineering and Information Sciences, University of Wollongong in Dubai, Dubai, United Arab Emirates*

[2]*Dept Design and Construction, Concrete Section, Faculty of Civil Engineering and Geosciences, Delft University of Technology, Delft, Netherlands*

It is widely known that as the structure of the size increases, its nominal strength decreases. In this paper, the effect of size on punching shear has been quantified for transversely post-tensioned deck slabs cast between flanges of precast concrete girders. A 1:2 scaled model of the bridge was constructed in the laboratory, and experimental and numerical analyses were carried out. However, in order to apply these results on a real bridge, simply using the geometrical scale factors is not sufficient and a structural size effect has to be taken into account. Since a full-scale experimental study was not possible due to the costs involved, a numerical approach using finite element analysis software package TNO DIANA was used to model both the prototype and the real bridge, and a comparison was made to estimate the effect of size on the bearing capacity. It was found that increasing the transverse prestressing level had a positive effect on the punching shear strength of the deck slab. Furthermore, a lower size effect was observed with higher transverse prestressing levels. It is concluded that if a suitable size factor is used, either numerical or small-scale experimental studies can be reasonably used to investigate existing structures.

Keywords: Bearing capacity, Punching shear, Scale factor, Transverse prestressing level, Numerical modeling, TNO DIANA.

1 BACKGROUND

A common research problem in the structural engineering world is to investigate the safety and capacity of old structures. One such investigation was done in the Netherlands on a particular type of bridge that consisted of transversely prestressed bridge deck slab. The striking feature of this type of bridge is that the deck slab is only 200 mm thick, is post-tensioned in the transverse direction, and cast between the flanges of the girders. Investigation by the Dutch Ministry of Infrastructure and the Environment, Netherlands (Rijkswaterstaat) showed that punching shear capacity of the deck slabs when calculated by EN 1992-1-1 (2005) did not meet the safety criteria. It is also understandable that the old design was based on traffic loads of that time, and modern traffic loads are much higher. Therefore, the ministry initiated a comprehensive research program in collaboration with Delft University of Technology to investigate the bearing (punching shear) capacity of these bridge decks.

The research was done in three stages: experimental, numerical and theoretical (refer to Amir 2014 for details). A 1:2 scaled model of the bridge (Figure 1a) was constructed in the laboratory.

Several tests were done with concentrated loads being applied on the deck slab at different locations (Figure 1b). The loads were according to the Eurocode Load Model 1 (EN 1991-2 2002). For the finite element analysis, the software TNO DIANA (FX+ 9.4.4) was employed to model the same bridge. The model was made in 3D and nonlinear analyses simulating the tests were performed. A comparison with the experimental results showed satisfactory results and validated the finite element model. The next step was to investigate the bearing (punching shear) capacity of the real bridge which is not a simple matter of projecting the model bridge results using the 1:2 scale. Bazant and Cao (1987) state that the nominal strength of structural members decreases with an increase in the structural size and is an established phenomenon for shear in beams and slabs. Since it is not economically feasible to carry out a full-scale experimental analysis, a finite element model of the real bridge was developed and analyzed. The results of the model bridge and the real bridge analyses were used to calculate the size factor. This size factor was then used to project the experimental model bridge ultimate capacity to the ultimate capacity of the real bridge (Amir 2014).

2 EXPERIMENTAL PROGRAM

2.1 Scale Factors and Size Effect

A 1:2 scale was used to build the prototype of the bridge (Figure 1a) given the economic and laboratory space constraints. The linear scale factors were geometry-based, and stress in the real bridge and prototype was considered to be the same. Previously, Savides (1989), He (1992) and Marshe (1999) have used this approach for modeling prestressed bridges. However, these scale factors ignore the size effect and assume that the nominal strength remains the same regardless of the structural size. In reality, the ultimate capacity calculated for the model bridge deck slab with 100 mm thickness cannot be simply projected to ultimate capacity of the real bridge with a deck slab thickness of 200 mm by only using the scale factors. Therefore, it is important to establish a size factor to use with the scale factors to calculate the bearing loads for the real bridge.

2.2 The Model Bridge

The model bridge is briefly described in this section. Refer to Amir (2014) for details of the construction and the test setup.

The scaled model of the bridge consisted of three main components: Four precast, pretensioned girders, three transversely post-tensioned deck slab panels, and post-tensioned transverse crossbeams close to the bridge deck ends. Unbonded prestressing bars (15 mm ϕ bars in 400 mm c/c ducts) were provided in the deck slab so that the prestressing level could be adjusted for the experiments. The deck slab and the transverse beams were prestressed to the same level for each test. Refer to Figure 2 for the laboratory experimental setup. The material properties are given in Table 1.

Nineteen tests were carried out with three levels of transverse prestressing: 0.5 MPa (representing a reinforced concrete deck with very little prestressing equivalent to regular reinforcement ratio), 1.25 MPa (some tendons in real bridge may be damaged or may have concurred losses), and 2.5 MPa (the actual prestressing level in the real bridge). The concentrated wheel print load (200×200 mm) was according to Eurocode 1 Load model 1, EN 1991-2 (2002) and was scaled down according to 1:2. Some parameters, for example, size of the loading plate, were varied in the tests. However, since this paper is focused on size effect, only the relevant test results are presented.

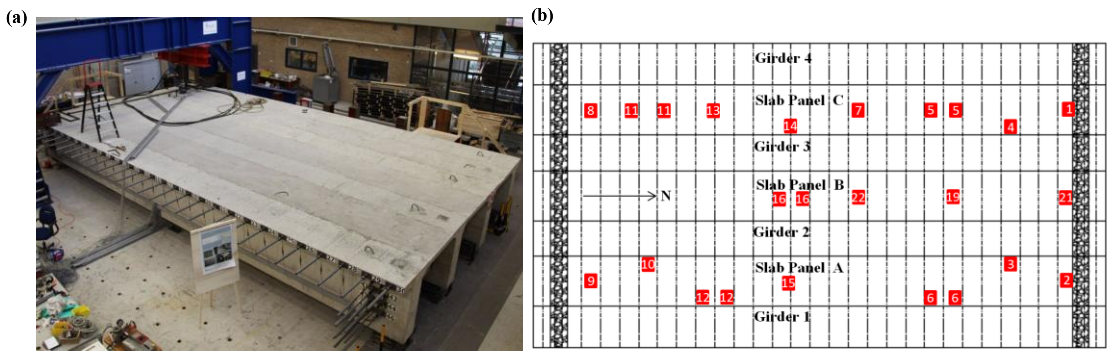

Figure 1. a) Model of the bridge in the laboratory; b) Location of the tests. Prestressing ducts are shown by dashed lines.

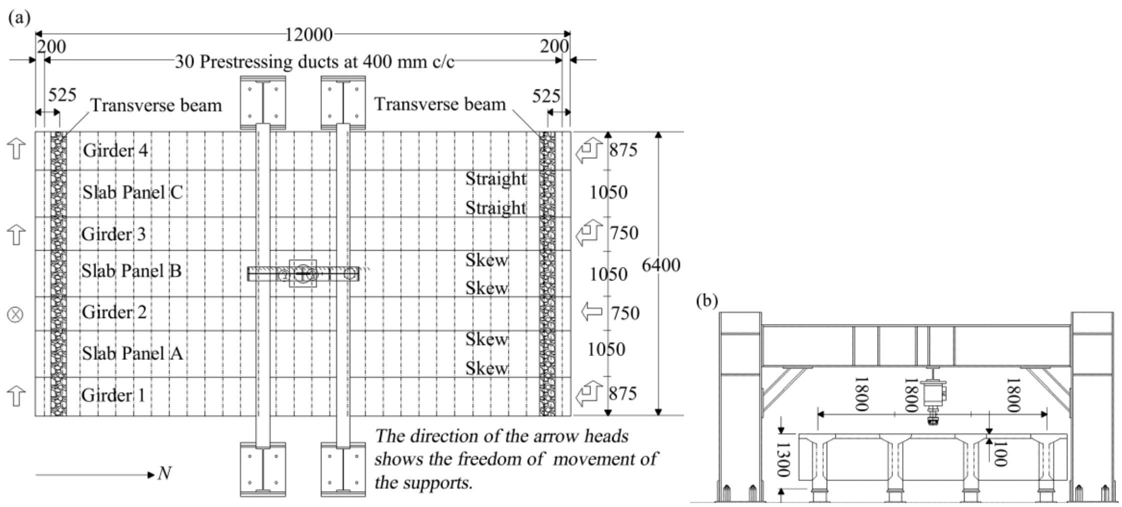

Figure 2. Experimental setup: a) Top view b) Transverse view. Shown dimensions are in mm.

The tests performed were of four different types depending on the number of loads applied at a time and the position of the load across the width of the slab panel: a) A concentrated load applied in the middle of the slab panel; b) A concentrated load applied close to the girder flange-deck slab joint; c) Two concentrated loads at a longitudinal spacing of 600 mm c/c applied in the middle of the slab panel; d) Two concentrated loads at a longitudinal spacing of 600 mm c/c applied close to the girder flange-deck slab joint.

3 NUMERICAL ANALYSIS

The numerical analyses were performed by using the software package TNO DIANA (FX+ 9.4.4). The model (Figure 3) was developed using solid CHX60 and CTP45 type elements and had a varying mesh size throughout the structure in order to reduce the computing time. Hollow round cylindrical spaces were provided only around the loading area. For simplicity, an external pressure on the sides of the bridge deck and the transverse beams was applied to simulate the prestressing effect. The level was varied depending upon the test being simulated. Regular steel reinforcement in the deck slab panels was modeled as an embedded grid. The model further had two types: 25 mm Φ ducts to scale down the 50 mm Φ ducts of the real bridge and 45 mm Φ ducts for comparison with model bridge experimental results. Refer to Amir (2014) for details of the numerical modeling and the procedure of the analyses.

Table 1. Material properties and material models used in the study.

Material	**Properties**	**Deck Slab**	**Transverse Beams**	**Girders**	**Units**
Concrete	Mean compressive cylinder strength	65	65	75	MPa
	Mean tensile strength	5.41	5.41	6.3	MPa
	Modulus of elasticity	39	39	40.26	GPa
Prestressing steel	Characteristic tensile strength	1100	1100	1100	MPa
	Characteristic 0.1% proof stress	900	900	900	MPa
	Modulus of elasticity	205	205	205	GPa
Ordinary steel	Mean yield strength	525	525	525	MPa
	Mean ultimate tensile strength	580	580	580	MPa
	Modulus of elasticity	200	200	200	GPa
FEA nonlinear properties for the deck slab					
Material	**Properties**	**Model**	**Value**		**Units**
Concrete	Compression	CONSTA			
	Tension	HORDIJK			
	Fracture energy		0.15		N/mm
	Poisson ratio		0.2		
Steel	Plasticity	Von Mises			
	Poisson ratio		0.3		

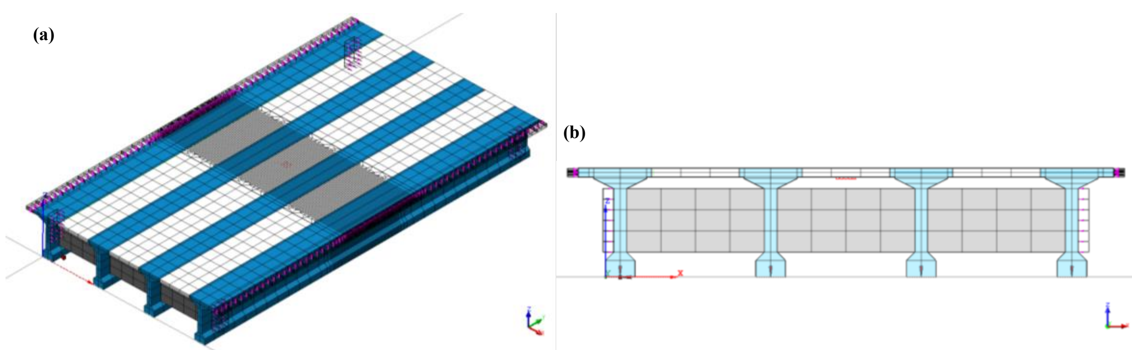

Figure 3. a) 3D finite element model simulating the lab prototype; b) Transverse section of the model.

3.1 FEA Nonlinear Material Properties and Material Models

The additional material properties needed for the nonlinear analyses of the model bridge deck slab are given in Table 1, section 2.2.

3.2 Iteration Method and Convergence Criteria

An incremental-iterative nonlinear analysis procedure considering both physical and geometrical nonlinearities were applied with modified Newton Raphson method used for the solution. First, the prestressing pressure was applied. Next, a 0.1 mm (or 0.05 mm) stepped, displacement-controlled load was applied on selected elements equal to the experimental load plate size (200×200mm). Convergence was obtained through a force and energy-based criterion.

4 COMPARISON OF EXPERIMENTAL AND NUMERICAL RESULTS

A comparison between the experimental and numerical failure loads shows a mean ratio of 1.02 and a standard deviation of 11% (Amir 2014). Higher punching shear capacities were observed for 2.5 MPa transverse prestressing cases as compared to 1.25 MPa. Failure in all cases was by brittle or flexural punching. Brittle punching is characterized by small deflections and flexural punching by large deflections with the final failure occurring by formation of a classic punching cone.

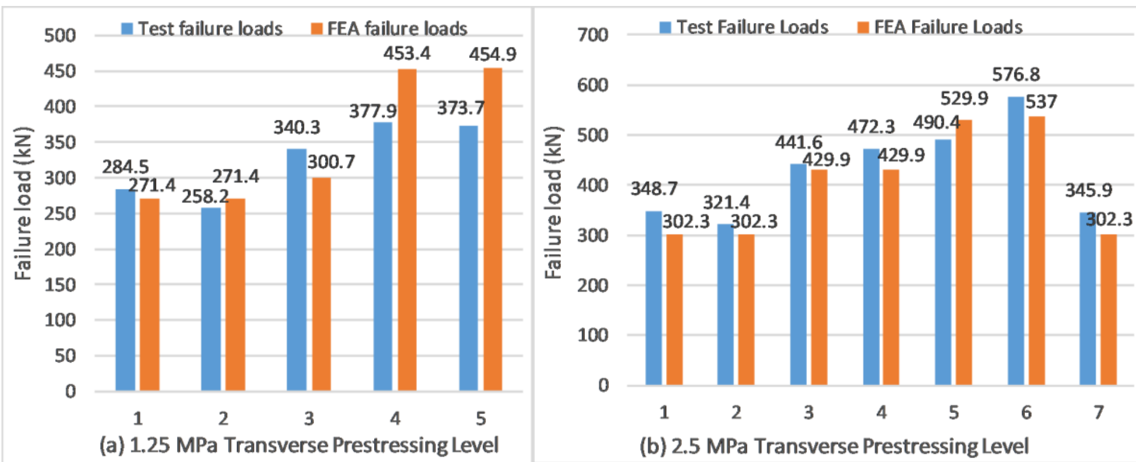

Figure 4. Comparison of the experimental and the finite element analyses failure loads.

5 SIZE EFFECT

In order to study the size effect, a 3D model of the actual bridge was constructed in DIANA (2012) (Figure 5). Refer to Amir (2014) for details of the modeling. A brief summary is provided below.

The real bridge model was made using the same modeling approach and parameters as the 1:2 scaled model bridge with the only difference being the size of the structural components. The girders were 3000 mm high with a web thickness of 200 mm. The width of the top flange was 1500 mm and that of the bottom flange was 580 mm. The deck slab panels were 2100 mm wide with a length of 12000 mm. The thickness of deck slab was 200 mm. The transverse beams were 400 mm wide and were present close to the supports. Four hollow ducts of 50 mm diameter were modeled transversely at a spacing of 800 mm c/c. A smeared embedded grid was modeled for the regular reinforcement. The applied load was displacement controlled and spread over a wheel print area of 400×400 mm according to Load Model 1 in EN 1991-2 (2002). Owing to a larger structural size and, therefore, a larger maximum aggregate size than the model bridge, a fracture energy of 0.175 N/mm was used in the analyses of the real bridge. An average size factor of 1.2 was obtained when the finite element bridge failure loads were projected using the scale factor and compared with the real bridge failure loads as shown in Table 2.

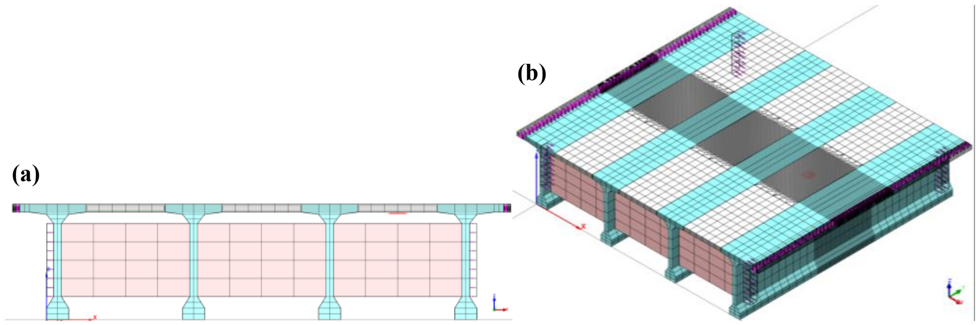

Figure 5. The FE real bridge model for P1M load case: a) Cross-section; b) 3D view.

Table 2. Calculation of the size factor.

TPL (MPa)	$P_{FEA,PR}$ (kN)		$P_{FEA,RB}$ (kN)	Size factor		Average size factor
	25 mm ducts	45 mm ducts	50 mm ducts	25 mm ducts	45 mm ducts	
				$P_{FEA,PR}/P_{FEA,RB}$	$P_{pr,FEA}/P_{FEA,RB}$	
0.5	1104	1016	678.3	1.63	1.5	
1.25	1168	1086	957.5	1.22	1.13	1.2
2.5	1332	1209	1228.8	1.08	0.98	

Note: $P_{FEA,PR}$ = Projected FEA ultimate load using the 1:2 model bridge results, $P_{FEA,RB}$ = FEA ultimate load for the real bridge. Length scale factor, $x = 2$. Force scale factor $x^2 = 2^2$. The result of the analyses of the P1M load applied on the exterior panel was used for the calculation, giving the maximum size effect.

6 CONCLUSIONS

It can be concluded that bearing or punching shear strength increases with an increasing transverse prestressing level and nonlinear fine element analyses can predict the failure loads quite reasonably. It is also noted that the effect of size reduces with the increasing transverse prestressing level. Furthermore, if size factors can be reasonably obtained from calibrated numerical models, small scale experimental studies can be carried out to investigate large scale problems with sufficient savings in cost.

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