Punching Capacity of a Reinforced Concrete Bridge Deck Slab Loaded to Failure

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ABSTRACT

Full-scale failure tests of a 55 year old prestressed concrete girder bridge have been carried out to calibrate models for assessment of existing bridges. This paper summarise the outcome from the punching test and analytical analysis according to the model stated in the Eurocode. The experimental load was approximately 2.4 times the code value using measured material properties.

Key words: Assessment, Bridge Deck Slab, Full-Scale Testing, Measurements, Punching, Reinforced Concrete

1. INTRODUCTION

In models for assessment of existing structures the real behaviour should be reflected in order to achieve a reliable estimation of the residual capacity. Accordingly, understanding the behaviour is crucial to accomplish an optimised bridge stock management. Due to assessment models, mainly developed based on laboratory studies of elements with simplified conditions, it is necessary to perform supplementary large- or full-scale experiments to calibrate and if necessary improve the models. The aim of the project, partly presented in this paper, is to acquire relevant data from a full-scale bridge test for such calibration. A particular focus is on existing punching resistance models for bridge deck slabs (Sundquist, 2005, CEN, 2005).

2. EXPERIMENTAL INVESTIGATION

The Kiruna Bridge, Sweden, constructed in 1959 has been subjected to an experimental investigation in 2014, including failure loading of both the main girders and the deck slab (Bagge et al., 2014). The road bridge was a 55 year old prestressed concrete girder bridge continuous in five spans with the total length of 121.5 m (tested span 20.5 m). The superstructure consisted of three longitudinal girders connected by the deck slab and cross beams (Figure 1). In the location of the punching test the slab was reinforced by steel bars (diameter 16 and 10 mm) with the average yield strength of 584 MPa (CoV = 0.022) and 667 MPa (CoV = 0.050) in transverse and longitudinal direction, respectively. The average value of the tested in-situ concrete compressive strength was 62.2 MPa (CoV = 0.16).

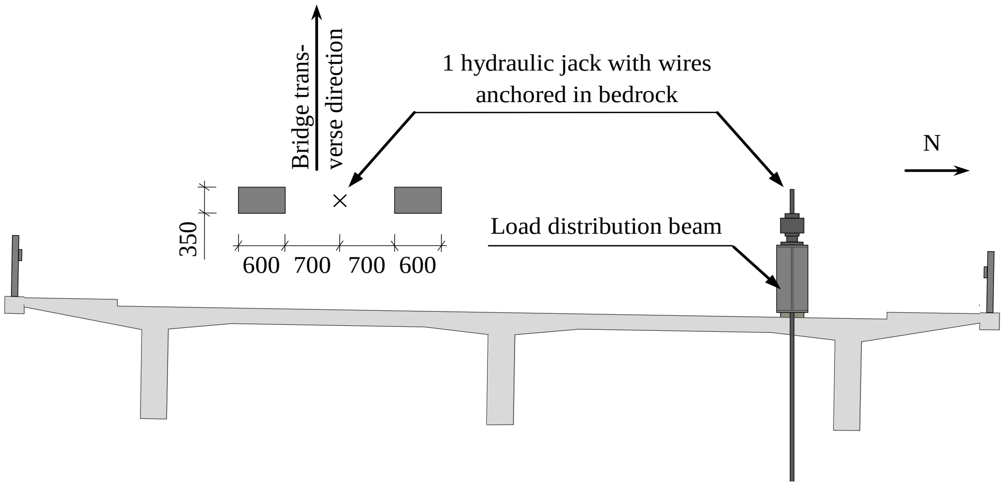

Figure 1: Arrangement for loading the bridge deck slab.

The bridge deck slab was loaded to failure with the setup in Figure 1. A force controlled hydraulic jack located in the midspan was used to apply load, through a load distribution beams and two steel plates, to the upper surface of the concrete slab. The load plates were 350x600 mm^2 spaced 2.0 m apart and located 470 and 330 mm in relation to the inner side of the northern main girder (880 mm to the outer side). Thus, the test setup corresponded to load model 2 in the Eurocode 1 (CEN, 2003).

The girders and the slab were monitored during the test. Applied force was derived from the oil pressure in the hydraulic jack. Draw-wire sensors were instrumented in the midspan of the northern (D1) and central (D2) girder in order to measure deflections. Moreover, deflections of the slab were measured underneath each loading plate (D3 and D4) and at the corresponding position of the northern girder (D5 and D6), i.e. 1.0 m to the midspan. At 500 and 1000 mm south of the centre line through the loading points the curvature was measured along the longitudinal bridge direction. Each curvature rig was composed by simply supported steel beams (length 4.82 and 5.08 mm) and five linear displacement sensors with 800 mm spacing. The midpoints of the curvature rigs coincided with the midspan. The instrumentation is described more in detail in (Bagge et al., 2014).

3. ANALYTICAL ANALYSIS

The load-carrying capacity of the bridge deck slab was analysed for the actual test setup, using the punching resistance design model stated in the Eurocode 2 (CEN, 2005). The capacity according to the model is represented by the shear force resistance for a specified control perimeter, proposed to be taken as the shortest length 2 times the cross-section effective depth

(2*d*) from the loaded area. Permitted shear stress, $v_{Rd,c}$, is calculated according to the empirical expression given by Equation (1) The load-carrying capacity was calculated for two cases: (a) design punching resistance considering the concrete partial safety factor and the in-situ concrete characteristic strength and (b) punching resistance excluding partial safety factors and replacing the characteristic in-situ concrete strength by the corresponding average value.

$$v_{Rd,c} = \max\left(C_{Rd,c} k \sqrt[3]{100 \rho_l f_{ck}}, v_{\min}\right) \quad (1)$$

where $C_{Rd,c}$ = 0.18/γ_C is a constant from experimental calibration including specified partial safety factor, γ_C = 1.5 is a concrete partial safety factor, k = 1.95 is a factor to account for size effects, ρ_l = 0.19 % is the reinforcement ratio, f_{ck} = 47.0 MPa is the characteristic value of concrete compressive strength and v_{min} = 0.65 MPa for case (a) and v_{min} = 0.75 MPa for case (b) is the minimum shear strength. The analytical punching resistance for one loading plate was for case (a) 680 kN and for case (b) 840 kN, with the control perimeter of 4.69 m.

4. RESULTS AND DISCUSSION

The bridge deck slab was loaded with an approximate loading rate of 80 kN/min. A sudden punching failure, without prior notice, occurred at the total applied load of 3.32 MN (1.66 MN for each loading plate). Figure 2 shows a photograph after the test, illustrating the failure under the western loading plate. In Figure 2 also the load-deflection behaviour monitored by the draw-wire sensors is given. An abrupt drop of the load at failure resulted in decreased deflection for all sensors except for D3 underneath the western loading plate where failure occurred.

At the punching test the girders were pre-cracked due to the previous loading. Nevertheless, the first part of the test (up to approximately 500 kN) indicated small deflections of the bridge, after

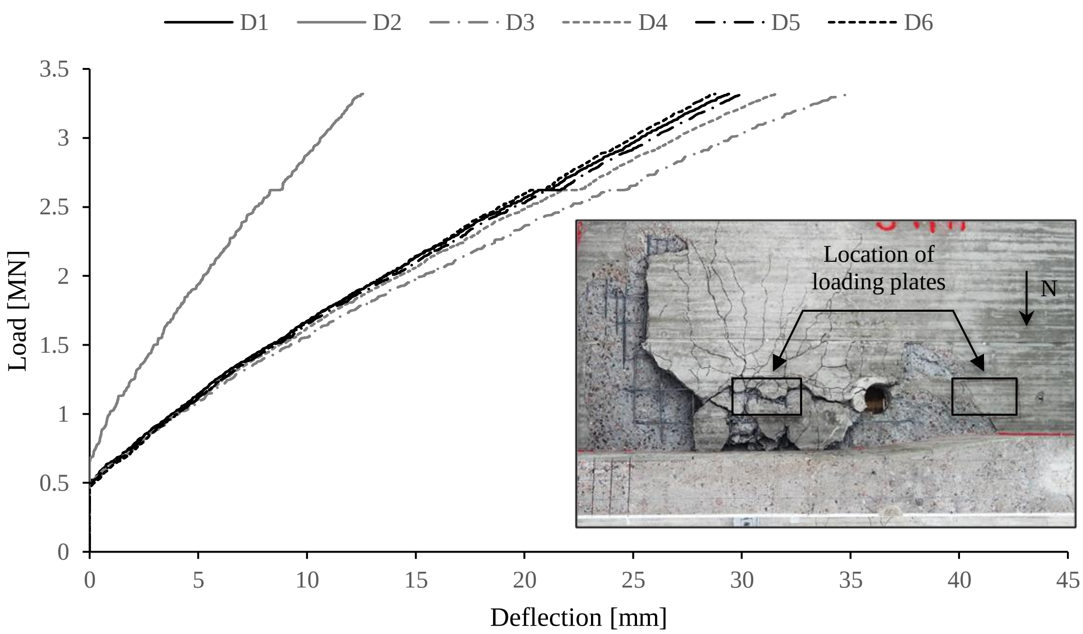

Figure 2. Measured load-deformation curves and photograph of punching failure (2014-07-01).

which they considerably increased. This is due to the prestressing system in the girders. In the test, the differences in deflections between the slab and the northern girder are relatively small (maximum 6.3 mm). Also the central girder deflected significantly. Just before failure the midspan deflection of the central and northern girder was 12.6 and 29.4 mm, respectively. Thus, the longitudinal girders should be regarded as flexible supports to the deck slab.

At the load level of 2.6 MN a new grip was required in order to accommodate deflections and loading cable extension exceeding the stroke length of the jack. Due to increased deformation of the structure at constant load this measure caused in a small shift in the load-deformation curve.

The test demonstrated a load-carrying capacity which was about 2.0 and 2.4 times the outcome from the analytical analysis, based on the punching resistance from average and design values, respectively. It indicates that the code model not fully represents the behaviour of the tested slab. For instance, the influence of flexible supports and membrane action is disregarded in the model.

5. CONCLUDING REMARKS

A full-scale punching test to failure has been carried out of a reinforced concrete bridge in order to acquire relevant data for calibration and development of assessment models. The experimental evaluation resulted in a punching capacity approximately 2.4 times the design resistance according to the Eurocode 2. Thus, the test indicated a load-carrying capacity upgrading possibility if using refined models. In the future research a further comparison between the experimental outcome and punching models is proposed. Moreover, refined evaluation is recommended in accordance to the multi-level assessment approach (Shu, 2015) in which 3D linear and nonlinear finite element analyses are proposed.

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