Structural assessment of bridge deck slabs

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ABSTRACT
The overall aim of this project is to develop improved methods for assessment of the load carrying capacity and response of bridge deck slabs. This research project was carried out by laboratory experiments, analytical computational analysis, linear and non-linear finite element analyses. The on-going non-linear analyses of tested slabs show possibility to accurately predict the load carrying capacity and to realistically simulate the crack pattern and load distribution.

Key words: bridge deck slabs, assessment, modelling, capacity

1. INTRODUCTION
Existing infrastructure represents a substantial part of the societal assets and existing bridges represent a huge capital that need to be well administrated. Bridge deck slabs are one of the most exposed bridge parts and are often critical for the load carrying capacity. Consequently, it is important to examine if the current analysis and design methods are appropriate. In a pre-study, the need for research and development to achieve more robust bridge deck slabs (Sundquist 2011) was identified. The overall aim of this project is to develop improved methods for assessment of the load carrying capacity and response of bridge deck slabs.

2. EXPERIMENTAL STUDY
Initially, a literature survey and laboratory experiments were carried out. Three two-way slabs were tested to failure and loads, deformations, and distribution of support reactions along the supporting edges were measured (Fall et al. 2014). The specimens were two-way octagonal slabs (80 mm in thickness) supported on four edges and subjected to a point-load at the centre, see Fig. 1. Moreover, the loading jack was coupled to a load cell which was placed over a steel plate (280 × 280 × 30 mm). Even load distribution was ensured by placing a wood fibre board (t = 12 mm) between the steel plate and the slab.

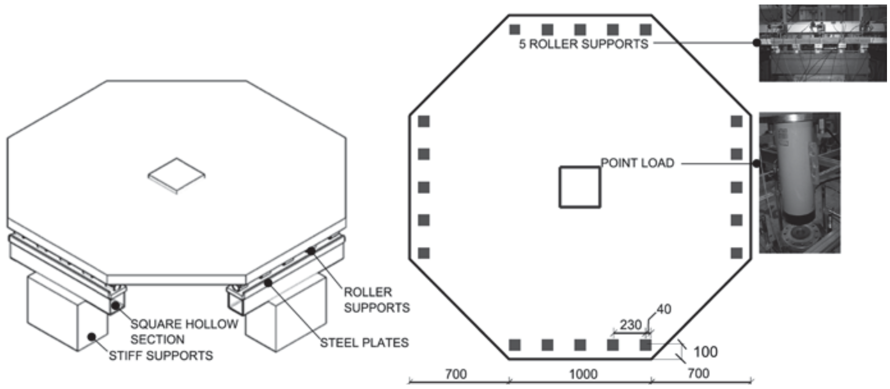

Fig. 1 Test set-up of two-way slabs; all dimensions in mm (Fall et al. 2014).

Regarding the material, the compressive strength (f_c = 50.9 MPa) and tensile strength (f_t = 2.7 MPa) of the concrete, together with the tensile strength of steel reinforcement (f_y = 621MPa, E_s = 210 GPa) were tested. The reinforcement had a bar diameter of 6 mm, placed with a clear cover of 20 mm from the bottom of the slab to the most dense layer. There were totally 25 reinforcement bars with 96 mm spacing in strong direction and 13 bars with 196 mm spacing in weak direction.

3. ANALYTICAL AND FINITE ELEMENT ANALYSIS

Analyses of the slabs on different levels of detail were carried out. First, analytical equations were used to calculate the load carrying capacity of the slabs using yield line method (Johansen 1972). Thereafter linear FE analysis was used to check the capacity according to a FE guideline (Pacoste, Plos, and Johansson 2012).

In order to increase the understanding of the response of the slabs, non-linear FE analysis was used to simulate the behaviour of slabs, and the results were compared to the tests. Parametric studies with non-linear FE analyses were carried out as a basis for further development of existing methods of calculation and design methodology.

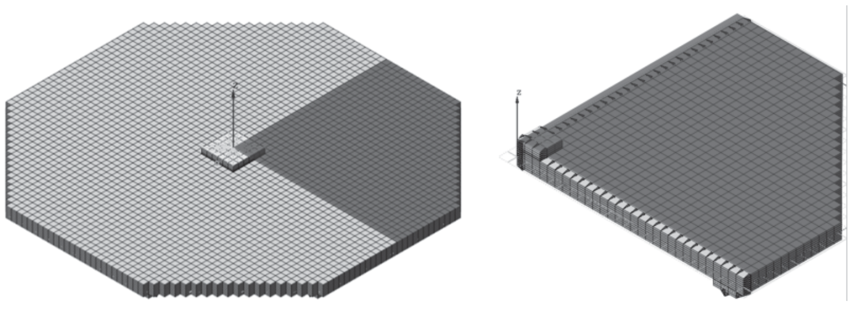

Fig. 2 FE model of the tested slabs

The finite element software DIANA 9.4.4 was used to model the slabs, using a 3D model. Due to symmetry, only a quarter of the slab was included in the model, to reduce the computation time, see Fig. 2. In the test, steel plates and roller bearings were used at the supports. In the FE model, the steel plates were modeled and interface elements were used between the concrete and the steel plates to account for friction. Under the steel plates at the supports, the nodes were supported both in vertical direction and along the roller supports. All nodes at the symmetry faces were fixed in the perpendicular direction. The material properties were taken from material test. Both geometrical and physical nonlinearity were included in the FE analysis.

To investigate the influence of varying modeling choices, several models with different element types, mesh density and ways to model the interaction between concrete and reinforcement were analyzed. The properties of the models are shown in Table 1. An analysis (B40F) with 40×40×10 mm brick elements and full interaction to the reinforcement was selected as reference. In analysis W40F, wedge elements were chosen to investigate the influence of element types. In analyses B30F and B20F, element sizes of 30 and 20 mm in plane were chosen, respectively, to study the influence of mesh density. In analysis B40B, a bond-slip relation was assumed for the interaction between reinforcement and concrete.

Table 1 Five analyses with varying modeling choices

Analysis	Element type	Element size (mm)	Bond model
B40F (reference)	Brick element	40×40×10	Full interaction
W40F	Wedge element	40×40×10	Full interaction
B30F	Brick element	30×30×9	Full interaction
B20F	Brick element	20×20×8	Full interaction
B40B	Brick element	40×40×10	Bond-slip

4. RESULTS AND CONCLUSIONS

The capacity calculated both from yield line method and FE linear analysis were 40.5 kN. Since the three tested specimens had the same dimensions and reinforcement arrangements, the results in the three tests were very similar. Here, the one with intermediate values (CR2) was taken as a reference to be compared with the analysis results, see Fig. 3; as can be seen, the agreement is good with nonlinear analysis but higher than the capacity obtained with yield line and linear FE analysis.

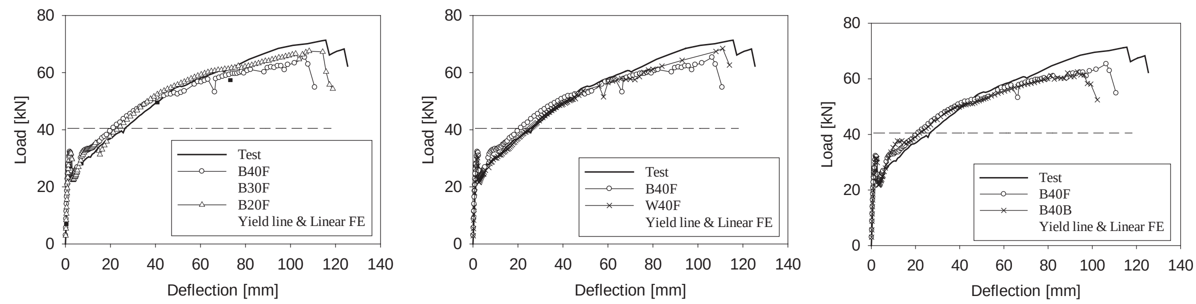

Fig. 3 Load-deflection curve with different mesh densities (left), element types (middle) and bond models (right)

Comparing the models with different modelling choices (Fig. 3), the models in B40F and B30F showed similar results, while the model in analysis B20F had a response closer to test because of the denser mesh. The analysis with wedge elements (W40F) gave better estimation of the load-deflection curve, but it was more difficult to achieve convergence than with brick elements. The analysis with bond-slip interaction (B40B) gave similar results but slightly less capacity and deflection at failure compared to fully bonded reinforcement.

Comparing the crack pattern of the FE analysis and experiment, see Fig. 4, crack localization became more visible for decreasing element size (Column C). Concerning element shape, the cracks tended to propagate along the mesh direction (Column D); as wedge elements gave more freedom in this sense, the crack pattern in the analysis with wedge elements therefore agreed best with the experimental response. The analysis including bond-slip showed more localized cracks, while the models with full interaction showed distributed cracks (Column E).

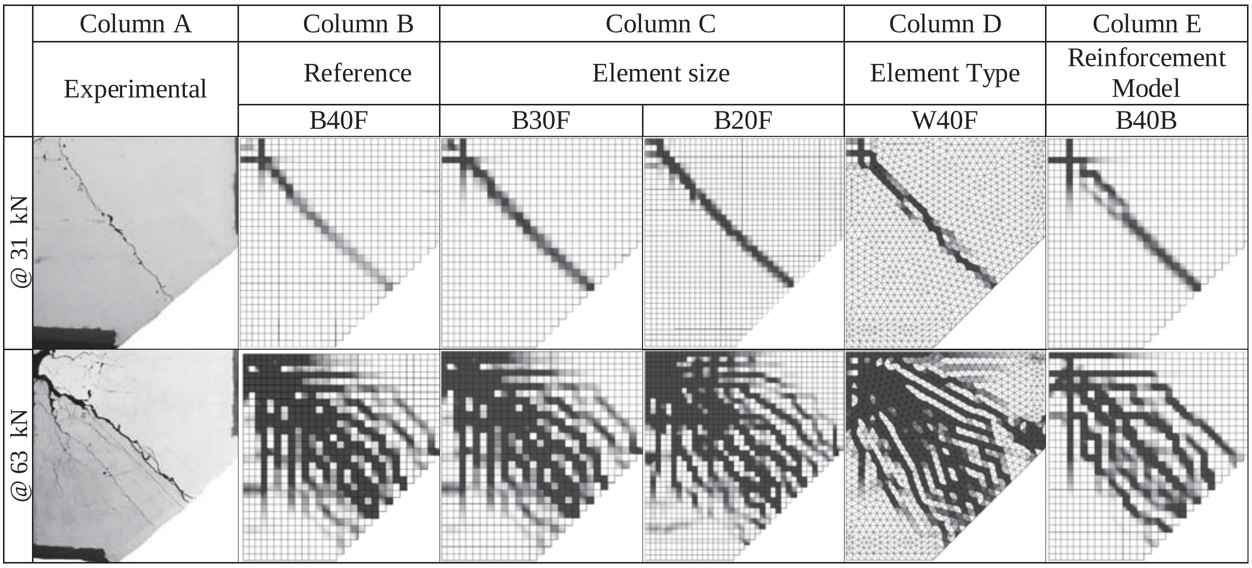

Fig. 4 Crack Pattern from experiment (column A); reference model (column B), and model with different element size (Column C), with different element types (Column D) and with different concrete-reinforcement interaction model (Column E) at initial crack state and ultimate state.

In the future, existing methodologies for the design and evaluation of bridge deck slabs are to be further developed, especially for structural assessment of existing bridge deck slabs using linear FE analysis and enhanced evaluation with nonlinear FE analysis. Recommendations for such analyses will be established and parameters for evaluation of safety will be developed.

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