Multi-level Assessment of a Field Tested RC Bridge Deck Slab

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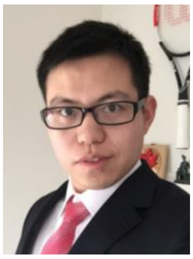

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ABSTRACT
This study proposes a Multi-level Assessment Strategy for reinforced concrete bridge deck slabs. The proposed methods were used for the analysis of previously a tested 55-year old existing bridge deck slab subjected to a shear type of failure, loaded with concentrated loads. The case studies show that the proposed assessment strategy and the analysis methods are feasible and yield reasonable estimates of the load-carrying capacity and structural behaviour such as arching action and load distribution.

Key words: Bridge deck slabs, Multi-level Assessment, FE analysis, Shear distribution

1. INTRODUCTION

In order to provide a systematic approach for the assessment of RC slabs, Plos et al. [1] has developed a "Multi-level Assessment Strategy" which provides recommendations for the assessment of RC slabs using analytical and finite element (FE) models; see Figure 1. The strategy is based on the principle of successively improved evaluation in structural assessment. Accordingly, the assessment of the load-carrying capacity with associated structural response can be conducted through the following levels and methods: (I) Simplified analysis (II) 3D linear (FE) analysis (III) 3D non-linear shell (FE) analysis (IV) 3D non-linear FE analysis with continuum elements and fully bonded reinforcement (V) 3D non-linear FE analysis with continuum elements including the slip between reinforcement and concrete. The aim of this study was to examine the Multi-level Assessment Strategy [1] and modelling methods developed by Shu el al. [2][3] and to investigate the response of a real structure in engineering practice.

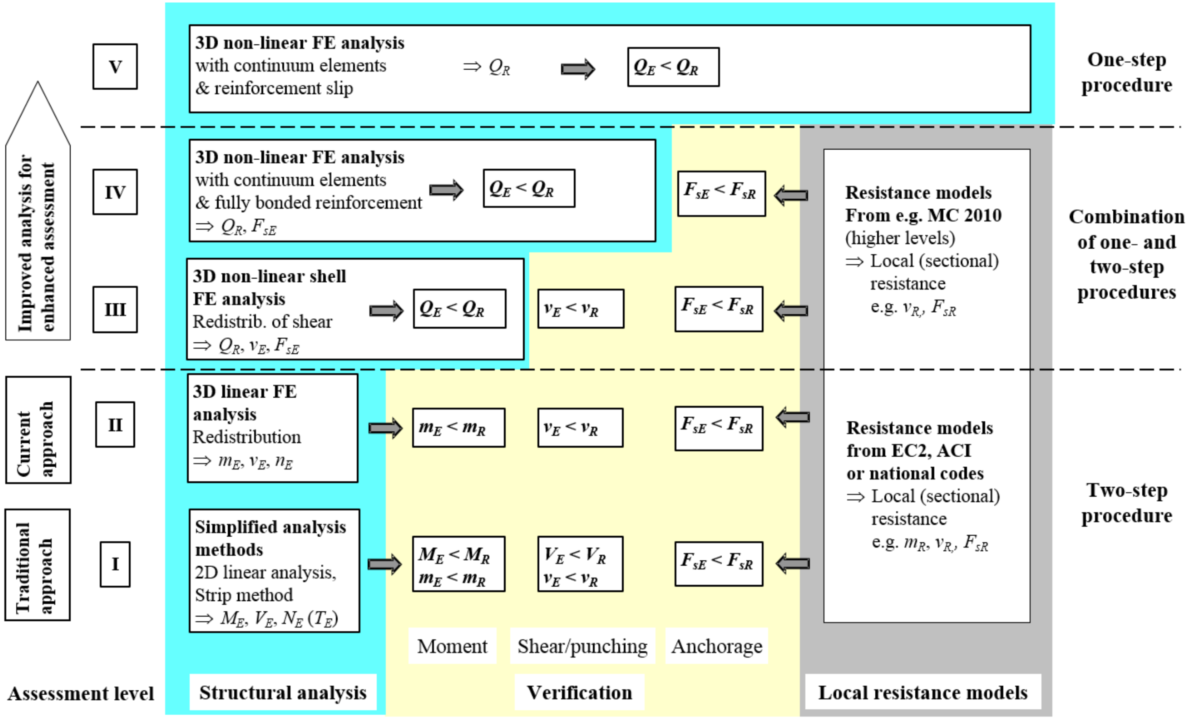


Figure 1: Multi-level Assessment Strategy of RC slabs; from Plos et al. [1].

2. FE ANALYSES OF FIELD TESTED BRIDGE DECK SLAB

To examine the Multi-level Assessment Strategy, this study was conducted by applying a the strategy to a 55-year old RC bridge deck slab subjected to concentrated loads near the main girder in a field failure test. More information about the field test can be found in Bagge et al [4]; see Figure 2. The shear and punching capacity $Q_{u.cal}$ of the deck slab calculated is compared to the failure load $Q_{u.exp}$ from the experiment in Figure 3. At levels I, II and III, one-way shear capacity and punching shear capacity were calculated according to different resistance models based on EC2. At level IV, the load-carrying capacity was obtained from the continuum non-linear FE analysis directly.

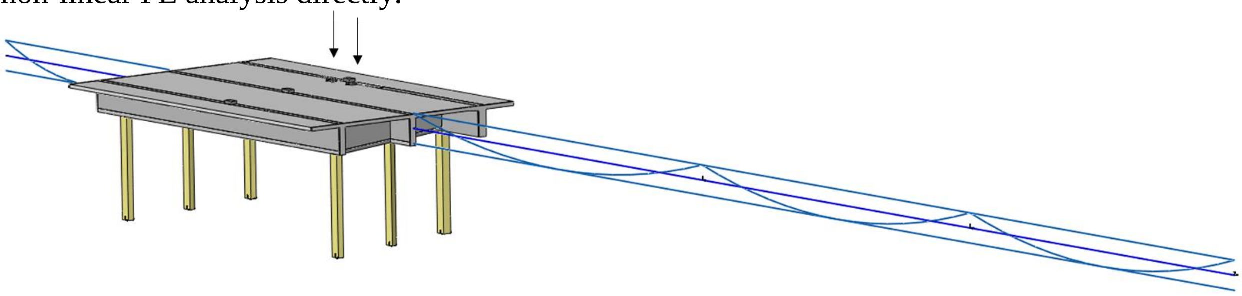
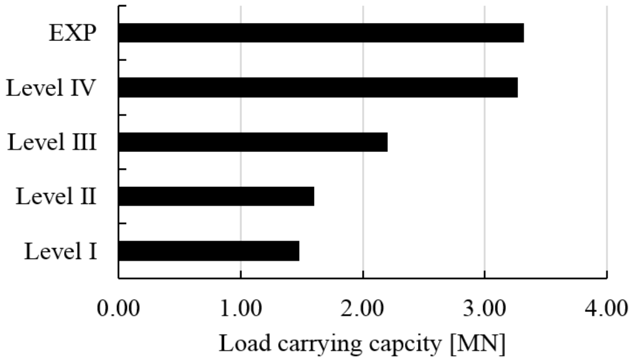


Figure 2: Level IV analysis: non-linear FE model of the tested bridge, showing supports.

3. RESULTS AND CONCLUSIONS

As observed in Figure 3, the shear resistance calculated based on EC2 at level I largely underestimated the real capacity. This indicates that the level I model does not fully represent the behaviour of the tested bridge deck slab. For instance, the influence of prestressing and boundary conditions was not fully taken into account, but are essential for the actual structure. By upgrading the level of approximation, the accuracy of calculated capacity increases. Level II gives similar results as level I, indicating that improved representation of the geometry when determining the load effect is not sufficient. Level III analysis provides a notably higher, still considerably underestimated, load carrying capacity just by representing the non-linear bending response more correctly. Finally, the continuum non-linear FE analysis at level IV provides a load-carrying capacity which is close to that obtained in the experiment.



Capacity	$Q_{u,cal}/Q_{u,exp}$	Control failure mode
3.32 MN		One-way shear & Punching
3.27 MN	0.98	One-way shear & Punching
2.20 MN	0.66	One-way shear
1.60 MN	0.48	One-way shear
1.48 MN	0.45	One-way shear

Figure 3: Load-carrying capacity calculated based on Multi-level Assessment Strategy and comparison to experiment.

In the bridge test, the distance from the edge of the loads to the edge of the girder were only 1.09*d* and 0.6*d* for load plate 1 and load plate 2, respectively. To study the influence of arching action, the loads were gradually moved further away (100 mm per step) from the girder in the level IV model (see Figure 4 (*a*)). The load on position 1 was the same as in the field test. The nominal shear strength (excluding influence of b_w, d & f_{cm}) were calculated assuming a pure one-way shear failure and then the values were compared to laboratory test results obtained by Natario et al. [5] and Lantsoght et al. [6]; see Figure 4 (*b*). From the analysis results, it was observed that the shear capacity decreased when loads were moved further away from the support. When the loading plates were placed in position 4, the failure mode even changed from shear to bending failure.

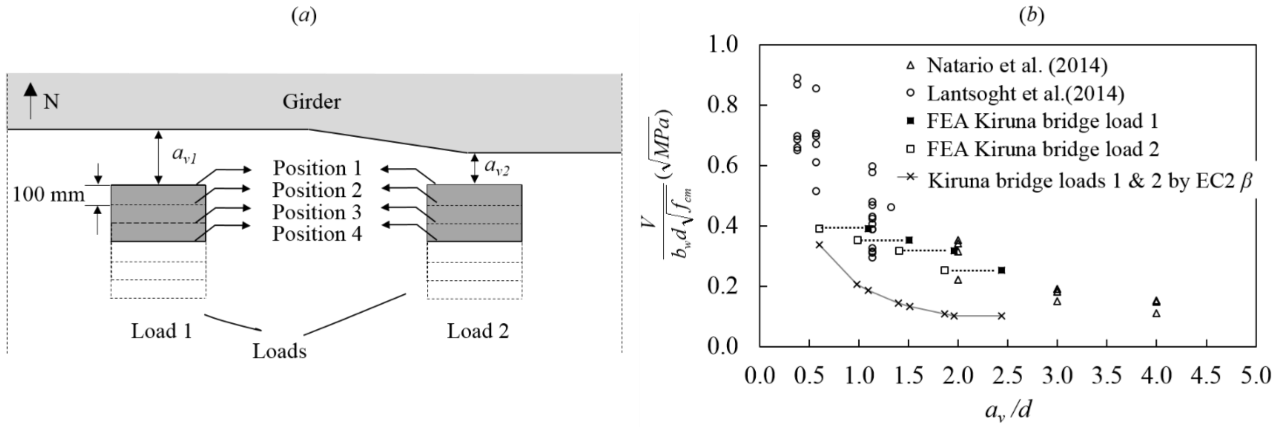


Figure 4: (a) Variation of load positions in the FE analyses; (b) nominal shear capacity of the slab subjected to loads at different positons, with comparison to literature [5][6].

The shear force distribution obtained from the FE analysis at level IV was investigated. In Figure 5, the shear force per unit length along a line in the longitudinal direction of the bridge close to the girder is presented for different load levels (Q/Q_u = 0.2, 0.4, 0.6, 0.8 and 0.95). As expected, force applied to the area of the loading plates is distributed over a larger width closer to the girder. A clear shear force redistribution was observed for the shear force near loading plate 1; the shear force close to loading plate increased fast as the applied load increased at low load levels ($Q/Q_u \leq 0.8$), but stopped to increase at higher load levels ($Q/Q_u > 0.8$). Instead, the shear force in the adjacent region increased faster. However, close to loading plate 2, the phenomenon of shear force redistribution was not as clear. Possible explanations for this are that

(1) there is not enough space for shear force redistribution since loading plate 2 is much closer to the girder and (2) the change in distance to the support due to the changing girder width clearly influenced the shear flow.

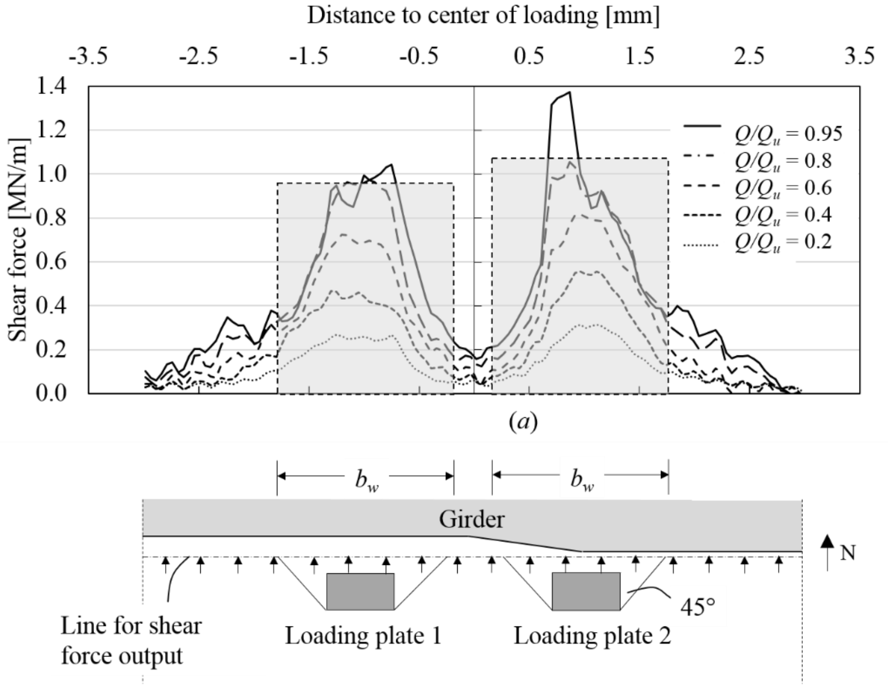


Figure 5: Shear force per unit length across a line parallel to the girder, from FE analysis

4. CONCLUSIONS

It can be learned that existing models in building codes for shear and punching can be underestimated. The analysis method based on the "Multi-level Assessment Strategy" is a straight forward approach to evaluate the load-carrying capacity of existing RC bridge deck slabs. By upgrading the level of assessment, the accuracy of the calculated capacity increases and the continuum non-linear FE analysis at level IV provided a shear capacity very close to the experiment. The shear force distribution is influenced by applied load levels and the failure mode is affected by factors such as boundary conditions and the locations of concentrated load.

ACKNOWLEDGMENT

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