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Numerical prediction of punching behavior of RC slabs using 3D non-linear FE analysis

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ABSTRACT: This study was conducted by carrying out nonlinear FE analysis of RC slabs subjected to punching failure, using three-dimensional (3D) continuum elements. The influence of several modelling choices for concrete material were investigated by comparing results such as load-carrying capacity, load-deflection response and crack pattern from the FE analyses with available experimental data. The analyses of the tested slabs show possibility to accurately predict the load-carrying capacity and realistically simulate the behavior of slabs using the proposed method.

1 INTRODUCTION

The existing infrastructure represents a substantial part of the societal assets and existing bridges represent a huge capital that need to be well administered. Bridge deck slabs are among the most exposed bridge parts and are often critical for punching failure (SB-ICA, 2007). This failure mode is associated with the local introduction of concentrated loads such as columns or wheel loads. It is brittle and therefore undesirable. In addition, punching is one of the most difficult design problem in structural concrete (CEB-FIP, 2001). Consequently, it is important to examine if the current assessment and analysis methods are appropriate. Nonlinear finite element analysis (FEA) has been proved to be an enhanced method to evaluate the punching capacity of Reinforced Concrete (RC) slabs with high level of accuracy (SB-LRA, 2007). However, even though nonlinear FEA has been used increasingly for the assessment of existing structures, building codes do not provide specific guidance on how to perform these analyses. Therefore, the overall aim of this study is to investigate how accurate the response of slabs can be predicted with nonlinear finite element analysis, and how the modelling choices might influence the analysis results. Moreover, these should complement more general guidelines already available in literature such as Fib (CEB-FIP, 2001), Fib (CEB-FIP, 2008) and Hendriks et al. (2012).

In the previous years, several numerical investigations were carried out to apply the finite element method (FEM) to predict punching behaviour of slabs. Among these, studies with 2D models has been conducted, e.g. Menetry et al. (1997) and Hallgren (1996). A separate application was investigated by those who implement layered shell elements which take into account an out-of-plane shear response and allow implementation of three-dimensional constitutive models, e.g. Polak (2005). Compared to 2D element models, a 3D element model offer higher flexibility and accuracy in the modelling of out-of-plane behavior of reinforced concrete structures and generally lead to more realistic results (Plos et al, 2015). 3D numerical analyses of punching failure of slabs can be found in e.g. Ozbolt & Vocke (2000), Zheng et al. (2009) Amir (2014) and Eder et al. (2010).

For the application of FEM for RC structure, a large number of constitutive models to describe the behavior, including cracking of concrete, has been developed. Among those, the models based on the theory of plasticity and continuum fracture mechanics along with smeared crack approach have been widely used. The FE analyses carried out in this study were compared to tests to investigate the influence of different material models as well as modelling choices on results regarding load-carrying capacity and structural behavior. The load-deflection relation and crack pattern from FE analysis have been compared with corresponding experimental data from Guandilini & Muttoni (2004, 2009).

2 DESCRIPTION OF EXPERIMENTS

Guandilini & Muttoni (2004, 2009) carried out a series of tests on slabs at EPFL in Lausanne; see Figure 1. The aim of these tests was to investigate the behavior of slabs failing in punching shear with dif-
ferent reinforcement ratios. The size of the specimens and the compressive strength of concrete were also varied to investigate their effect on punching shear capacity. The test series consisted of eleven reinforced concrete slabs representing internal slab-column connections without transverse reinforcement.

![Figure 1: The dimensions and rebar layout of a “full-size” experimental slab, from Guandilini & Muttoni (2009).](image)

The columns were cast to the slab, with a side dimension $c$ slightly larger than the thickness $h$ of the slabs. The dimensions of the tested specimens were of three types: “full-size” specimens (PG1, PG2b, PG4, PG5, PG10 and PG11), “double-size” specimen (PG3) and “half-size” specimens (PG6, PG7, PG8 and PG9). Table 1 shows the main parameters and characteristics of each specimen. The full-size specimens were loaded through eight concentrated forces acting on the perimeter of the specimen; the load was introduced using four hydraulic jacks placed underneath the laboratory strong floor. During the punching test, the load was increased at a constant speed up to failure. For all specimens, the final failure mode was punching shear, with a clearly delimited punching cone.

Figure 1: The dimensions and rebar layout of a “full-size” experimental slab, from Guandilini & Muttoni (2009).

### Table 1: Dimensions and material properties of test series, from Guandilini & Muttoni (2009).

<table>
<thead>
<tr>
<th>Specimen dimension [m]</th>
<th>Concrete</th>
<th>Reinforcing steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f_c$ [MPa]</td>
<td>$d_c$ [mm]</td>
</tr>
<tr>
<td>Full size 3×3×0.25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PG1</td>
<td>27.6</td>
<td>16</td>
</tr>
<tr>
<td>PG2b</td>
<td>40.5</td>
<td></td>
</tr>
<tr>
<td>PG4</td>
<td>32.2</td>
<td>4</td>
</tr>
<tr>
<td>PG5</td>
<td>29.3</td>
<td></td>
</tr>
<tr>
<td>PG10</td>
<td>28.5</td>
<td></td>
</tr>
<tr>
<td>PG11</td>
<td>31.5</td>
<td></td>
</tr>
<tr>
<td>Double size 6×6×0.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PG3</td>
<td>0.456</td>
<td>32.4</td>
</tr>
<tr>
<td>Half size 1.5×1.5×0.125</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PG6</td>
<td>0.096</td>
<td>34.7</td>
</tr>
<tr>
<td>PG7</td>
<td>0.1</td>
<td></td>
</tr>
<tr>
<td>PG8</td>
<td>0.117</td>
<td></td>
</tr>
<tr>
<td>PG9</td>
<td>0.117</td>
<td></td>
</tr>
</tbody>
</table>

3 NUMERICAL MODELLING

The finite element software DIANA 9.5 (2014) was used to model the slabs, using a 3D tetrahedron 4-node element model, as displayed in Figure 2. Due to symmetry and to reduce the computation time, only a quarter of the slab was included in the FE model. The loading steel plates were included and interface elements were used between the concrete and steel plates to describe the friction. The translation of all nodes at the symmetry faces were fixed in the perpendicular direction. The load was applied as a prescribed vertical displacement to the center node at the bottom surface of the column. To model the distributed load from the hydraulic jack, all the nodes on bottom surface of the column were tied to the center node so that they had the same vertical deflection. An incremental static analysis was performed using specified increment sizes. Each increment was equivalent to a vertical displacement of 0.1 mm until the deflection became 3 mm. After that, the increments were increased to 1 mm to save computation time. The analyses were carried out using a regular Newton-Raphson iteration method based on force and energy convergence criteria, with a tolerance of 0.01.

Figure 2: FE model of a quarter of slab PG2b.

3.1 Material model for concrete

The analyses were performed with different material models for concrete. A Total Strain (TS) rotating crack model (TNO, 2014) which is implemented in
DIANA was selected as reference model since it has been proven able to predict the structural behaviour of slabs accurately (Plos et al., 2015) & (Shu et al, 2014). For comparison with the FE results, a Total Strain (TS) fixed crack model with various shear retention factors was also used.

3.2 Reference model

In the reference model, the TS rotating crack model with crack band approach was used. In this approach, the crack width $w$ is related to the crack strain $\varepsilon_{cr}$ perpendicular to the crack via a characteristic length - the crack band width $h_b$. The reinforcement is modelled assuming complete interaction with the surrounding concrete; consequently, the crack band width equal to the mean crack distance. Figure 3 (top) shows the tensile property of concrete according to Hordijk et al. (1985), used for the reference model. The behavior of concrete in compression was described according to Thorenfeldt et al. (1987). When the stress-strain relationship is used in numerical analyses, the localization of deformations in compressive failure needs to be taken into account. The compression softening behavior is related to the boundary conditions and size of the specimen in the material tests. Consequently, as the stress-strain relation has been calibrated by measurements in compression tests on 300 mm long cylinders, the softening branch needs to be modified for the concrete element size used in the FE model (2013); see Figure 3 (bottom), in which $X$ indicates original strain. The fracture energy was calculated according to MC1990 (2013) and the influence of lateral confinement was taken into account according to Selby & Vecchio (1997).

3.3 Alternative modelling choices

In order to evaluate the influence of variable modelling choices, parameters possibly influencing the shear behavior were investigated. Modelling parameters such as mesh sensitivity, lateral confinement, and fracture energy were varied and the structural response was compared with the reference model. The influence of using a stress-strain relation in tension according to the modified compression field theory (MCFT) by Collins & Mitchell (1986) was also studied since it attempts to take into account the concrete contribution in shear, i.e. friction, aggregate interlocking and dowel action. In the comparative analyses with the TS fixed crack model, parameters such as the shear retention factor was varied with constant values ($\beta = 0.01, 0.1, 0.25$), an aggregate size based function (see Equation 1, $d_{agg} = \text{aggregate size}, \varepsilon_n = \text{total strain}, h = \text{crack band width}$) and the Maekawa shear model (1989).

$$\beta = 1 - \left( \frac{2}{d_{agg}} \right) \varepsilon_n h$$

4 RESULTS AND DISCUSSION

4.1 Mesh sensitivity

The size of finite elements, i.e., mesh density, has effect on the results of analysis. A mesh size study was performed for specimen PG2b for three different mesh sizes with 5, 6 and 7 layers of elements through the slab thickness; see Figure 4 (top). The study confirmed that a mesh with 7 layers was fine.
enough and a further refinement did not bring a significant improvement. Therefore, the model with 7 layers through the thickness was used in the following analyses (reference model). The crack pattern and failure mode obtained from the model with 7 layers of elements is also in a good agreement with experimental result; see Figure 4 (middle and bottom).

4.2 Modelling choices for material behavior in FE models

The alternative modelling choices for the TS rotating crack model included alternative fracture energy according to MC2010 (CEB-FIP, 2013), stress-strain relation according to the MCFT by Collins and Mitchell (1986) and excluding of the confinement model by Selby & Vecchio (1997). The results of analyses with these three modelling choices were compared with the reference analysis as well as with experimental results;

The fracture energy was calculated as 142N/m according to MC2010 (CEB-FIP, 2013) instead of 80 N/m according to MC1990 (CEB-FIP, 1993), leading to an increase of 8% on the load carrying capacity. The tensile model from MCFT obviously has significant influence on the results since the load carrying capacity increased by 91%.

The modelling choices for the TS fixed crack model included different constant shear retention factors, aggregate size based shear retention factor and Maekawa shear model (1989). The results of the analyses with these modelling choices were compared with experimental results and displayed in figure 6. It was observed that only the model with constant $\beta = 0.01$ provides a reasonable results whereas higher $\beta$ values, including the aggregate size based, overestimate the load carrying capacity. The Maekawa model (1989), on the other hand, underestimated the load carrying capacity considerably.

4.3 Results of analyses for all experiments of the test series

Based on the analyses above, the FE model with the TS rotating crack model and the modelling choices of the reference model (section 3.2) was judged to be a reliable method to simulate the structural behavior of slabs subjected to punching failure. Consequently, this method was selected for the analyses of the experiments of the punching test series by Gandelini & Muttoni (2004). The load carrying capacity according to the analyses is tabulated and compared with the experimental results and simplified results according to EC2 (EN 1992-1-1, 2004), in Table 2. The results show that the proposed reference model is able to predict punching capacity rather accurately, with a smaller coefficient of variation (0.13).

Table 2: Comparison between FEA results, EC2 results and experiment results

<table>
<thead>
<tr>
<th>Specimens</th>
<th>$V_{R,EXP}$ [kN]</th>
<th>$V_{R,FEA}$ [kN]</th>
<th>$V_{R,EC2}$ [kN]</th>
<th>$V_{R,EXP}$</th>
<th>$V_{R,FEA}$</th>
<th>$V_{R,EC2}$</th>
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<tbody>
<tr>
<td>PG1</td>
<td>1023</td>
<td>755</td>
<td>950</td>
<td>1.35</td>
<td>1.61</td>
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<tr>
<td>PG2b</td>
<td>440</td>
<td>431</td>
<td>594</td>
<td>1.02</td>
<td>1.11</td>
<td></td>
</tr>
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<td>2153</td>
<td>1910</td>
<td>2347</td>
<td>1.13</td>
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<td></td>
</tr>
<tr>
<td>PG4</td>
<td>408</td>
<td>423</td>
<td>550</td>
<td>0.96</td>
<td>1.11</td>
<td></td>
</tr>
<tr>
<td>PG5</td>
<td>550</td>
<td>498</td>
<td>585</td>
<td>1.10</td>
<td>1.41</td>
<td></td>
</tr>
<tr>
<td>PG6</td>
<td>238</td>
<td>232</td>
<td>223</td>
<td>1.03</td>
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</tr>
<tr>
<td>PG7</td>
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<td>192</td>
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<td>586</td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

| Coefficient of variation | 0.12 | 0.18 |

5 CONCLUSIONS

By comparing the results of the FE analyses with experiment, it is concluded that the reference model was capable of predicting the punching capacity and
crack pattern with good accuracy. Through the parameter study, the results show that the capacity predicted by FE analysis was significantly affected by modelling choices. For the TS rotating crack model, the fracture energy was an important influencing factor. In addition, using the stress-strain relation according to the modified compression field theory (MCFIT) leads to overestimation of the punching capacity even though it was proven to be useful for simulating shear panel tests (Collins & Mitchell, 1986). For the TS fixed crack model, the punching capacity was affected by the shear retention considerably. Consequently, the modelling choices of the reference model could be recommended as a reliable modelling method for RC slabs subjected to punching failure.

6 REFERENCES


Li, B., Maekawa, K., & Okamura, H. (1989). Contact density model for stress transfer across cracks in concrete. Journal of the Faculty of Engineering, the University of Tokyo, 61(1).


