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Application of damage–plasticity models in finite element analysis of punching shear

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ABSTRACT

The paper presents numerical simulations of punching shear in a reinforced concrete slab-column configuration formerly tested in the laboratory. A brief description of the test program at the University of Waterloo is reported. For the simulation, a symmetric quarter of the test configuration is employed. Full three-dimensional finite element discretized geometry is considered together with elastic–plastic reinforcement embedded as truss elements in concrete. Two regularized numerical models of concrete, formulated within elastic–plastic-damage theories, are applied. The first one, called gradient damage, is refined by an additional averaging equation where gradient enhancement involves an internal length scale. In the second one, called rate-dependent damaged plasticity model, a viscoplastic strain rate is introduced. The results for the first model implemented in FEAP and the second model from ABAQUS are discussed in detail. Different issues of numerical simulation to properly predict punching shear behaviour in the slab-column configuration are presented.

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1. Introduction

In multi-storey buildings, designers commonly utilize flat slabs on column systems instead of slab-girder-column combinations. The advantages of flat floors are well-known; they include the reduced height of buildings, and more economical design and performance. However, the problem of punching shear failure occurs in flat slabs.

When a typical internal slab-column connection is considered, visible cracks usually form first due to bending of a slab, on its surface under tensile stresses, in radial directions from the column. On the other hand, punching shear cracks are caused by three-dimensional stress conditions with the concentration of shear stresses in the vicinity of the column. These cracks are formed inside the slab and are not visible. As the load (and stresses) increases these cracks open especially if no transverse reinforcing elements are present; these internal cracks will eventually reach the surface, join the flexural cracks, and form a conical failure surface. At failure, cracking along circular lines is observed directly around the column on the compression surface of the slab and, at a certain distance around the column, on the tensile surface of the slab. The description of this phenomenon, some experimental results and analytical models can be found, in e.g. [1,2].

Punching failure of reinforced concrete slabs without shear reinforcement is brittle and thus dangerous, possibly leading to a progressive collapse of the structure. There are several methods to avoid such failures in flat concrete slabs with shear reinforcement, as described e.g., by [3,4]. Using transverse steel reinforcing elements is one of the most effective methods to prevent punching. These elements transfer tensile stresses and avoid opening of the punching shear crack. Properly placed and anchored punching shear reinforcing elements increase the strength and ductility of the slab-column connections.

Testing of flat concrete slabs for punching shear has been performed in several structural laboratories with many tests done in the 1950s [5–7]. These tests provided the basis for the calibration of the ACI design provisions that appeared in the 1963 version of the code. Since then, many more tests, e.g. [8–10], were done on slabs in punching shear but little changes were implemented in the design formulas. The debate continues on the methods of accounting for concrete contribution to shear carrying capacity, effects of unbalanced moments at the column and of longitudinal reinforcement on punching strength, as well as the methods and effectiveness of shear reinforcement in slabs. Although more testing is still needed, not all of these issues can be rationally and objectively addressed through testing which is time, space and money consuming. Properly calibrated nonlinear finite element

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studies can be a useful source of information on slabs behaviour, complementing experimental investigations. Moreover, such models can be used for extensive parametric studies addressing different aspects of punching shear effects in flat RC slabs.

This paper outlines finite element studies on punching shear in slabs using damage–plasticity finite element (FE) models. The presented simulations are performed for the slabs tested by [11] with the goal to provide adequate predictions of the slabs behaviour, calibrate the models and include rational discussion on the material parameters influencing the FE predictions. The problem of punching shear is not trivial to simulate, since the interaction between flexural and shear failure, while localized fracture zones in a slab evolve, needs to be addressed. Some numerical simulations of punching shear are presented in [12–16]. The simulations done herein utilize two constitutive formulations for concrete; namely gradient enhanced damage coupled to plasticity and so-called damaged plasticity, which is available in commercial FE program ABAQUS [17] and allows an extension to viscosity-dependence. For the enhanced damage–plasticity, the code developed by [18] is applied. This damage model which had first been proposed in [19] was developed and implemented into the open-source code FEAP [20]. Although the problem of proper simulation of the localized failure seems to be not severe in reinforced structures, it has been decided to employ a regularized continuum description to minimize the effects of pathological mesh sensitivity and numerical instabilities. Continuum models contain various regularization methods, which are also called localization limiters [21]. Such a limiter can be included in many ways, for example: a certain variable averaged by a gradient operator is incorporated in the formulation [19,22–27]; or an additional rate-dependent term like viscosity is enclosed in the constitutive equation [28–33].

In the presented analysis linear kinematics is assumed.

2. Test program at the University of Waterloo

The slab specimens analyzed in this work were tested by [11]. These were full-scale models representing interior slab-column connections with the column stub of 150×150 mm cross section and simply supported along the edges with corners restricted from lifting. The overall dimensions of slab-specimens were $1800 \times 1800 \times 120$ mm and they were supported along the 1500×1500 square perimeter placed on neoprene pads. The specimens represented portions of a slab-column continuous system, bounded by the lines of contraflexure around the column.

The specimens were loaded downwards through the column until failure. Note that the experimental configuration was in an upside down position in comparison with the real structural case. The specimen in the test frame is shown in Fig. 1(a). The flexural reinforcement was formed by 10 M bars of nominal cross-section area $A_r = 100\,\text{mm}^2$. The bars in the tension mat had the spacing of 100 mm and 90 mm for the upper and lower orthogonal layers in order to produce almost identical bending capacities in the two directions. The reinforcement bars on the compression side formed a grid with 200 mm spacing. The column segments were reinforced with four 20 M bars.

The full test program consisted of six specimens, two of which had openings constructed near the column. The control slab SB1 had no shear reinforcement and failed by punching shear in brittle manner. All other specimens were strengthened using an increasing number of 9.5 mm diameter bolts symmetrically placed in concentric rows around the column. They showed substantial ductility and failed in flexure or combination of flexure and shear., cf. [11]. However, for all specimens, the flexural cracks initiated at column corners and propagated radially towards the slab edges. In this paper only slab SB1, without shear reinforcement, is used for the FE studies. The concrete compressive strength was 44 MPa and the measured tensile strength was 2.13 MPa. Flexural reinforcement yield strength was 455 MPa.

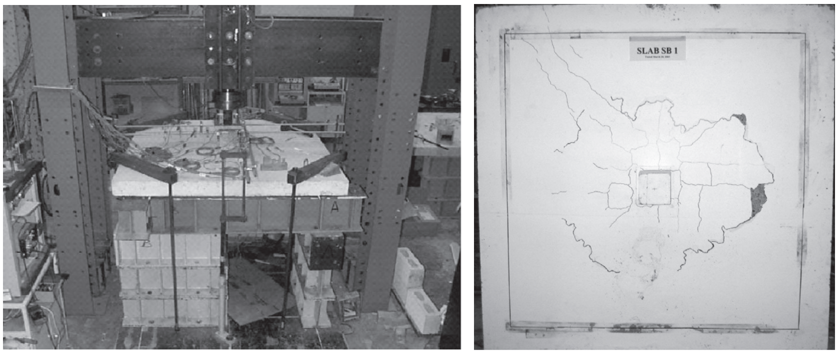
The results in terms of deflections, strains and crack widths were monitored. The experimental load–deflection diagram for SB1 is reproduced further in figures together with numerical results. The final experimental crack pattern, as seen from the tension side, is presented in Fig. 1(b).

3. Review of employed constitutive models

Two inelastic constitutive models have been used in the simulations. They both assume that concrete is initially an isotropic elastic material, described by elastic constants: Young modulus E and Poisson ratio ν. The oldest approach to fracture modelling called smeared cracking originated in [34] and was further developed for concrete fracture simulations in the eighties, for an overview of those efforts the reader is referred e.g. to [35–37].

In this paper the authors decided to employ two versions of coupled inelastic models involving damage and plasticity. They are both based on the concept of effective stress acting on the undamaged skeleton of the material and involve permanent strains when a yield limit is reached.

The first approach is the gradient-enhanced damage–plasticity model described in [38,39] and implemented by the authors in FEAP. This model involves a simplified representation of cracked concrete based on a single scalar damage parameter, but it is nonlocal and removes the pathological discretization sensitivity commonly encountered when standard continuum models are used to simulate softening related to damage. The model incorporates



(a) Experimental setup. (b) Crack pattern for SB1.

Fig. 1. RC slab-column connection. Experimental setup and final crack patterns for slabs SB1 visible from the tension side. Photos quoted from [11].

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