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Kam-Tim Chau
Jidong Zhao *Editors*

Bifurcation and Degradation of Geomaterials in the New Millennium

Proceedings of the 10th International Workshop on Bifurcation and Degradation in Geomaterials

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Preface

Interests in localization and related instabilities in the field of geomechanics date back to the JMPS paper of Rudnicki and Rice (1975) "Conditions for the localization of deformation in pressure-sensitive dilatant materials". It models strain localization in rocks as material instability. Subsequently, strain localization in soils was considered as shear band by Vardoulakis et al. (1978) "Formation of shear bands in sand bodies as a bifurcation problem" in IJNAMG. Research interests in this area expanded considerably and resulted in the first international workshop on Localization of Soils organized in Karslrule, Germany, February 1988, and this international workshop became the first sequel to our current International Workshop on Bifurcation and Degradation in Geomaterials (IWBDG). This aroused so much enthusiasm and interest in the fundamental aspects of bifurcation theory to soils that the second workshop followed in Gdansk, Poland, September 1989. The topic was then extended to rock mechanics at the third international workshop in Aussois, France, September, 1993. In 1997, this international workshop series was expanded to include instabilities and degradations in geomaterials at the fourth workshop in Gifu, Japan, September 1997. Since then, the name of IWBDG was adopted and subsequent international workshops were held at Perth, Australia, November 1999 (fifth), at Minneapolis, USA, June 2002 (sixth), at Crete, Greece, June 2005 (seventh), at Lake Louise, Canada, May 2008 (eighth) and at Porquerolles, France, May 2011 (ninth). The tenth international workshop of this series continued this central theme of bifurcation and degradation of geomaterials, and was held in Hong Kong during May 28–30, 2014 (10th IWBDG) at the beautiful campus of the Hong Kong Polytechnic University.

The 10th IWBDG was attended by 66 participants representing 16 countries or regions, including Australia, Austria, Belgium, Canada, Chile, France, Greece, Hong Kong China, Iran, Japan, Mainland China, Norway, Poland, Sweden, UK and USA. A total of 55 presentations were delivered, covering three full days. Among them, 17 were registered as students. This proceedings published by Springer contains 54 peer reviewed full papers.

The workshop would not be possible without the help of qualified and diligent reviewers, and they include Mustafa Alsaleh, Ronaldo I. Borja, Jacques Desrues, Zhiwei Gao, Ning Guo, Peijun Guo, Marte Gutierrez, Wenxiong Huang, Mingjing Jiang, Xia Li, Francois Nicot, Fusao Oka, Jacek Tejchman, Antoinette Tordesillas, Richard Wan, Gang Wang, Jeff Jianfeng Wang, Wei Wu, Zhenyu Yin, Jidong Zhao (in alphabetical order). Their helps are highly appreciated. The financial sponsors are Fong On Construction Limited (courtesy of Dr. James C.K. Lau, JP) and the Faculty of Construction and Environment, The Hong Kong Polytechnic University (through Conference Support Scheme). Non-financial sponsors include Geomechanics Committee, AMD of ASME, Elasticity Committee, EMI of ASCE, HKGES, Geotechnical Division of HKIE and TC103 Numerical Methods of ISSMGE. The clerical and logistics supports from the Department of Civil and Environmental Engineering are highly appreciated.

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Contents

Simulations of Shear Zones and Cracks in Engineering Materials Using eXtended Finite Element Method

Jerzy Bobiński and Jacek Tejchman

Abstract Numerical simulations of cracks and shear zones in quasi-brittle materials are presented. Extended Finite Element Method is used to describe both cracks and shear zones. In a description of tensile cracks, a Rankine criterion is assumed. A discrete Mohr-Coulomb law is adopted for simulations of shear zones. Results of simple numerical tests: uniaxial tension, bending and biaxial compression are demonstrated.

1 Introduction

Localization of deformation is observed in many materials like concrete, glass, metals, polymers, soils and rocks. In soils, rocks and concrete this phenomenon is manifested by the presence of shear zones and cracks. The numerical modelling of strain localization within continuum mechanics requires the use of advanced constitutive models, based e.g. on an elasto-plasticity or hypoplasticity theory. All continuum constitutive laws have to include a characteristic length of micro-structure to obtain mesh-independent results of the width and spacing of localized zones. Shear or tensile zones can be also simulated more explicitly by using interface (cohesive) elements or the discrete element method (DEM).

Another method, which gains strong popularity in different areas to describe shear and tensile zones in quasi-brittle materials and soils as discontinuities in a displacement field is the eXtended Finite Element Method (XFEM). It is based on the Partition of Unity Method and it assumes an enrichment of displacements to capture jumps across localized zones. Extra degrees of freedom are added in regions where strain localization occurs. This approach allows for a placement of localized zones within finite elements.

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The paper presents numerical simulations of the formation and growth of shear and tensile zones in soils with the aid of the eXtended Finite Element Method. To describe shear zones, the constitutive law was based on the Mohr-Coulomb model. To simulate cracks under tensile loading, the Rankine approach was used. Some tests like biaxial compression, uniaxial tension and three-point bending were simulated.

2 Extended Finite Element Method

2.1 General Description

The eXtended Finite Element Methods (XFEM) allows for simulating displacement jumps across finite elements (Belytschko and Black 1999). It can be used to simulate brittle materials (Moës and Belytschko 1999), cohesive cracks (Wells and Sluys 2001) or shear zones in soils (Song et al. 2006). The formulation used follows (with some slight modifications and improvements) the general idea presented by Wells and Sluys (2001). It is based on the so-called shifted-basis enrichment (Zi and Belytschko 2003) to describe a displacement field with discontinuous jumps. This modification has two advantages over the standard version: the total nodal displacements are equal to the standard displacements and the implementation of finite elements is simpler since two types of elements exist only.

In a non-cracked region, a linear elastic constitutive law between stresses and strains was always assumed. To create a new crack segment, a crack creation criterion has to be fulfilled at least in one point of the element at the front of the crack tip. Crack tips (end points) can be placed only at finite elements edges. In order to smoothen the stress field around the crack tip, the averaged stresses were used in determining the crack direction (Wells and Sluys 2001) defined as:

$$\sigma^* = \int_V \sigma \, w dV \text{ with } w(r) = \frac{1}{(2\pi)^{3/2} l_{av}^3} \exp\left(-\frac{r^2}{2l_{av}^2}\right), \tag{1}$$

where the domain V is the semicircle at the front of the crack tip, w—the weight function, r—the distance between points and l_{av}—the averaging length, related to the size of finite elements.

2.2 Discrete Rankine and Mohr-Coulomb Model

A discrete Rankine model was defined to simulate tensile cracks. To activate a crack, the following condition was assumed:

$$\max\{\sigma_1, \sigma_2, \sigma_3\} > f_t, \tag{2}$$

where σ_1, σ_2 and σ_3 are the principal stresses and f_t is the tensile strength. The direction of the crack extension was assumed to be perpendicular to the direction of the maximum principal averaged stress (see Eq. 1). The following loading function within a discrete cohesive law was chosen:

$$f([[u_n]], \kappa) = [[u_n]] - \kappa \tag{3}$$

with the history parameter κ equal to the maximum value of the normal component $[[u_n]]$ of the displacement jump achieved during deformation. With $f > 0$, a loading case occurs, while $f < 0$ stands for a reloading or an unloading phase. The softening of the normal component of the traction vector was described using an exponential relationship:

$$t_n = f_t \exp\left(-\frac{f_t \kappa}{G_f}\right)\left(1 - \exp\left(-d_f \frac{f_t}{G_f}\kappa\right)\right), \tag{4}$$

where G_f is the fracture energy and d_f is the numerical drop factor (Cox 2009). In the tangential direction, a linear relationship between displacement jump and traction was defined by the stiffness T_s.

To simulate shear zone, a discrete version of the elasto-plastic Mohr-Coulomb law with the internal friction angle ϕ and dilatancy angle ψ was used. The activation function was also based on Mohr-Coulomb criterion:

$$\frac{1}{2}(\sigma_1 - \sigma_3) + \frac{1}{2}(\sigma_1 + \sigma_3)\sin\phi - c\cos\phi > 0. \tag{5}$$

Linear softening of material cohesion c was defined as:

$$c(\kappa) = \max\{c_{\max} - H\kappa, c_{res}\}, \tag{6}$$

where $c_{\max}$—the maximum cohesion, c_{res}—the residual cohesion and H—the softening modulus. The penalty stiffnesses: normal K_N and shear K_S were defined to calculate elastic displacement jumps. It allows to use standard plasticity algorithm. High value of K_N prevents over penetration of shear zone surfaces. The direction of the propagation θ was calculated based on a bifurcation analysis with respect to principal averaged stresses directions:

$$\tan^2\theta = \frac{2 + \sin\phi + \sin\psi}{2 - \sin\phi - \sin\psi} \tag{7}$$

3 Numerical Examples

First, a simple uniaxial tension test was simulated. The width of the specimen was 100 mm, height 150 mm and thickness 1 m (Fig. 1a). The starting point of the crack propagation was defined in the middle of the left edge. The modulus of elasticity was equal to $E = 30$ GPa, the Poisson's ratio was $\nu = 0.2$, the tensile strength was $f_t = 3$ MPa and the fracture energy $G_f = 100$ N/m with exponential softening (Eq. 4). The drop factor was chosen as $d_f = 10^4$. The stress averaging

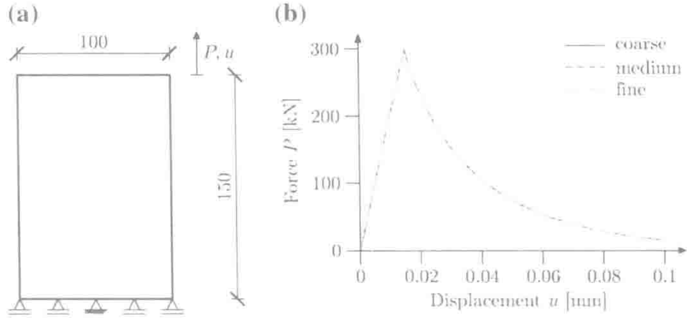

Fig. 1 XFEM results: for uniaxial tension **a** geometry and boundary conditions, **b** force-displacement diagrams

length was $l_{av} = 0$. The shear stiffness was $T_S = 10^{12}$ MPa/m. To examine the mesh insensitivity, simulations were performed with three different 3-node triangle FE-meshes: coarse (600 elements), medium (2,400 elements) and fine (5,400 elements). Almost the same force-displacement curves were obtained (Fig. 1b). A horizontal crack was properly reproduced.

Next, the simulations of a three-point bending test of notched concrete beams were carried out. The geometry was taken from experiments by Le Bellego et al. (2003) Three different beam sizes were numerically investigated: small ($h = 8$ cm), medium ($h = 16$ cm) and large ($h = 32$ cm). The span length of the beam was equal to $L = 3\,h$ (Fig. 2a). The loading was prescribed at the top edge at the mid-span via the vertical displacement. In the simulations, $E = 38.5$ GPa and $\nu = 0.2$ were taken with the tensile strength of $f_t = 3.2$ MPa. The exponential softening with the fracture energy $G_f = 80$ N/m was defined. The drop factor was chosen as $d_f = 10^4$. The stress averaging length was $l_{av} = 1$ cm. The material parameters were the same for all beams. Three different meshes with 3,068, 4,956 and 9,132 3-node constant strain triangles were defined for a small, medium and large beam, respectively. The crack starting points were located at the left side near the node at the line of the symmetry of the each beam.

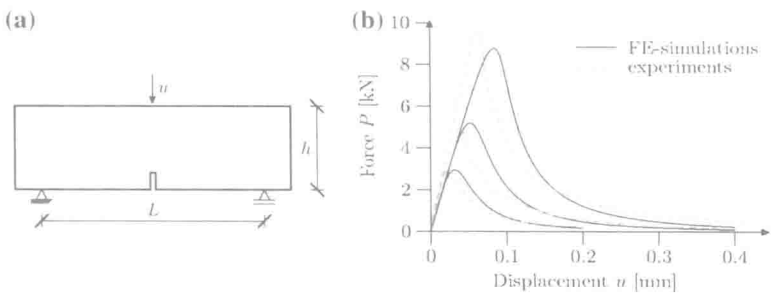

Fig. 2 XFEM results for three-point bending test: **a** geometry and boundary conditions, **b** force-displacement diagrams

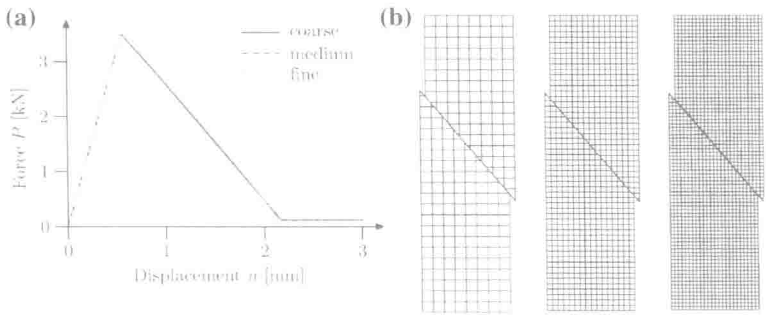

Fig. 3 XFEM result for biaxial compression: **a** force-displacement curve, **b** deformed meshes

Figure 2b shows the calculated force-displacement diagrams for notched concrete beams as compared with the experimental curves. Since it was not our intention to perfectly reproduce experiments, some differences can be seen, especially in a linear elastic regime. In turn, the softening behaviour of beams was properly captured. The maximum calculated force was equal to 2,945, 5,185 and 8,784 N for the small, medium and large beam, respectively. They were similar to the values obtained in experiments. Thus, a strong size effect in concrete beams (expressed by the increase of the load bearing capacity with decreasing size) was also properly reproduced.

Finally, biaxial compression was tested to simulate discrete shear zones. The specimen of 4 cm wide and 14 cm high was loaded via imposed vertical displacements on the top edge. The following material parameters were assumed: $E = 50$ MPa, $\nu = 0.3$, internal friction angle $\phi = 20^\circ$, dilatancy angle $\psi = 0^\circ$, maximum cohesion $c_{\max} = 30$ kPa, residual cohesion $c_{res} = 1$ kPa and softening modulus $H = 10$ MPa. The penalty stiffnesses were equal to $K_N = 5$ GPa and $K_S = 0.5$ GPa. The stress averaging length was $l_{av} = 0$. The starting point of the shear zone was located at the left edge (weak spot). Three meshes with 4-node quad elements were defined with 224, 896 and 2,016 elements for the coarse, medium and fine mesh, respectively. The obtained force-displacement curves are presented in Fig. 3a. Identical responses were achieved. The inclination of a shear zone was also properly reproduced (Fig. 3b).

4 Conclusions

The numerical results with the aid of the eXtended Finite Element Method have shown that this method is able to properly reproduce tensile cracks and shear zones as discontinuities in the displacement field under different loading conditions.

The present research activity is focused on combining XFEM with continuous cracks/shear zones descriptions using elasto-plastic and hypoplastic constitutive laws with non-local softening to describe the entire failure mechanism. More advanced boundary value problems with localization will be analysed.

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Discrete Modelling of Micro-structural Phenomena in Granular Shear Zones

Michal Nitka, Jacek Tejchman and Jan Kozicki

Abstract The micro-structure evolution in shear zones in cohesionless sand for quasi-static problems was analyzed with a discrete element method (DEM). The passive sand failure for a very rough retaining wall undergoing horizontal translation towards the sand backfill was discussed. To simulate the behaviour of sand, the spherical discrete model was used with elements in the form of rigid spheres with contact moments.

1 Introduction

Earth pressure on retaining walls is one of the soil mechanics classical problems. In spite of intense theoretical and experimental research works over more than 200 years, there are still large discrepancies between experimental results and relevant theoretical solutions. The reason is the complexity of deformation field in granular bodies, especially near the wall, created by spontaneous emergence of shear localizations in a form of single or multiple narrow zones—the fundamental phenomenon characteristic for a granular material at shear deformation.

The patterning of shear zones is usually not taken into account in engineering calculations due to the lack of the basic knowledge on the phenomenon, which gives some practical importance to the research described in this paper. Its objective is to investigate, using the discrete element method DEM, the quasi-static evolution

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