NUMERICAL SIMULATION OF SHEAR WALL IN CONCRACK BENCHMARK

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Abstract: The paper presents results of numerical simulation of shear wall test made within the CONCRACK bench mark project organized by French national research program CEOS. Monotonic load test is simulated by computer code ATENA. Results of predictions performed prior tests as well as follow-up analysis are shown.

1 INTRODUCTION

The CONCRACK bench mark organized by French research project CEOS was devoted to cracking phenomenon in reinforced concrete structures. Several real scale specimens were produced and tested to failure, whereas a detail measurements of load, deformations and cracking were recorded. Authors participated in a prediction bench mark study of experimental behavior of shear wall under monotonic loading. Numerical models used by authors for simulation of the shear wall behavior were based on 2D and 3D representations. The paper describes the models and results of analysis and aspects of model uncertainties involved in prediction and follow-up phases of resistance verification.

2 NUMERICAL MODELS

Numerical simulation was made by software ATENA developed by authors [1]. In following the material model as well as the finite element representation used in analysis are briefly described.

2.1 Material models

Concrete is simulated by ATENA’s fracture-plastic material model, which treats the cracking and compressive behavior in interaction. Here we mention briefly some features of the constitutive modeling relevant to presented analysis. More details can be found in [2].

For crack propagation a crack band model based on fracture mechanics is used as illustrated in Figure 1, where L_t is the crack band size. Crack band is further modified for effect of crack orientation with respect to finite element [2].

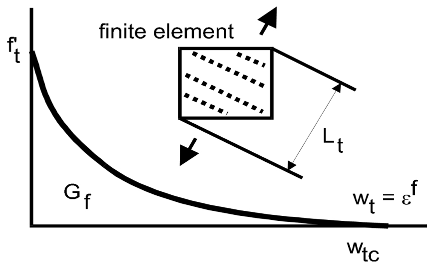


Figure 1: Tensile softening.

The compressive behavior is modeled by plasticity with yield function due to Menetre-Willam shown in Figure 2.

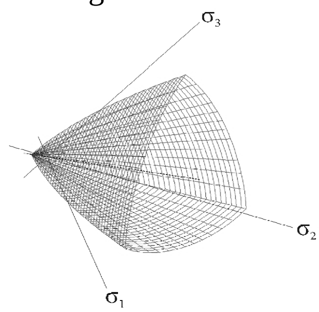


Figure 2: Yield function due to Menetre-Willam..

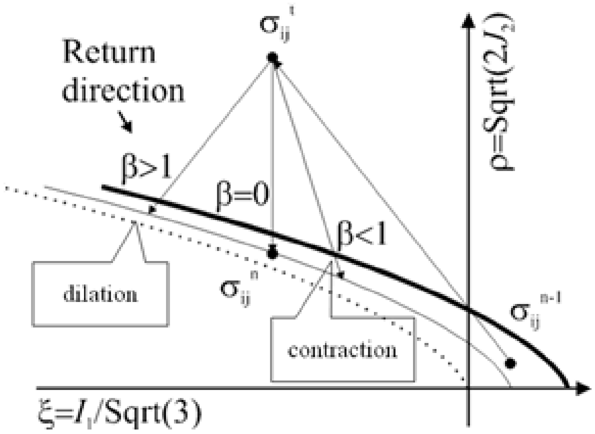
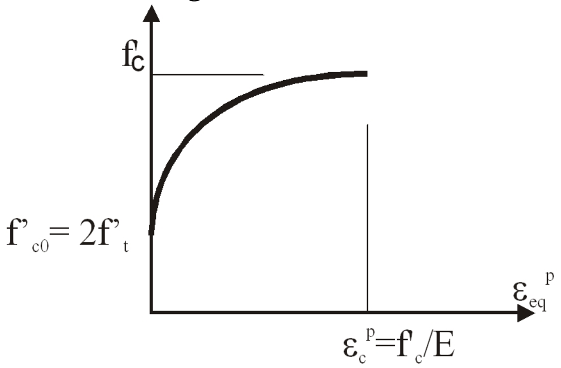


Figure 3: Return direction β in predictor-corrector plastic algorithm.

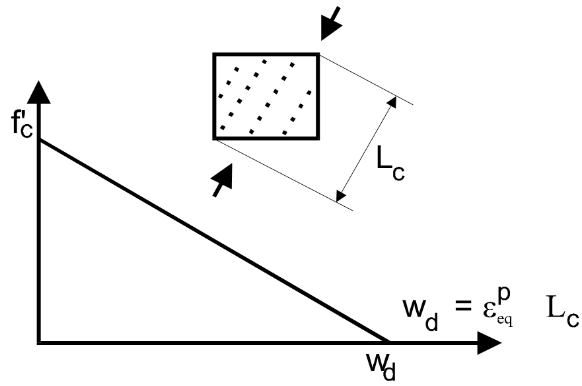
Non-associated flow rule of incremental plasticity is used in the predictor-corrector algorithm as illustrated in Figure 4. Direction of return vector in the High-Westergard space (where ξ represents the volumetric axis, and ρ deviatoric axis) is controlled by parameter β. In case of positive β>0, volume of material increased during plastic distortion, which can generate a confinement.

Both, compression and tension, exhibit softening after reaching strength. This leads to a strain localization into cracks and crushing zones, which is controlled by localization limiters according to the crack band theory.

Concrete in compression exhibits hardening prior peak stress and softening in the post-peak as illustrated in Figure 4.



(a) Concrete hardening.



(b) Concrete softening.

Figure 4: Hardening and softening in compression.

The ductility of compressive failure can be controlled by parameter w_d, which can be regarded as a final plastic deformation of the crush zone. In analogy with fracture energy the model describes the energy dissipated in the process of compressive failure. It was found that the ultimate load of the shear wall was sensitive to this parameter as will be discussed later.

Furthermore, the model has a provision for effect of cracks on reduction of compressive strength and shear resistance (aggregate interlock).

For reinforcement a bi-linear stress-strain law is used. For bond behavior of reinforcement the CEB model for bond slip is considered.

2.2 Numerical model

Two models were considered, one in full 3D representation and other in 2D plane stress simplification. The geometry of the model is shown in Figure 5. The model includes the shear wall (1x4m, subject of investigation), edge beams for load introduction and external steel rods simulating vertical confinement. The load was applied as a shear force P to the top loading beam and the displacement d was recorded as a difference between horizontal displacements of points on top and bottom shear wall edges, $d = u_T - u_B$, as shown in Figure 5.

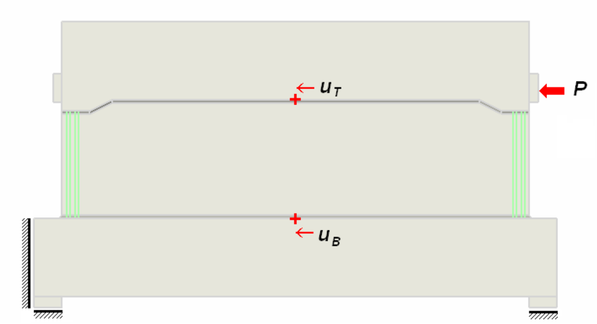


Figure 5 Geometry of test specimen, side view.

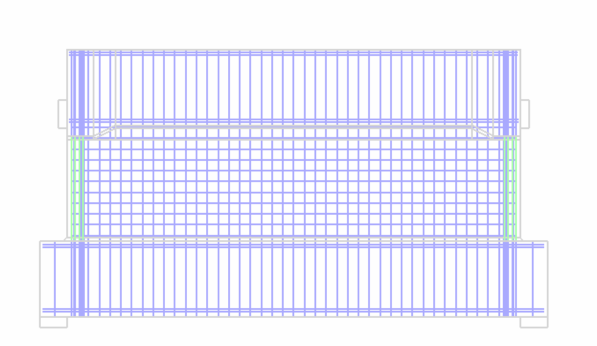


Figure 6 Reinforcement layout in side view..

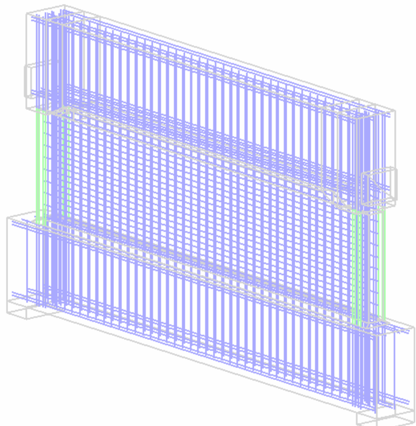


Figure 7 Reinforcement layout in side view.

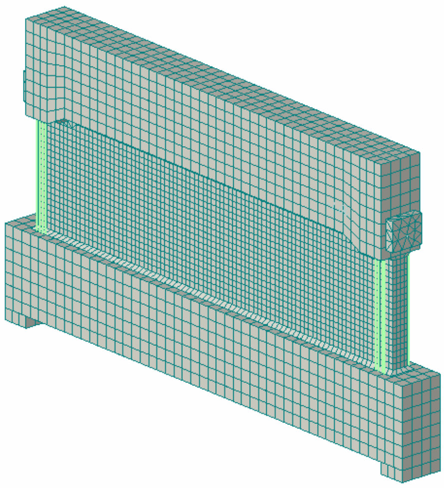


Figure 8 FE mesh of 3D model.

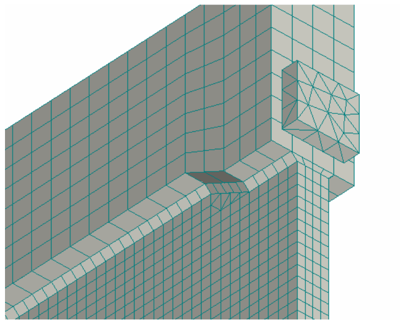


Figure 9 Detail view of loading plate.

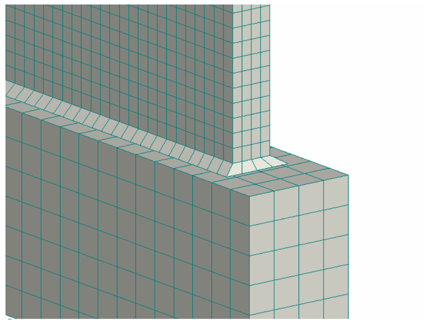


Figure 10 Detail view of wall support

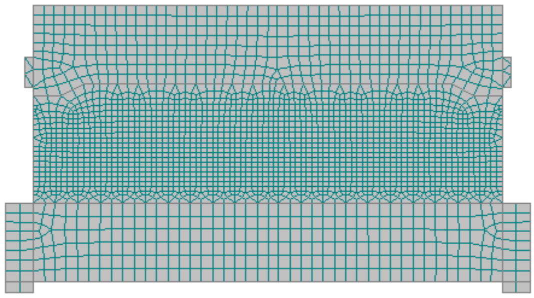


Figure 11 FE mesh of 2D model.

2.3 Material parameters

Material parameters of concrete in shear wall and support beams are listed in Table 1.

Table 1: Material parameters of concrete.

Compressive strength f_c [MPa]	42.5
Tensile strength f_t [MPa]	3.3
Specific fracture energy G_f [N/m]	100
Critical compressive displacement w_d [m]	0.5
Dilation β	0
Fixed cracks	-

The above parameters were assumed for prediction phase of the bench mark. After publishing of experimental data, in a follow-up analysis, different parameters for parameters w_d and β were used in order to better capture the ultimate load.

Reinforcement yield strength was set as fy=554 MPa. In the bond stress-slip relation according to CEB formula the peak point (bond strength) was considered as 8.9 MPa.

2.4 Loading and solution

Loading was applied by prescribed displacement and the loading force was found as a reaction at the loading point. Numerical solution was done by Newton-Raphson method with tangent predictor and with the line search. Equation solver was iterative. Residual error tolerance was 0.001 and maximum number of iterations 80.

3 RESULTS

Load-displacement diagrams are shown in Figure 12.

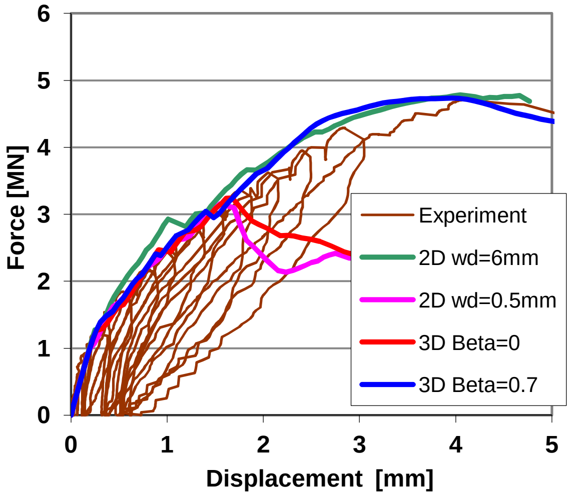


Figure 12 Summary of load-displacement diagrams.

A second version of experimental data with corrected deformations is used for this evaluation. Parameters used for prediction in case of 2D model: compression parameter w_d=0.5mm; in case of 3D model dilation parameter β=0 (no dilation).

These simulations show more brittle failure than experiment. Experimentally obtained maximum load was 4710 kN, while in prediction analysis based on 3D model P_{max}=3246 kN and the one based on 2D model P_{max} =3153 kN.

After comparison with experimental data it was believed that the concrete ductility in compression was underestimated and a second set of analyses was performed with parameters for 2D model w_d=6mm and 3D model β=0.5 (dilation). These data provided a good agreement with experiment (3D P_{max}=4730 kN and 2D model P_{max} =4785 kN.

Analysis of 3D model offered a good insight in failure mode of the shear wall. Crack pattern and crack widths at failure are shown in Figure 13-15.

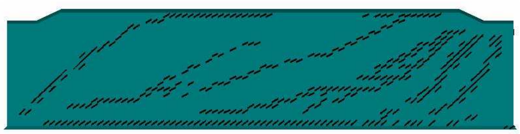


Figure 13 Crack pattern at Mximum load.

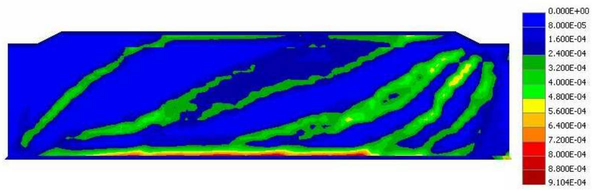


Figure 14 Crack widths at Mximum load.

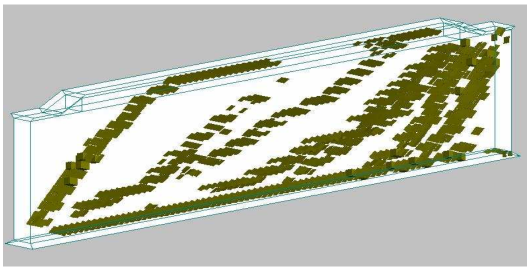


Figure 15 Crack pattern inside of wall at Mximum load.

Crack location and direction was in good agreement with experiments (not shown here). Failure mode was due to concrete in compression at the wall corner near the loading point.

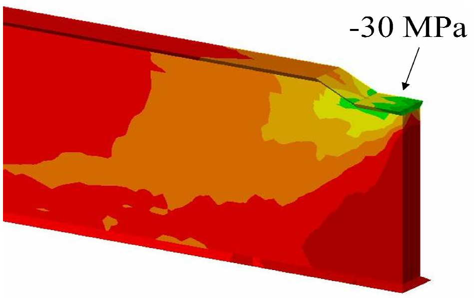


Figure 16 Stress distribution at failure region, β=0.

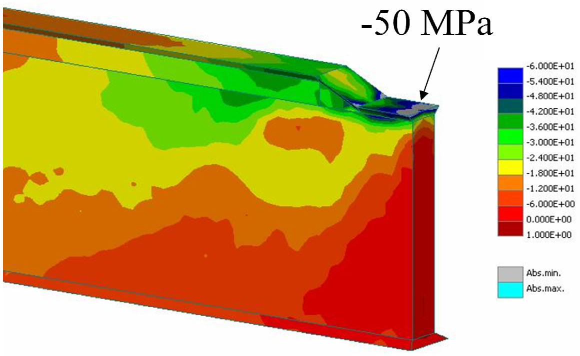


Figure 17 Stress distribution at failure region, β=0.7.

The stress state in concrete at this location is shown in Figure 16 and Figure 17. Note the severe damage due to cracks at this location shown in Figure 13.

Reinforcement stress is increased in cracks but it is not reaching the yield level.

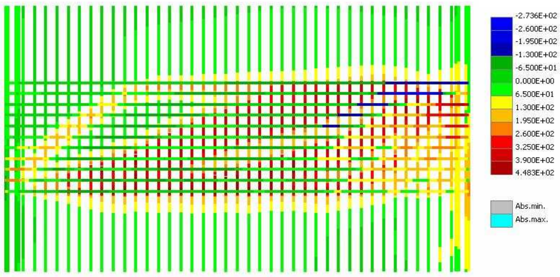


Figure 18 Stress in reinforcement.

Both 3D and 2D models provided similar results. Crack patterns in 2D and 3D were almost identical. The differences were at concrete stress state at failure. In 3D the confinement could be increased by setting the dilation parameter b=0.7. This caused a stress increase due to confinement from -30 to -50 MPa. However, the concrete resistance in 2D could be increased only increasing softening parameter form wd=0.5 to 6mm. It shows, that in 2D analysis the resistance furnished by confinement effect in 3D stress state can be in 2D artificially generated by means of increased ductility softening parameter w_d.

4 MODEL UNCERTAINTY

The present study illustrates the issue of model uncertainty in application of numerical simulation for verification of resistance safety.

In prediction stage a brittle material model was chosen leading to underestimation of resistance to 0.71 of reality. Such a provision is on safe side and can be considered as a rational and conservative solution in case of not experimentally validated model. After validation the model can fit the reality, model uncertainty can be significantly reduced and resistance reserves can be utilized.

12 CONCLUSIONS

Simulation of shear wall resistance was performed by a non-linear finite element analysis based on fracture-plastic material model of concrete. In a prediction phase the ultimate load was underestimated by 31% due to low of confinement effect.

In the follow-up phase after model validation by experimental results the ultimate load and failure mode could be well reproduced. Importance of dilation effect in plastic deformation of concrete was confirmed.

Both 3D and 2D numerical models performed in good agreement. However, the study illustrated differences of both models in confined regions. In case of 3D resistance is facilitated by confinement generated by general strength criteria in multi-axial stress state. This is of course not available in 2D plane stress model. However a similar effect can be fulfilled by increasing concrete ductility by modifying the softening of concrete in compression.

ACKNOWLEDGEMENT

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