FINITE ELEMENT ANALYSIS OF CLAMPED SHEAR WALLS UNDER EARTHQUAKE ACTION

FINITE-ELEMENTE-BERECHNUNG EINGESPANNTER SCHUBWÄNDE UNTER ERDBEBENEINWIRKUNG

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SUMMARY

Reinforced concrete shear walls are vital in multi-story buildings for mitigation of seismic loads. Their complex load bearing and failure mechanisms are still being extensively investigated in research programs focused on cyclic behaviour relevant for seismic mitigation. Under a research program at the University of Applied Sciences of Northwestern Switzerland (FHNW), large-scale shear wall experiments are being performed. Four full-scale reinforced concrete shear walls were tested under cyclic loading to assess their seismic performance. The test boundary conditions are selected to represent the clamping of a shear wall having two upper floors and one basement floor by means of a deck slab. The objective is to develop and validate a finite element (FE) modelling approach for reasonable simulation of the observed first two specimens' behaviour. To this end the commercial software *ANSYS* is used. The FE modelling approach developed in this paper shall facilitate detailed parametric studies to be conducted in the future.

ZUSAMMENFASSUNG

Stahlbetonschubwände sind zur Aussteifung von mehrgeschossigen Gebäuden gegen Erdbebeneinwirkungen von zentraler Bedeutung. Ihre Tragwirkung und möglichen Versagensmechanismen sind jedoch komplex, insbesondere unter zyklischer Beanspruchung wie einem Erdbeben, und daher Gegenstand von weiterführender Forschung. Im Rahmen eines Forschungsprojekts an der Fachhochschule Nordwestschweiz (FHNW) wurden zu diesem Thema großmaßstäbliche Wandschubversuche durchgeführt [1], [2]. Dabei wurden vier Schubwände aus

Stahlbeton in Echtgröße unter zyklischer Beanspruchung untersucht, um die Effekte von Erdbebeneinwirkungen zu simulieren. Die Versuchskörper bilden auf idealisierte Weise den Einspannbereich einer Schubwand mit zwei Obergeschossen und einem Untergeschoss ab, die durch einen Deckenabsatz getrennt sind. Ziel ist die Entwicklung und Validierung eines Ansatzes nach der Finite-Elemente (FE) Methode, der das Last-Verformungsverhalten der ersten beiden Schubwände zuverlässig simulieren kann. Hierfür wird die kommerzielle Software *ANSYS* herangezogen. Der entwickelte numerische Ansatz soll in Zukunft Parameterstudien ermöglichen.

1. INTRODUCTION

1.1 MOTIVATION AND CONTEXT, APPROACH

A better understanding of the behaviour of reinforced concrete shear walls under earthquake actions is in the interest of the engineering community. In general, this allows more realistic performance assessment, which can lead to an efficient design of resistance to earthquakes. Especially as dealing responsibly with resources and reducing emissions become essential, but buildings need to meet ever growing earthquake standards, efficient earthquake engineering is needed.

Real-scale experiments bring extensive information about the behaviour of shear walls, but they are limited in numbers because of their time, material, and financial expense. The number of usable results is thus limited. However, this can be complemented through numerical analysis, provided that the numerical modelling approach is well validated to simulate the various physical effects characterizing different load resisting mechanisms possible for the structural elements. In this study, a numerical modelling approach for evaluating cyclic performance of shear walls under bending dominant in-plane loading is developed using the software *ANSYS* [1]. The paper presents the process underlying this development.

1.2 STATE OF THE ART

Shear walls are categorized as *squat* or *slender* shear walls. This study focusses mainly on the latter, which is used to describe structural concrete walls with a typically greater height than length. The in-plane behaviour under load of these structural members is expected to be dominated by bending with a span-to-depth ratio higher than 3, otherwise dominated by shear. For mainly bending-dominated elements, an efficient analysis can happen based on the *Euler-Bernoulli* beam theory by making use of the plastic hinge model by Paulay & Pristley [4].

More precision can be reached by considering effects such as the penetration of yield strains within the clamping element. Shear-dominated structural elements require more sophisticated methods. The application of the *tension-chord*-model to further developments of the *compression-field* theory by Vecchio & Collins [5], [6] delivers extended possibilities in computational structural analysis. Shear walls or girder webs can be properly assessed with this approach.

In contrast to squat shear walls, slender shear walls are less prone to sliding shear failures. Nevertheless, sliding can occur under certain conditions and in interaction with bending and normal forces [7]. Fig. 1 shows a typical example of sliding failure within a flexural compressive zone, in opposition to classical sliding as it would occur within a squat shear wall.

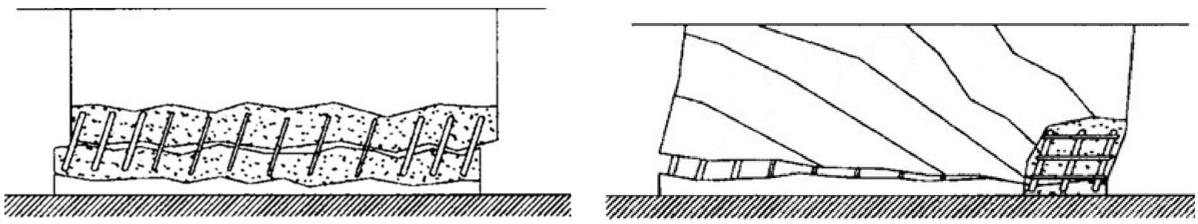

Fig. 1: Typical sliding shear failure (left) and sliding shear failure of a flexural compression zone (right) [8]

Regarding the behaviour of structural concrete under cycling loading, decisive aspects must be considered both for the concrete matrix and the reinforcement. Typically, concrete loses stiffness under cyclic loading [9]. This is observed both in compression and tension, as shown in Fig. 2. The respective softening behaviour doesn't change compared to monotonic loading for monodirectional cycling.

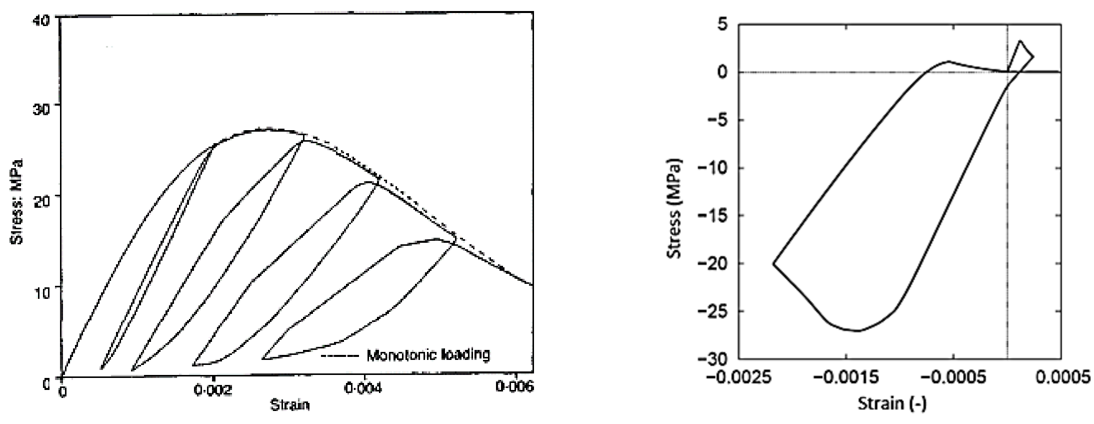

Fig. 2: Stress-strain behaviour in cyclic compression (left) and subsequent tension-compression (right) [1],[9]

In the case of change of direction, it is observed that tensile softening doesn't influence the compressive behaviour because of crack closure. On the other hand, compressive softening can impact the tensile capacity of concrete, as shown in the right curve of Fig. 2.

Reinforcement steel has the same cyclic properties as conventional steel, but also some specificities due its role in reinforced concrete and the bond behaviour. First, the shift of the yield limit after yielding in the opposite direction, also known as *Bauschinger-effect*, is a well-known phenomenon. Secondly, due to tensile strains being generally greater than compressive strains in reinforced concrete elements, the stress-strain-hysteresis is observed to shift in direction of the tensile strains, which can be observed as a shift to the right in the following Fig. 3.

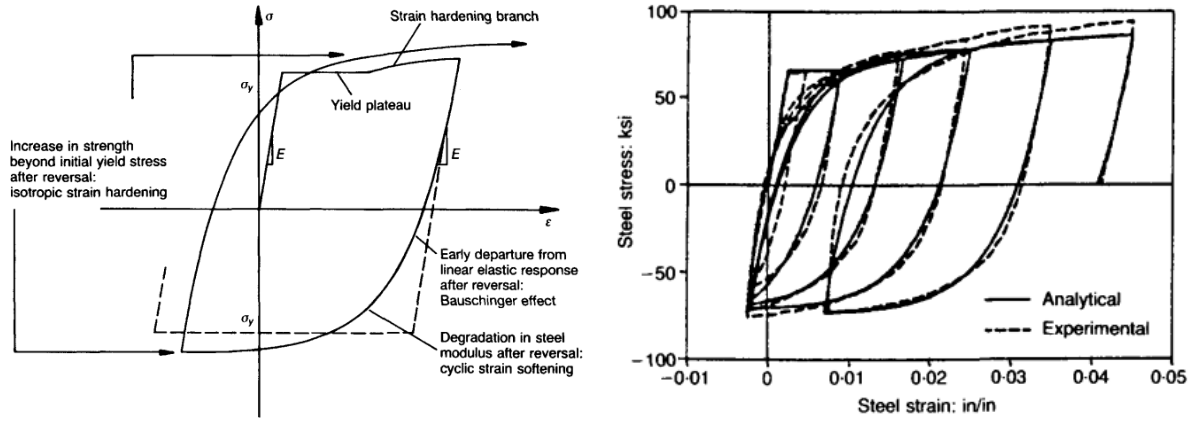

Fig. 3: Schematic and experimental hysteresis of reinforcement steel [9]

Typically, the hardening pattern of cyclic shear wall tests is dominated by the behaviour of the reinforcement, as displayed in Fig. 4. Regarding the unloading pattern, a phenomenon which is caused by the degradation of stiffness in concrete and is described as *pinching* of the load-displacement-curve is often observed.

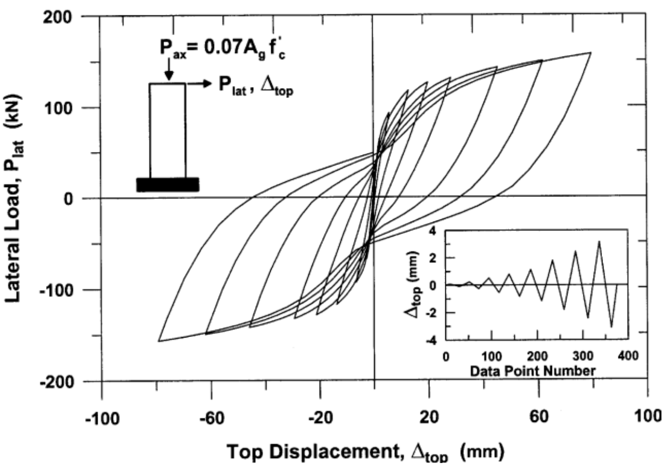

Fig. 4: Typical stress-strain hysteresis of a cyclic shear wall experiment [10]

1.3 FULL SCALE EXPERIMENTS

Within the research program, four full-scale experiments are conducted. The first two specimen, which provide the basis for the numerical simulation, only differ in their amount of reinforcement, as portrayed in Fig. 5. Wall 1 is designed to represent a wall of an existing building with a small reinforcement ratio. Wall 2 represents an earthquake wall with ductile boundaries at the edges of the wall. A special focus is set on the influence of the clamping part on the top load-displacement behaviour. In the clamping part a sliding shear failure cannot be excluded.

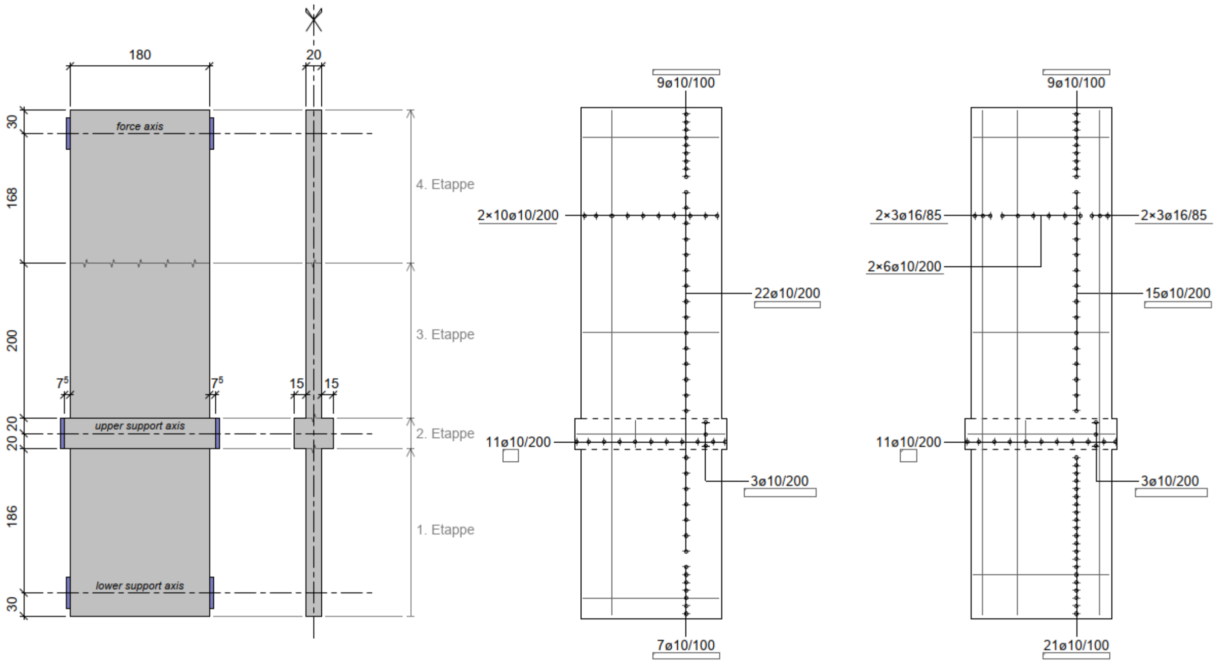

Fig. 5: Geometry and reinforcement scheme of shear wall 1 (left) and 2 (right) [1],[2]

In parallel to concreting, several samples were taken and then evaluated through cylinder compression tests and splitting tests. Subsequently, the first specimen was assigned with the strength class *C25/30*, the second with *C30/37*.

Table 1: Material properties of concrete

Description	Property	C25/30	C30/37
Elasticity-modulus	E_c	32000 N/mm^2	34000 N/mm^2
Poisson's ratio	v_c	0,2	0,2
Uniaxial compressive strength	f_{cm} or R_c	33 N/mm^2	38 N/mm^2
Biaxial compressive strength	$f_{c2c,m}$ or R_b	38 N/mm^2	44 N/mm^2
Tensile strength	f_{ctm} or T	2,6 N/mm^2	2,9 N/mm^2

Table 1 displays the most relevant material properties. Properties such as the biaxial compressive strength and nominal tensile strength were calculated according to the recommendations of the *fib Model Code* [11].

As the Reinforcement consists of bars ø10 mm and ø16 mm, various testing distinguished between diameters. The relevant parameters are displayed in Table 2. The yield strength f_{ym} is hereafter defined as $R_{P0,2}$, the 2 ‰ yield strength.

Table 2: Material properties of reinforcement steel

Description	Property	ø10	ø16
Elasticity-modulus	E_s	250000 N/mm^2	195000 N/mm^2
Poisson's ratio	v_s	0,3	0,3
Yield strength	f_{ym}	570 N/mm^2	520 N/mm^2
Proportional limit, start of multi-linear hardening	σ_0	425 N/mm^2	390 N/mm^2
Tensile limit	f_{um}	635 N/mm^2	660 N/mm^2
Ultimate strain limit	ε_{su}	70 ‰	120 ‰

The two wall tests showed the following results:

- Flexural deformation dominates in the cantilever and the clamping part of the wall. According to inclined cracking due to interaction of flexure and shear, the tension shift effect must be considered.
- A sliding failure occurred in wall 2. On the third load stage sliding displacements began to develop in the construction joint under the basement ceiling. The experiments show that a sliding failure does not only depend on the aspect ratio of the basement wall, which was equal for both specimens, but also from the shear force, which was more than twice as high for wall 2.

2. NUMERICAL APPROACH

The modelling approach adopted in this study is aligned with the studies conducted in the past at IWB [12],[13],[14],[15], University of Stuttgart. The discretization and material models adopted for concrete, structural steel and reinforcing bars is discussed in the following sections. A comparison of the numerical results with the test is discussed. Modifications essential in the numerical modelling approach are highlighted and considered to update the modelling approach.

2.1 MODELLING CONCRETE

The concrete volume was discretized as solid elements. The *Drucker-Prager* model implemented in ANSYS was used to define the constitutive relationship for concrete. It consists of a *Drucker-Prager* yield surface in compression which is defined through nonlinear hardening and transitions into an exponential softening law. In Tension, a *Rankine* failure surface considers the comparatively brittle behaviour of the material. The material parameters for the model are summarized in Table 3. Detailed documentation about the concrete model is available in the *ANSYS Mechanical APDL* Material Reference [1].

Table 3: HSD constants for the Drucker-Prager concrete model ($\varepsilon_{c1,pl}$, $\varepsilon_{c2,pl}$ and G_{ft} according to [11])

Description	Parameter	C25/30	C30/37
Plastic strain at uniaxial compressive strength	$\varepsilon_{c1,pl}$ or κ_{cm}	0,00117	0,00118
Plastic strain at transition from power law to exponential softening	$\varepsilon_{c2,pl}$ or κ_{cu}	0,00268	0,00261
Relative stress at start of nonlinear hardening	Ω_{ci}	0,4	0,4
Residual relative stress at κ_{cu}	Ω_{cu}	0,8	0,8
Residual compressive relative stress	Ω_{cr}	0,1	0,1
Mode I area-specific fracture energy	G_{ft}	0,137 N/mm	0,141 N/mm
Residual tensile relative stress	Ω_{tr}	0,1	0,1

The concrete body is discretized through the element *SOLID185*. In this study, a mesh of linear tetrahedra with an element size of h = 30 mm was chosen.

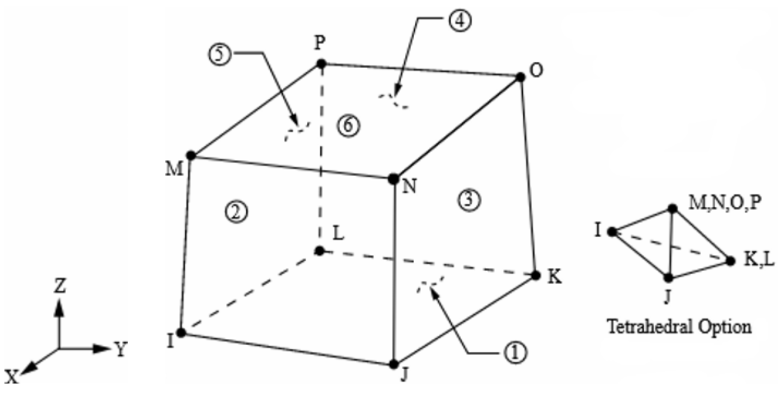

Fig. 6: Options of the element formulation SOLID185 [1]

2.2 MODELLING OF REINFORCEMENT

The reinforcing bars geometries were modelled using curves and the corresponding cross sections were associated. The rebars were meshed as 2 node line elements. The element *REINF264* was used for modelling the reinforcements. The constitutive behaviour of the reinforcing steel is defined through using a multilinear hardening law. The initial yield stress, characterized as σ_0 in Fig. 7, is chosen as 75% of the mean yield strength f_{ym} since the same approximately corresponds to the linear elastic limit as observed in the rebar testing results, the latter being defined as $R_{P0,2}$. To enable cyclic action, kinematic hardening is chosen over isotropic hardening. This considers the particularities of highly nonlinear hysteretic behaviour of steel such as the previously introduced *Bauschinger-effect*.

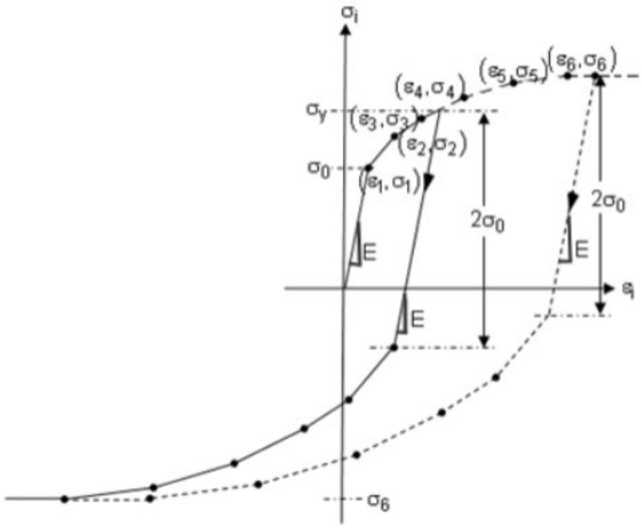

Fig. 7: Schematic stress-strain diagram with multilinear kinematic hardening [1]

2.3 BOUNDARY CONDITIONS AND LOADING

In the experiments, a horizontal displacement is applied at the top of the clamped shear walls and the resulting reactions in the actuators are measured. Whereas in case of the monotonic simulation a typical 3-point-loading is sufficient, cyclic simulation requires direction-specific boundary conditions, as shown in Fig. 8. To maintain the same mechanical state, *compression-only* supports are used in the clamping area, both on the deck slab and the basement. The top displacement, however, is still applied on one side to keep a simple loading procedure in *ANSYS Mechanical*. Because of the tensile stresses which occur with this configuration, defining of an upper body with only linear elastic properties is needed.

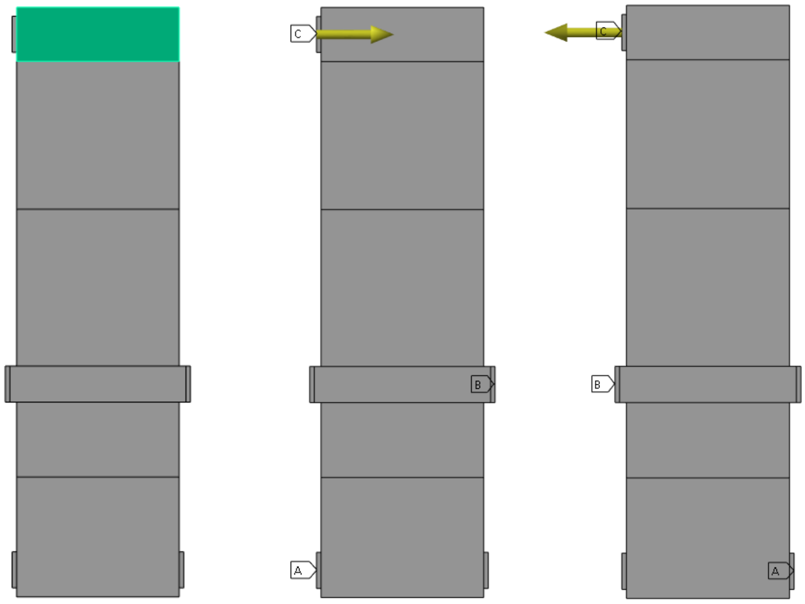

Fig. 8: Upper body with linear elastic characteristics (in green) and direction-specific alternating supports

A perfect bond condition between the reinforcement and the surrounding concrete matrix is assumed because of the absence of overlapping and the capacity of the reinforcement elements to consider tension stiffening. The anchorage zone is situated at the very top and very bottom of the shear walls, which maintained an entirely elastic state during the experiments. Thus, no explicit definition of bond behaviour is needed in this context. However, an approach including a more precise definition of bond behaviour could be necessary when studying scenarios where failure of specific reinforcement bars is occurring, and thus new anchorage configurations are formed.

3. RESULTS AND COMPARISON

3.1 MONOTONIC ANALYSIS

The specimen assessed during the first experiment has a comparatively low reinforcement ratio of $\rho_l = 4{,}4$ ‰ which limits its bending capacity. Despite the aspect ratio of approximately 1:2, its behaviour was nearly entirely bending-dominated, and the consequent failure induced by a strongly localized crack pattern with rupture of the reinforcement. The low amount of reinforcement also limits the specimen's ductility, as the cracked area is centered around the deck slab and does not extend past half the upper and lower panels' height.

The goal of the monotonic simulation is the validation of the model's ability to determinate reasonably realistic load-displacement behaviour and failure mechanism.

As displayed in Fig. 9, the simulation results of the first shear wall fit as an envelope curve for the experimental hysteresis. A similar load capacity of F_{max} = 250 kN is reached. The maximum displacement of u_{max} = 55 mm reached in the monotonic simulation cannot be directly compared with the cyclic result. However, comparison can be made with an analytical estimate through the plastic hinge approach [4] which results in a similar value of u_{max} = 50 mm.

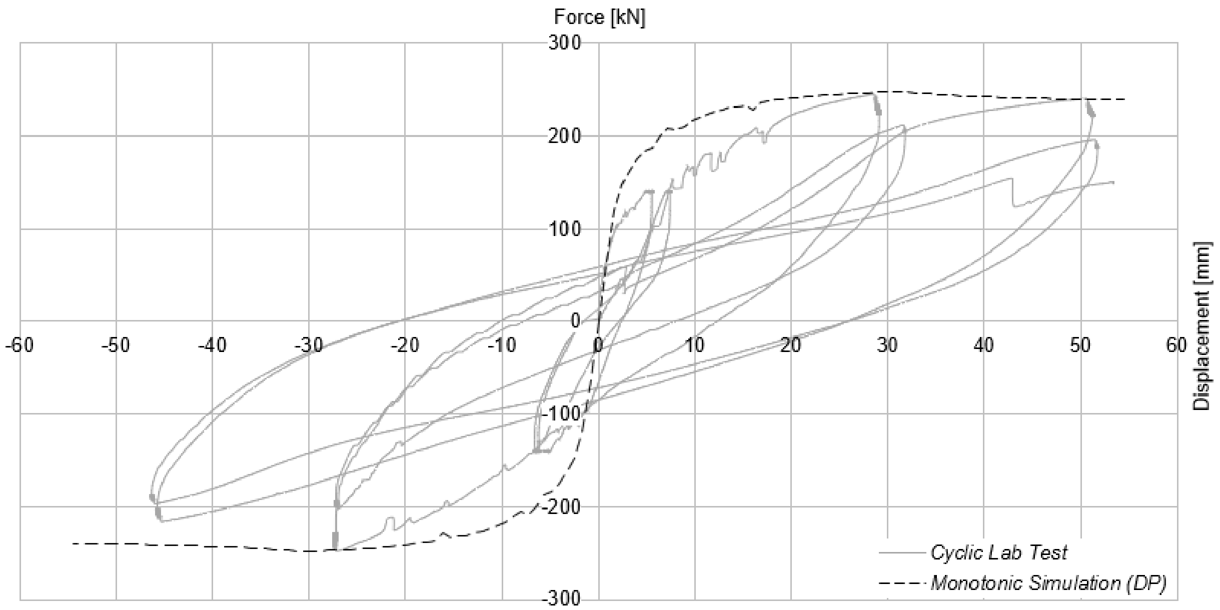

Fig. 9: Monotonic simulation of the first wall (Drucker-Prager concrete model), experimental curve [1],[2]

Regarding the failure mechanism, a further look in the FE results is necessary. The highest load level can indeed be observed at a displacement of around 30 mm. After this, a slight load decrease can be observed until the maximum strain of the reinforcement of ε_s = 70 ‰ is reached, which occurs at a displacement of 55 mm.

Fig. 10 shows that the increase of the reinforcement strain is very localized. The corresponding increase of stress also extends to a relatively restricted area, which corresponds to the cracking area. Fig. 11 displays a comparison between the simulated and experimental crack pattern. The expansion of the cracking area is nearly equal and similar crack inclination and spacing can be observed.

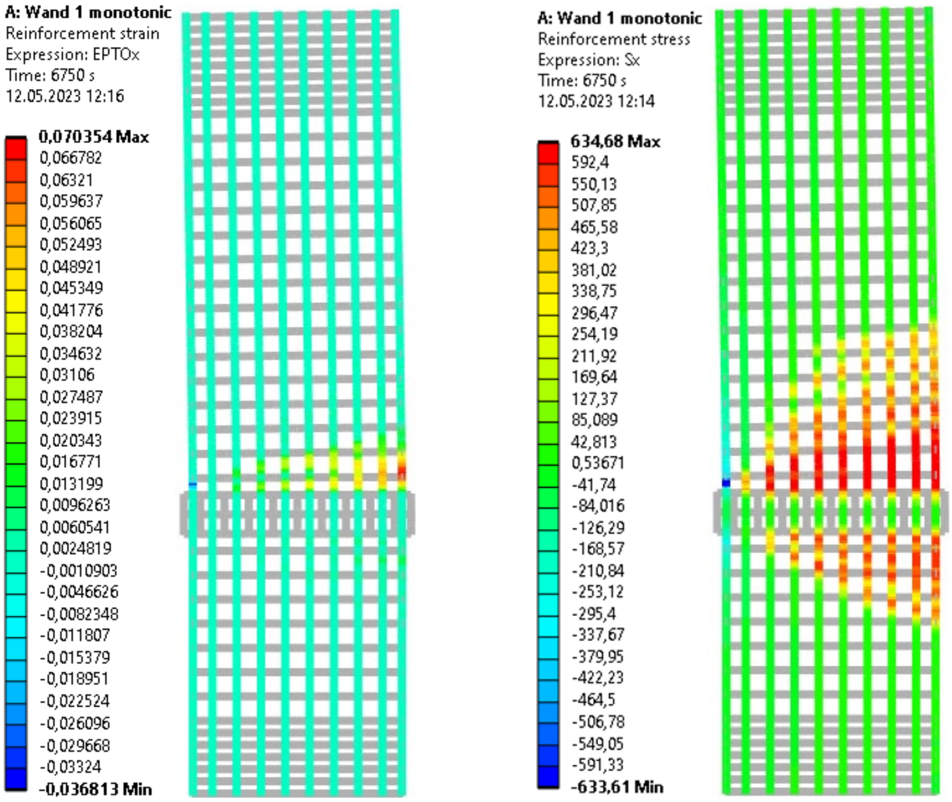

Fig. 10: Maximal total strain (left) and corresponding stress (right) in the reinforcement cage

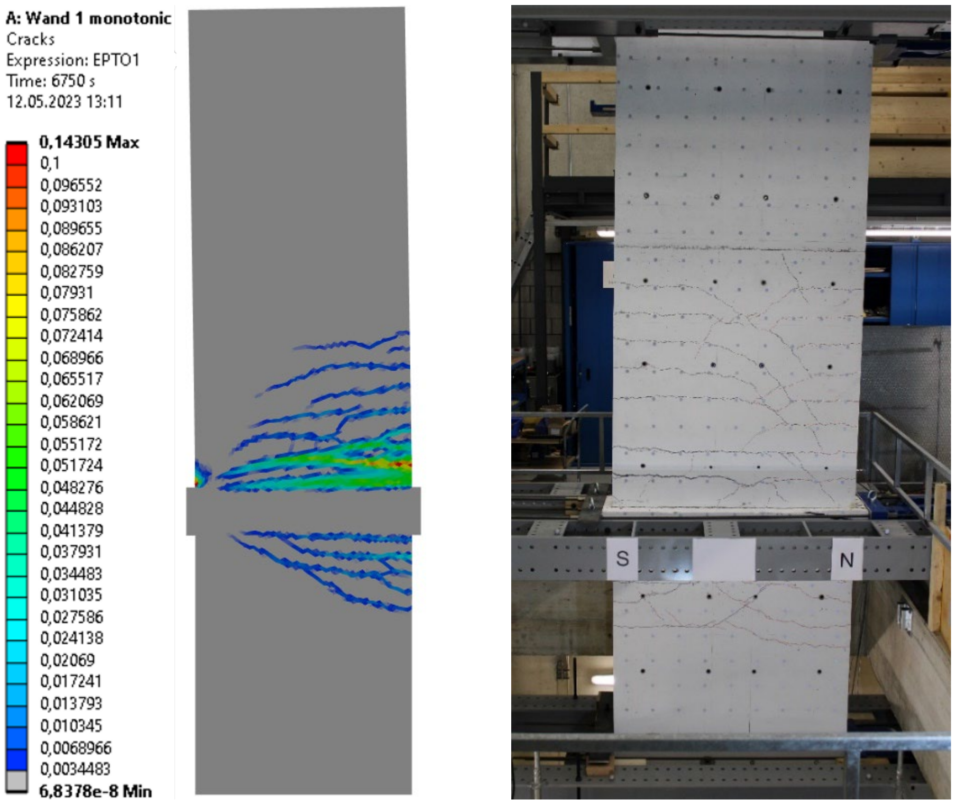

Fig. 11: Crack pattern, simulated (left) and experimental (right) [1],[2]

The monotonic results are thus sufficiently close to the experimental results to proceed to a cyclic simulation.

3.2 CYCLIC ANALYSIS

The main goal of the cyclic simulation is an estimation of the hysteretic behaviour and energy dissipation characteristics in addition to other features in a monotonic analysis. The cyclic simulation using the adopted modelling approach is shown in Fig. 12. The modelling approach is unable to correctly represent the hysteretic behaviour in a realistic manner, although the other characteristics are simulated reasonably well. It was identified that the *Drucker-Prager* concrete model is incapable to consider the effect of cyclic damage in concrete. It is a plasticity-based model and does not consider any damage parameters required to model the corresponding effects.

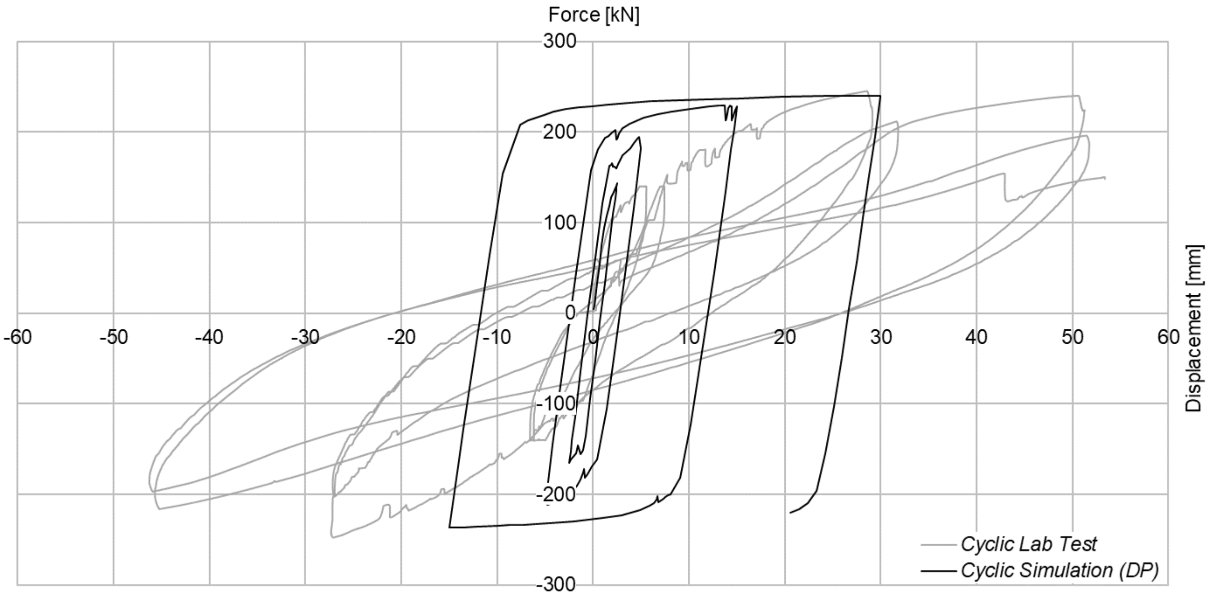

Fig. 12: Cyclic simulation of the first specimen (Drucker-Prager concrete model), experiments from [1],[2]

For successful cyclic simulation, the concrete model should consider the effects of cycling on concrete stress strain behaviour as discussed earlier. To this end, the *Coupled Damage-Plasticity Microplane Model* developed by Zreid & Kaliske [16] is used. This model uses a *Drucker-Prager* surface for definition of the plasticity as in the case of the *Drucker-Prager* model. Additionally, the model uses a modified microscopic free-energy function to include damage, which enables considering cyclic effects.

Furthermore, a *tension-compression split* is implemented to consider the transition of the stress state between tension and compression [1].

The model is thus capable to simulate the cyclic stress-strain behaviour of concrete as discussed in Fig. 2.

The choice of the model parameters requires model calibration on a reduced model as on the single element scale which typically does not account for nonlocal interaction. A set of calibrated parameters to match the stress strain behaviour expected for the concrete properties in the presented study is shown in Table 4.

Table 4: Coupled damage-plasticity microplane model parameters

Description	Parameter	C25/30	C30/37
Intersection point abscissa between compression cap and *Drucker-Prager* yield function	σ_V^C	-25 N/mm^2	-29 N/mm^2
Ratio between the major and minor axes of the cap	R	2	2
Hardening material constant	D	1*10^5 N^2/mm^4	1,5*10^5 N^2/mm^4
Tension cap hardening constant	R_T	1	1
Tension damage threshold	γ_{t0}	0	0
Compression damage threshold	γ_{c0}	1,0*10^{-5}	1,0*10^{-5}
Tension damage evolution constant	β_t	4500	4500
Compression damage evolution constant	β_c	3000	3000
Non-local interaction range parameter	c	2500 ($c \approx h^2$)	2500 ($c \approx h^2$)
Over-non-local averaging parameter	m	2,5	2,5

In contrast to the *Drucker-Prager* concrete model, the implementation of the *Coupled damage-plasticity microplane model* requires two additional degrees of freedom per node. This is due to the implicit gradient regularization scheme the model uses via a nonlocal field [3]. Consequently, the elemental formulation is switched to the element *CPT215*, in its hexahedral option. The average element size of 50 mm was used for considering definition of the nonlocal parameter c.

Changing the concrete model to the *Coupled Damage-Plasticity Microplane Model* to consider the effects of cycling yields equivalent results for monotonic loading (relative to those discussed above). Furthermore, the modelling approach could simulate the load-displacement hysteresis in a reasonably realistic manner compared to the experimental result, as shown in Fig. 13. Convergence could be achieved through 3 cycles.

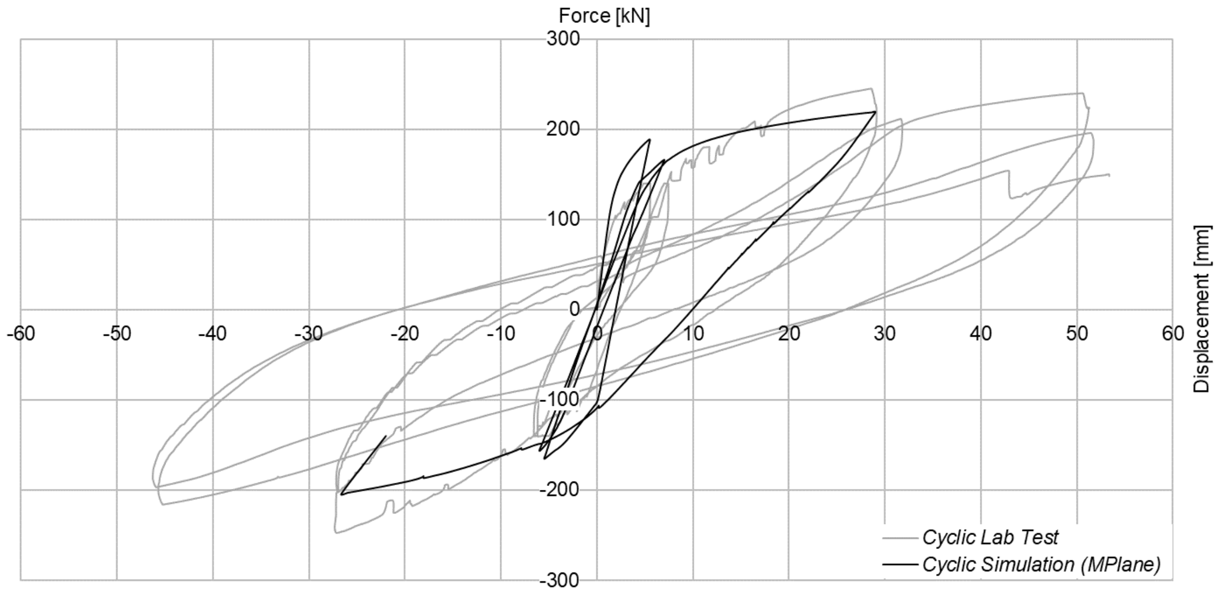

Fig. 13: Cyclic simulation of the first specimen (MPlane concrete model), experiments from [1],[2]

4. FINAL REMARKS

A numerical modelling approach for simulating the cyclic effects of shear walls was discussed in this paper. Given the time intensive efforts for performing cyclic analyses, it is often a case that monotonic analyses results are compared with cyclic test results. In this study, it was shown that although monotonic results compare well with test results, it is not necessary that the cyclic results will also follow. This was shown to be depending on the capability of the concrete material models to simulate the different effects of damage.

The *Drucker Prager* Model was observed to provide reasonably realistic comparison of monotonic analyses with cyclic test results. However, since the model does not account for the damage within the cyclic stress-strain relation for concrete, the simulation of hysteretic behaviour was rather erroneous. The *Coupled Damage-Plasticity Microplane Model* was however able to provide reasonable results for both monotonic as well as cyclic analyses.

The microplane-based model is a rather recent development are needs to be tried out extensively to understand its potential for simulating different application. The method certainly has some great potential, both in research as a complement to experiments and in practice to perform detailed seismic assessment of both existing and new structural elements.

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