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Experimental versus numerical response of RC walls subjected to earthquake loading

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Abstract

Reinforced-concrete walls have been frequently used as a bearing system for seismic loads in the last half of the century. They partially replaced the traditional European way of building with brick walls, and they are most often used for the construction of residential and business buildings of medium height. The aim of this study is to numerically examine the influence of different reinforcement layouts on the load bearing capacity and ductility of slender shear walls. Three walls experimentally tested at ETH-Zürich under in-plane static cyclic action were used for comparison with finite element models. The walls were designed for different ductility classes according to Eurocode 8. Two types of modelling techniques were made employing SAP2000nl. A frame model with nonlinearity localized in a plastic hinge was made using Takeda and Pivot hysteretic rules. A nonlinear layered shell model was applied using Mander's definition of confined and unconfined concrete while reinforcement was assumed as hardening plasticity. Advanced numerical analysis on 2D models was performed using DIANAFEA software package assuming smeared cracking approach. Behaviour of concrete was described with total strain rotating crack constitutive law while von Mises yield criterion defines the stress-strain relationship of embedded reinforcement. Plastic hinge models are simple, require short calculation time and predict load-bearing capacity quite well. Shell layered model was mesh dependent and extremely time-consuming resulting in premature failure or mismatch in energy dissipation with respect to experimental findings. Refined numerical modelling using powerful solvers in DIANA proved to be the most successful regarding computational time and feasible mesh densities. Good agreement of dissipated energy, load bearing capacity and crack layout with test results was obtained.

Key words: reinforced-concrete wall, earthquake, capacity design method, nonlinear analysis

1 Introduction

Reinforced-concrete walls have been used frequently as a bearing system for seismic loads in the last half of the century. They partially replaced the traditional European way of building with brick walls, and they are most often used for the construction of residential and business buildings of medium height. Generally, they possess sufficient resistance to withstand the earthquake loads and adequate stiffness to limit the horizontal floor drifts. Slender walls (with a height to length ratio of h/l>2) are mainly loaded in bending and as such are suitable for ductile shaping. Ductile shaping of reinforcement and proper choice of concrete geometry are the basic features of capacity design method, whose main task is to avoid a sudden, unannounced failure. Even for moderate earthquake intensities, some parts of structures exhibit non-linear behaviour. It is important to develop acceptable plastic mechanisms and to achieve energy dissipation mostly by flexure, which is obtained by the formation of plastic hinges at the bottom of the wall. Experiments on scaled or even full-scale models subjected to dynamic or cyclic static loading are a reliable way to examine seismic response of reinforced-concrete walls. Series of such experiments on scaled walls were performed at ETH-Zürich [1,2]. The specimens were designed according to contemporary European seismic codes respecting the rules of capacity design method.

The 6 wall specimens WSH1 to WSH6 were tested under static-cyclic action simulating the lower half of a wall of a 6-storey prototype building as shown in Fig. 1. The test set-up is shown in Fig 2. The wall footing is rigidly connected to the strong floor, and the wall is laterally guided by low friction sliding bearings at the levels of the first and second floor. In reality, the seismic forces acting at the floor levels of the prototype building are distributed nearly triangularly with height. However, for testing purposes, the seismic forces are concentrated into a resultant F_k at one level near the top of the wall and simulated by a two-way acting hydraulic actuator (Fig. 2).

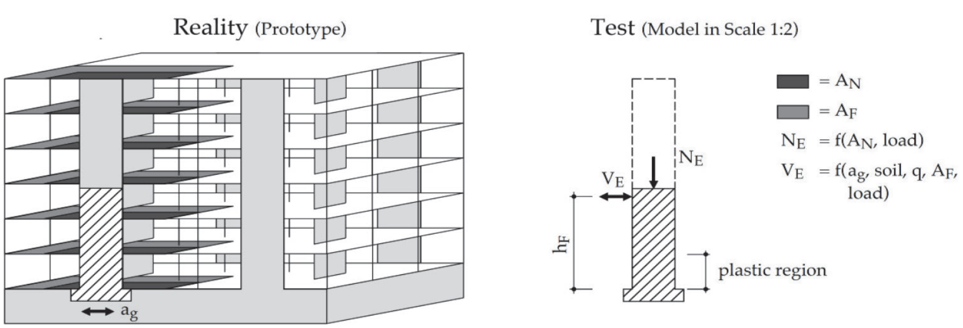

Figure 1. Real building (left) and scaled model (right) [1]

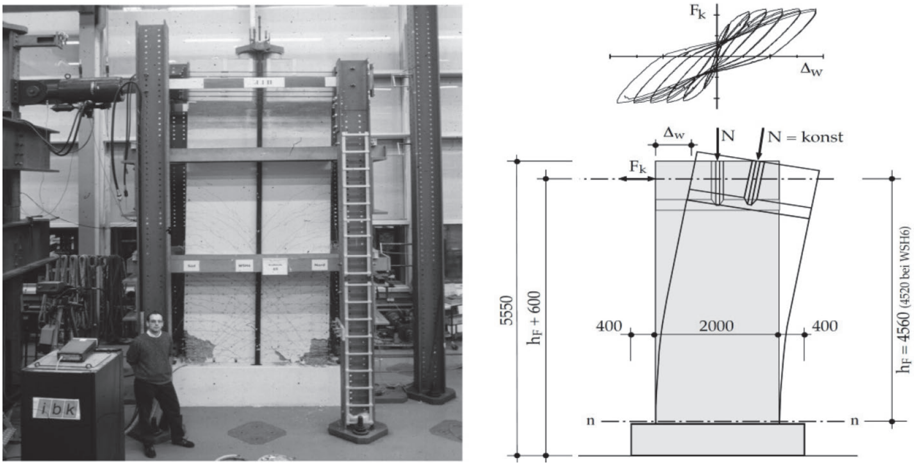

Figure 2. Front view of the test set-up [1]

Tests on concrete and reinforcement were performed to determine mechanical properties of the materials used in the experiment. The properties of concrete for three typical walls (WSH1, WSH3, WSH6) are provided in Table 1 and the properties of reinforcement are given in Table 2 [1].

Table 1. Mechanical properties of concrete

Model	**WSH1**		**WSH3**		**WSH6**	
Group	1-4	5-7	1-4	5-7	1-4	5-7
Age [day]	59	48	192	178	227	216
Density ρ_c [kg/m^3]	2397 ±28	2399 ±22	2381 ±18	2373 ±15	2383 ±22	2384 ±8
Cylinder compression strength f'_c [MPa]	45.0 ±2.1	46.0 ±0.5	39.2 ±2.2	39.6 ±2.1	45.6 ±0.3	44.7 ±1.2
Ultimate deformation ε'_{cu} [‰]	1.96 ±0.03	2.10 ±0.17	1.81 ±0.20	2.04 ±0.26	1.99 ±0.18	2.02 ±0.13
Elasticity modulus E_c [GPa]	44.4 ±5.1	41.4 ±5.5	35.2 ±1.5	36.2 ±3.8	36.9 ±0.7	38.0 ±0.3

Table 2. Mechanical properties of reinforcement

$Ø_{nom}$ [mm]	$R_{p0,2}$ [MPa]	R_m [MPa]	$E_{s,nom}$ [GPa]	$f_{yi,stat}$ [MPa]	f_{ti} [MPa]	$E_{s,eff}$ [GPa]
12	576.0 ±2.6	674.9 ±1.8	210.3 ±4.4	542.2 ±3.6	663.2 ±2.1	206.6 ±4.2
8	583.7 ±5.5	714.4 ±5.1	219.5 ±2.3	539.6 ±4.5	689.7 ±4.7	211.9 ±2.3
6	518.9 ±13.8	558.7 ±6.7	210.3 ±2.3	501.6 ±13.3	562.7 ±7.1	211.8 ±2.4

The specimen dimensions were 2.00 m (horizontal wall length l_w) x 0.15 m (wall width b_w) x 5.55 m (total height including footing). The vertical reinforcement in the web region consisted of bars with a diameter of either 6 mm or 8 mm and a horizontal spacing of 12.5 cm or 14 cm. The horizontal web reinforcement consisted of bars Ø 6 mm and a vertical spacing of 15 cm (minimum reinforcement). In the boundary regions 6 vertical bars had a larger diameter (Ø up to 12 mm) than the vertical bars in the web region. With the exception of wall WSH4 these bars were stabilized against buckling by additional closed ties and S-shaped ties with a diameter of 6 mm or 4.2 mm and a vertical spacing s_h of 5 cm or 7.5 cm, acting also as confinement for the concrete compression zone (Fig. 3).

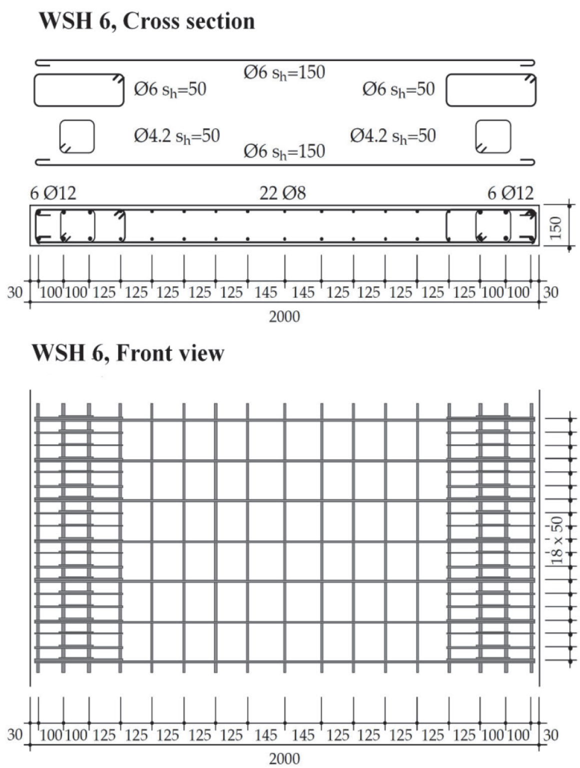

Figure 3. Reinforcement drawing for Wall 6 - Cross Section and front view [1]

By measuring the applied force and displacements of the walls, the F_k-Δ_w force-displacement curves were determined for all the specimens separately. Besides this, wall deformations, formation of cracks and crack width were monitored. This work is focused on force-displacement curves i.e., determining the load bearing capacity and energy dissipation of the walls. Example of the force-displacement curve, and crack formation from the experimental research can be seen in Fig. 4.

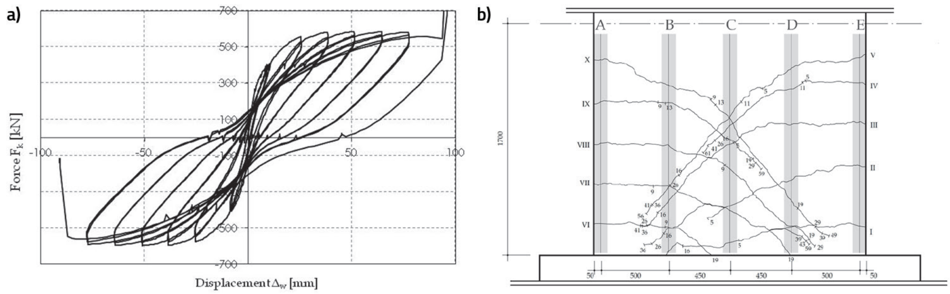

Figure 4. a) Force-displacement curve, b) crack pattern for Wall 6 [1].

2 Numerical models

Numerical modelling was performed using SAP2000nl (link element, nonlinear shell elements) [3] and DIANA FEA (regular plane stress elements) [4].

2.1 Numerical analysis in SAP2000

The walls tested in SAP2000nl were modelled in two basic ways [5]. The first way is by using a "Frame element", i.e., a beam element with the wall of a cantilever statical system. The second method of modelling was made with "Layered Shell" elements, i.e., a surface element made of "layers" of concrete and reinforcement. Takeda and Pivot models were used for modelling [6, 7] hysteretic behaviour localized in the plastic hinge at the bottom of the cantilever. For these models the length of the plastic hinge was an essential value. First, the moment – curvature diagram was calculated discretizing the section into fibers using Section designer subprogram and Mander confined and unconfined concrete models [8]. Then, the moment-curvature curves had to be transformed into moment-rotation curves by multiplying the curvature with the length of the critical region, $\varnothing = \kappa \cdot l_p$, in order to be used in the program. In this work, the length of the critical region (plastic hinge) was determined observing the formation of cracks from experimental tests as well as using recommendations from literature [9-11]:

- $l_p = l_w$,
- $l_p = 0.5 \cdot d + 0.05 \cdot a$,
- $l_p = 0.25 \cdot d + 0.075 \cdot a$

where, l_p is height of the plastic region, h is height of the cross section, d is static height and a is a distance between the zero point of the moment diagram and the support axis (critical section). For Wall 1 the length of the plastic hinge was 80 cm, which best corresponds to the third recommendation and amounts to less than a half wall length. As a comparison, it was found that the length of the critical region is equal to the beam depth for reinforced concrete beams investigated in [12].

For the Takeda model, aside from moment-curvature relationship, no additional coefficients need to be provided for definition of the hysteretic behaviour. Although the model is simple, it cannot describe the energy dissipation of reinforced concrete structure well. Pivot model offers more possibilities for description of structural behaviour. However, the choice of parameters is not simple and they should be calibrated with respect to experiment. Using the experimental results, coefficients α and β that define the so-called pivot points and define the shape of the curve in the Pivot model, were calibrated iteratively.
As an example, specific values for Wall 1 are α_1 = 1.6, α_2 = 1.5, β_1 = 0.5 and β_2 = 0.7. It can be seen in Fig. 5. that this model shows much better agreement. However, it is clear that without prior knowledge and analytical experience with similar structures it is difficult to apply the Pivot hysteretic model for response prediction. Shell layered model resulted in hysteretic response quite different from the experimental (Fig. 5).

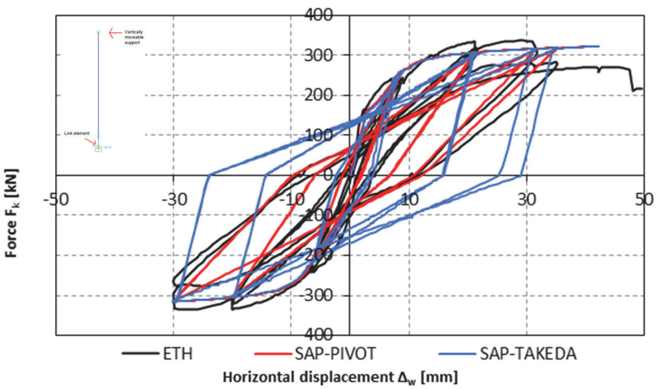

Figure 5. Comparison of F_k-Δ_w diagrams for Wall 1 (Takeda and Pivot model)

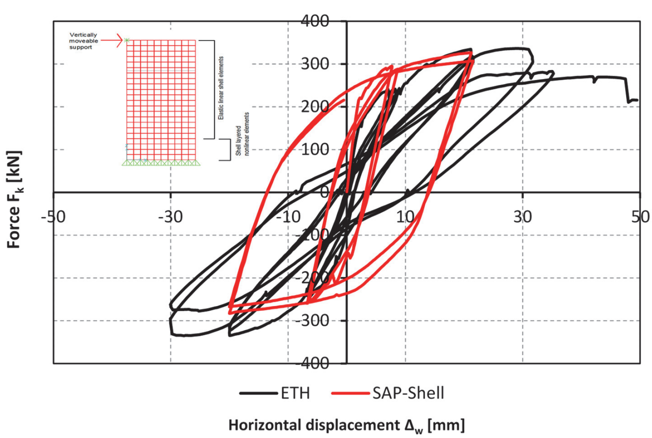

Figure 6. Comparison of F_k-Δ_w diagrams for Wall 1 (shell layered)

2.2 Numerical analysis in DIANA FEA

Concrete was modelled using Total strain rotating crack model [4], embedded reinforcement was assumed von Mises with isotropic hardening. The constitutive model of concrete used is based on total strain and it was developed along the lines of the Modified Compression Field Theory, originally proposed by Vecchio and Collins [13]. Like the multidirectional fixed crack model, the total strain-based crack models follow a smeared approach for the fracture energy. The three-dimensional extension to this theory is proposed by Selby and Vecchio [14]. The hypoelastic concept is used for description of the stress as a function of the strain, with a modification which includes secant unloading. Stress and strain are coaxial, which means that the stress-strain relations are evaluated in the principal directions of the strain vector. There are two approaches regarding the crack model which pertain to calculation of stress-strain law in a fixed (fixed upon cracking) or a rotating (cracks can continuously rotate with the principal directions of the strain vector) coordinate system. The parameters of the model are given in Table 3 and Table 4.

Table 3. Mechanical properties of concrete

Material	Unconfined concrete	Confined concrete
Elasticity modulus [N/mm^2]	38000	38000
Poisson coefficient	0.2	0.2
Mass density [T/mm^3]	$2.384 \cdot 10^{-9}$	$2.384 \cdot 10^{-9}$
Crack orientation	rotating	rotating
Tensile curve	linear ultimate crack strain	linear ultimate crack strain
Tensile strength [N/mm^2]	4	4
Ultimate strain	0.0029	0.0029
Compression behaviour	fib Model for Concrete 2010	fib Model for Concrete 2010
Compressive strength [N/mm^2]	45	63
Strain at ultimate stress	0.002	0.006
Ultimate strain	0.0023	0.013
Reduction model due to lateral cracking	no reduction	no reduction
Confinement model	no increase	no increase

Table 4. Mechanical properties of reinforcement

Material	Reinforcement Ø8	Reinforcement Ø12
Elasticity modulus [N/mm^2]	219500	210000
Plastic hardening	plastic strain-yield stress	plastic strain-yield stress
σ-ε diagram	0 583 0.124 715	0 550 0.108 675
Hardening hypothesis	Strain hardening	Strain hardening
Hardening type	Isotropic hardening	Isotropic hardening

The model is shown in Fig. 7, the numerically obtained hysteretic curve and comparison with the experimental results are given in Fig. 8. The cracking pattern for different stages is provided in Fig. 9.

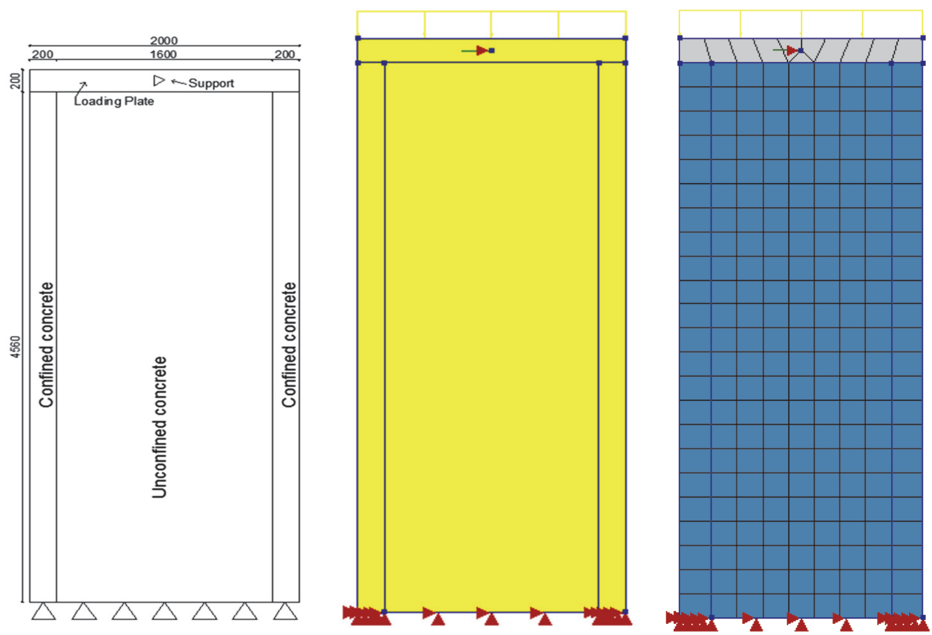

Figure 7. Numerical model

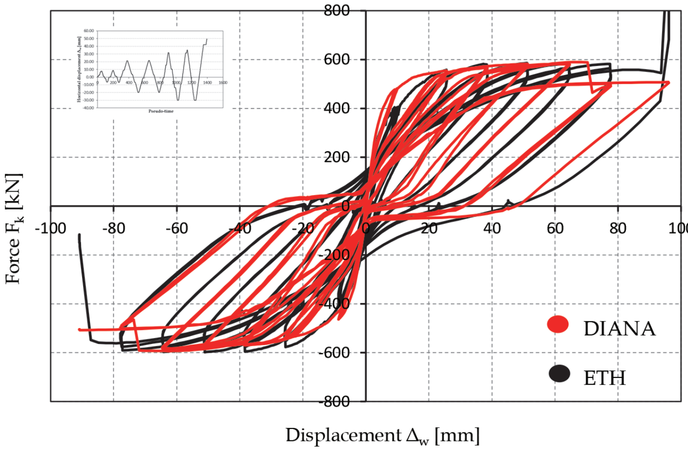

Figure 8. Comparison of response: experiment and FE model

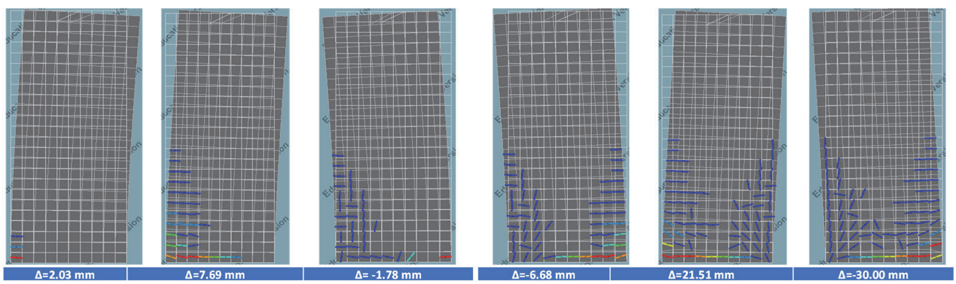

Figure 9. Cracking pattern for different stages of the loading program

3 Conclusion

The establishment of an appropriate mathematical model for the analysis of an engineering problem is to a large degree based on sufficient understanding of the problem under consideration and a reasonable knowledge of the finite element procedures available for solutions. This observation is particularly applicable in nonlinear analysis because the appropriate nonlinear kinematic formulations, material models and solution strategies need to be selected.

Accurate results using a beam model can be obtained if we use a Pivot model of hysteresis behaviour for the link element. Using the coefficients that define the position of the characteristic points on the hysteresis diagram, the model can describe the behaviour from the test very well; for example, dissipated energy is much more similar to that of the test. However, the key disadvantage is that such a model is not predictive. Namely, the mentioned coefficients of the model are determined solely by using items from the results of the experimental test itself. In engineering practice these "finished" results are exactly what we are trying to get, and in this sense this model is useless. The model can be useful in the case of very structures where we expect similar behaviour, and we have experimental test results for some of these structures. Generally speaking, a beam model can describe the load-bearing capacity of a structure, requires a short calculation time, is easily formed and is nicely interpreted using the concept of stress resultants (sectional forces), and therefore can serve as a good engineering basis for modelling these kinds of walls. The disadvantage of the model is the inability to perform more complex analyses and the inaccuracy of the results regarding the decrease in stiffness and energy dissipation, as well as the difficulty in determining the height of the plastic joint.

In DIANA, numerical modelling was done with regular plane stress elements. The diversity of material modelling capabilities and the ease of defining the finite element mesh make it practical and useful for engineering tasks. The analysis was performed on models with different finite element dimensions (10 x 10 cm, 20 x 20 cm, 40 x 40 cm, and 80x80cm). All calculations were successfully completed using powerful solvers, and the longest (for Wall 6, 12 load cycles, about 2600 steps) was no longer than 120 minutes. Relatively simple models for concrete and reinforcement were selected, where it was necessary to define strength and deformation values without additional damage parameters. However, with this model, very good agreement with the test results was achieved for the load capacity as well as for the formation and propagation of cracks. The model showed good agreement on dissipated energy for Wall 6, while for Wall 1 there was some difference due to higher rigidity at unloading. Initial stiffness is the only parameter that does not have a nice matching with experimental tests, which might be due to slight deformation of the footing in the experiment itself.

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