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NONLINEAR FINITE ELEMENT ANALYSIS OF REINFORCED CONCRETE SLIT WALLS WITH ANSYS (I)

BY

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Abstract. Structural reinforced concrete walls represents a system that provides lateral resistance, high stiffness and strength to a building. Because the energy dissipation is made only by the base of the structural walls, they do not exhibit ductile and redundant behavior. The structural reinforced concrete wall energy dissipator, named and *structural slit wall with shear connections*, remove some of the problems encountered with ordinary structural walls. Yielding of shear connections in this wall may cause increase in energy dissipation, forming a structural damper that is based on structural passive control. The overall ductility of the structure increases, considering the energy dissipation solution, resulting a supplementary safety for the building. The aim of these solutions is to create an ideal structure for tall multi-storey buildings, that has a rigid behavior at low seismic action and turns into a ductile one in case of a high intensity earthquake action.

In this paper, the static nonlinear analysis on energy dissipator wall with shear connectors is performed and the influence of the elasto-plastic behavior of the shear connections is evaluated. This type of structural wall is compared to an ordinary solid wall. The proposed solutions for increasing ductility are viable and easily to use in constructions practice. For the analysis of the proposed structural wall was used the finite element method; in this way is simulated accurate and realistic the behaviour of the reinforced concrete walls.

Key words: reinforced concrete slit walls; ductility; finite element method; nonlinear behaviour; lateral strength; pushover analysis.

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1. Introduction

Nonlinear finite element analysis helps researchers to conduct more detailed investigations on the behavior of reinforced concrete structural elements. Several three-dimensional finite element structural models are proposed in this paper in order to predict the nonlinear behavior of reinforced concrete slit walls energy dissipators with shear connections (Fig. 1). Numerical simulations are carried out with finite element software ANSYS 12.

In recent years, using ANSYS finite element software, many research works have been performed successfuly to simulate the seismic behavior of reinforced concrete elements (beams, walls, columns, etc.). These studies show that ANSYS can precisely simulate concrete, results shows a very good accuracy being close to the experimental ones (Raongjant & Jing, 2008).

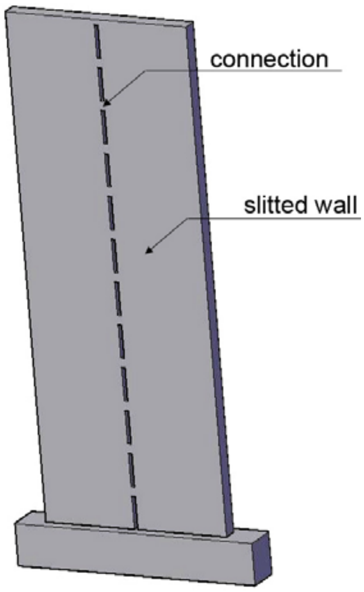


Fig. 1 – Reinforced concrete slit wall with shear connections.

Reinforced concrete walls are important structural elements that are placed in multistorey buildings from seismic zones, because they have a high resistance to lateral earthquake loads. Reinforced concrete structural walls must have sufficient ductility to avoid brittle failure under the action of strong lateral seismic loads. For the design of a ductile structural wall it is desirable that yielding of flexural reinforcement in plastic hinge region, normally at the base of the wall, would control the strength, inelastic deformation and energy dissipation. To enhance ductility, the concrete in the compression zone of the shear wall should not fail *prior* to the yielding of the flexural reinforcement

(Kheyrodin & Naderpour, 2008). By placing vertical slits in the wall structure a substantial increase in ductility is obtained. With this solution the degradation in the shear wall are greatly reduced, potential plastic zone is positioned along the height of the wall and energy dissipation is achieved by the crushing of the reinforced concrete shear connections (Kwan *et al.*, 1999). The plastic hinge formation furnishes to the structure kinetic energy dissipation capacity, but also constitutes a state of structural damage. A performance-based design will ensure the life safety and viable rehabilitation from economical point of view to a building subjected to a major earthquake.

2. Finite Element Analysis by ANSYS

2.1. Concrete Modelling

Concrete is a quasi-brittle material and has different behaviour in compression and tension. Development of a model for the behaviour of concrete is a challenging task for researchers. The Solid 65 element was used to model the concrete. This element has eight nodes with three degrees of freedom at each node – translations in the nodal *x*-, *y*- and *z*-directions. This element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. The geometry and node locations for this element type are shown in Fig. 2.

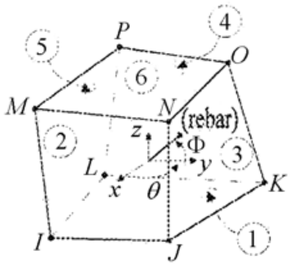


Fig. 2 – Solid 65 geometry.

The Solid 65 element requires linear isotropic and multilinear isotropic material properties to properly model concrete as can be seen in Table 1.

Table 1
Material Properties for Solid 65

Linear Isotropic (C20/25)		
E_c	3E + 010 Pa	
v	0.2	
Multilinear Isotropic (C20/25)		
	Strain ε, [m/m]	Stress f_c , [Pa]
0	0	0
1	0.0003	9,000,000
2	0.0006	16,774,676.7
3	0.0012	27,859,807.8
4	0.0018	32,580,872.6
5	0.00222	33,300,000

The multilinear isotropic material uses von M i s e s failure criterion along with the W i l l a m and W a r n k e (1974) model to define the failure of the concrete. E_c is the modulus of elasticity of the concrete, and ν is the Poisson's ratio. The compressive uniaxial stress–strain values for the concrete model was obtained using the following equations with which is computed the multilinear isotropic stress–strain curve for the concrete (Kachlakev, 2001; Wolanski, 2004; Raongjant & Jing, 2008):

$$f = \frac{E_c \varepsilon}{1 + \left(\varepsilon / \varepsilon_0\right)^2}, \tag{1}$$

$$\varepsilon_0 = \frac{2 f_c^{'}}{E_c}, \quad (2) \qquad\qquad E_c = \frac{f}{\varepsilon}, \quad (3)$$

where: f is the stress at any strain ε, ε – strain at stress f, ε_0 – strain at the ultimate compressive strength $f_c^{'}$.

The multilinear isotropic stress–strain implemented curve requires that the first point of the curve to be defined by the user, this must satisfy Hooke's law.

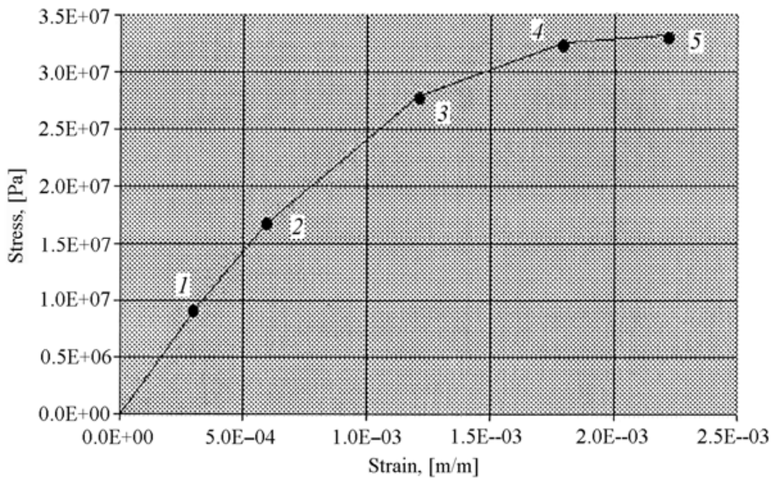


Fig. 3 – Uniaxial stress–strain curve for concrete C20/23.

The uniaxial stress–strain curve (Fig. 3) is built using eqs. (1),…,(3). Point *1* is defined as $0.30 f_c^{'}$ and is calculated in linear range. Points *2*, *3* and *4* are calculated from eq. (1) with ε_0 obtained from eq. (2). Point *5* is defined at $f_c^{'}$ and $\varepsilon_0 = 0.00222$ m/m, indicating crushing strain for unconfined concrete.

Implementation of the Willam and Warnke material model in ANSYS requires different constants to be defined (Table 2).

Table 1
Concrete Constants

1	Shear transfer coefficients for an open crack (β_t)	0.4
2	Shear transfer coefficients for an closed crack (β_c)	0.8
3	Uniaxial tensile cracking stress (f_r)	2E+006 Pa
4	Uniaxial crushing stress (f_c)	3.33E+007 Pa
5	Biaxial crushing stress	0
6	Ambient hydrostatic stress state for use with constants 7 and 8	0
7	Biaxial crushing stress under the ambient hydrostatic stress state	0
8	Uniaxial crushing stress under the ambient hydrostatic stress state	0
9	Stiffness multiplier for crack tensile condition	0

The shear transfer coefficient, β, represents conditions of the crack face. The value of β ranges from 0 to 1, with 0 representing a smooth crack (complete loos of shear transfer) and 1 representing a rough crack (no loss of shear transfer) (Kwan *et al.*, 1999; Terec *et al.*, 2010).

2.2. Reinforcement Modelling

The reinforcement bars may be included in the finite element model in two ways: as a discrete model (individual bars) (Fig. 4), or through a smeared model (Fig. 2) (Raongjant & Jing, 2008; Kheyroddin & Naderpour, 2008; Kachlakev *et al.*, 2001; Wolanski, 2004).

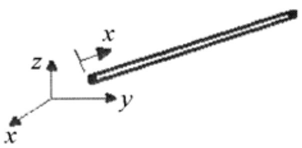


Fig. 4 – Link 8 geometry.

In the cases analysed in this paper reinforcement were modeled by using separate element called Link 8, a 3-D spar element. The geometry and node locations for Link 8 are show in Fig. 4. This element has two nodes and three degrees of freedom for each node, translations in the nodal *x*-, *y*- and *z*-directions. The bond between concrete and reinforcement was assumed to be perfect. In Fig. 5 is shown the stress–strain curve of reinforcement used in this study. The bilinear kinematic hardening model (BKIN) was used (Kachlakev *et al.*, 2001; Wolanski, 2004). The bilinear model requires the yield stress (f_y) and the hardening modulus of the steel ($E_s^{'}$).

Constitutive law for steel behavior is

$$\begin{cases} \sigma_s = E_s \varepsilon_s \,, \quad \varepsilon_s \le \varepsilon_y \,, \\ \sigma_s = f_y + E_s^{'} \varepsilon_s \,, \quad \varepsilon_s > \varepsilon_y \,, \end{cases} \tag{4}$$

where: σ_s is the steel stress, ε_s – the steel strain, E_s – the elastic modulus of steel, $E_s^{'}$ – the tangent modulus of steel after yielding , $E_s^{'} = 0.01 E_s$, f_y and ε_y – the yielding stress and strain of steel, respectively.

Table 3
Material Properties for Link 8

Linear Isotropic (S 355)	
Elastic modulus, E_s	2.1E+0.11 Pa
Poisson's ratio, v	0.3
Bilinear Isotropic (S 355)	
Yield stress, f_y	3.55E+0.08 Pa
Tangent modulus, $E_s^{'}$	2.1E+0.09 Pa

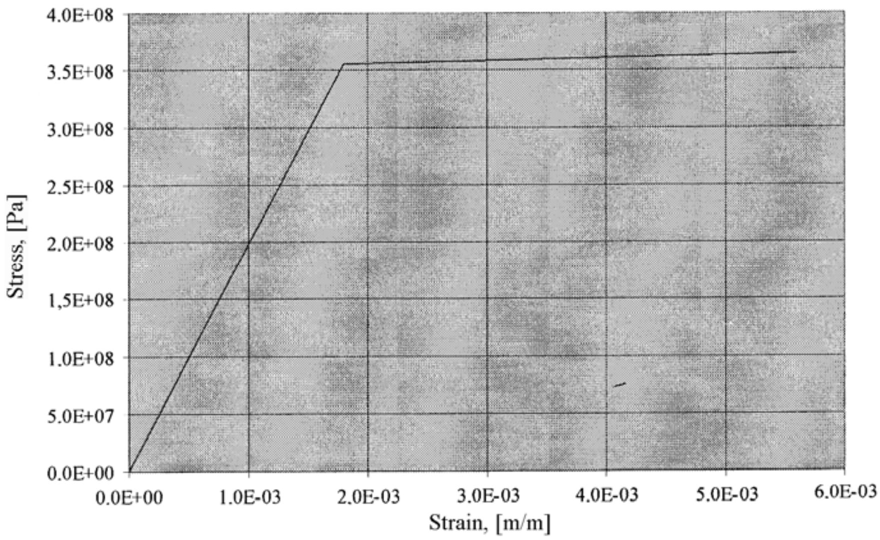


Fig. 5 – Stress–strain curve for reinforcement in ANSYS model.

2.3. Steel Plate Modelling

The Solid 45 element was used for the steel plates at loading points on the structural wall (Fig. 6). This element has as a linear isotropic behavior with modulus of elasticity and Poisson's ratio as steel (Kachlakev *et al.*, 2001; Wolanski, 2004).

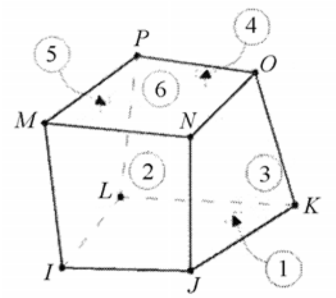


Fig. 6 – Solid 45 geometry.

2.4. Finite Element Mesh

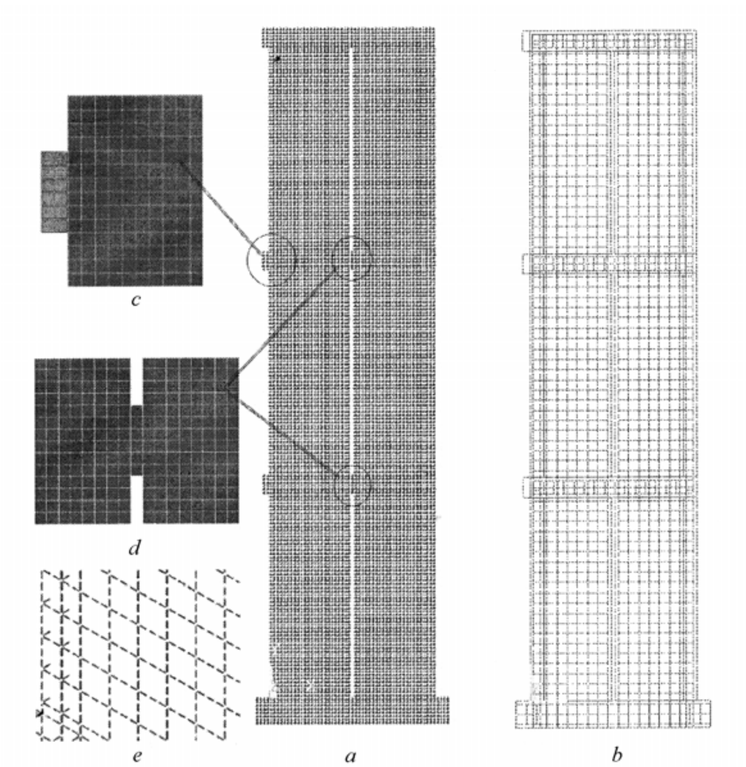


Fig. 7 – Finite element model: *a* – concrete Solid 65, *b* – reinforcement Link 8, *c* – detail of loading zone, *d* – details of connections, *e* – 3-D detail of reinforcement arrangement.

To obtain good results from Solid 65 element, the use of a rectangular mesh is recommended (Fig. 7). Concrete of the reinforced concrete structural slit walls was meshed with cubic element of dimension 50 mm and all the reinforcement were meshed with 50 mm long link element (Raongjant & Jing, 2008).

2.5. Loading and Boundary Conditions

The structural wall is fully restrained at the base. It is symmetrical about *XY* plane, in this way we reduce in half the number of finite elements and computation time, and symmetry boundary conditions will be set so that all nodes at $Z = 0$ plane of symmetry were given the constraint $UZ = 0$.

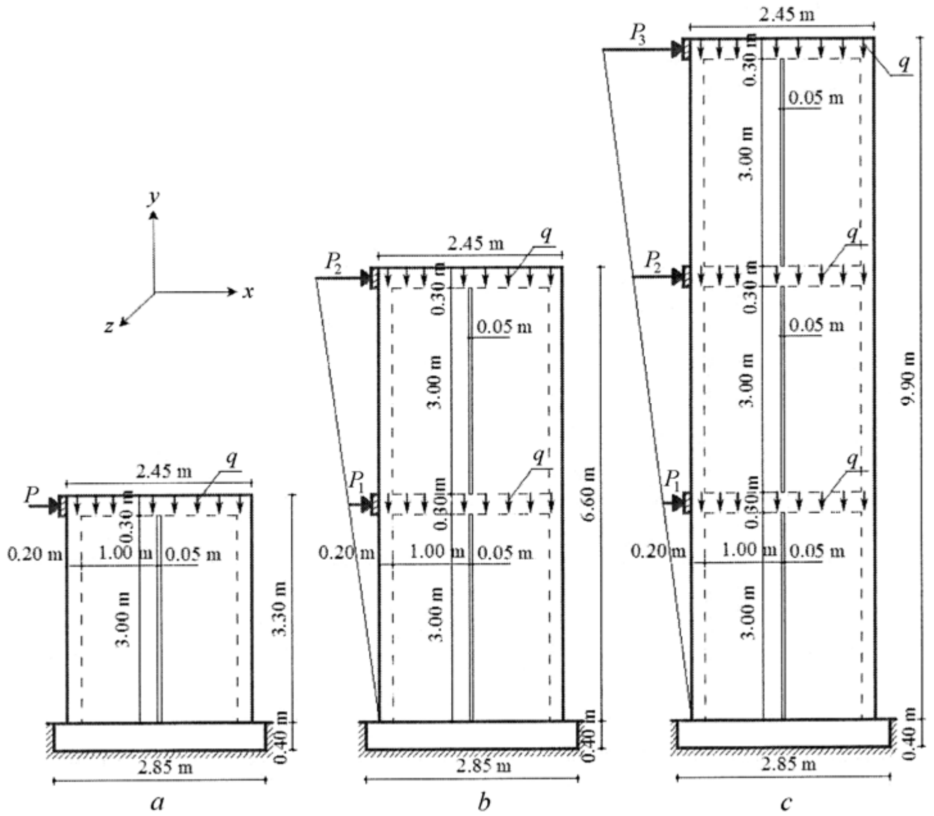


Fig. 8 – Force applied on steel plate for each case of wall considered:
a – one level; *b* – two levels ans *c* – three levels.

Lateral loads are disposed linearly along the height of wall. Concentrated forces are applied at each level through steel plates to avoid the stress concentrations (Fig. 8). The gravity loads include structural wall weight which is taken by the program and loads from floor connected to the wall, q = 35,000 N/m.

3. Structural Walls Considered for Analysis

Six reinforced concrete structural walls were proposed to investigate the influence of vertical slits in the wall structure. Comparative analysis were conducted on slitted walls and solid walls. There are considered three types of structural walls: with one level (3.30 m), with two levels (6.60 m) and with three levels (9.90 m) (Fig. 8). The height of each connection is of 0.30 m and the thickness of the slit is of 5 cm. The thickness of the wall is of 0.20 m and the lenght of all walls is of 2.45 m. All the reinforcements were designed about minimum ratio. The structural walls are reinforced with vertical bars Φ10/15 and horizontal bars Φ8/15, boundary columns are reinforced with four vertical bars Φ10 and stirrups Φ8/15 and loading beams are reinforced with four horizontal bars Φ10/15 and stirrups Φ8/15.

4. Comparison of Analytical Results

The pushover analysis of a structure is a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear *versus* top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load. The maximum resultant of lateral forces applied on proposed structural walls is of 600 kN. In nonlinear analysis, the total load applied to a finite element model is divided into a series of load increments called *load steps*. At the completion of each incremental solution, the stiffness matrix of the model is adjusted to reflect nonlinear changes in structural stiffness before proceeding to the next load increment. ANSYS uses Newton-Raphson equilibrium iterations for updating model stiffness. For the reinforced concrete solid elements, convergence criteria were based of force and displacement, and the convergence tolerance limits are 0.5% for force checking and 5% for displacement checking in order to obtain convergence of the solutions. For the nonlinear analysis, automatic time stepping predicts and controls load step sizes. If the convergence behavior is smooth, automatic time stepping will increase the load increment up to a selected maximum load step size and if the convergence behavior is abrupt, auto matic time stepping will bisect the load increment until it is equal to a selected minimum load step size.

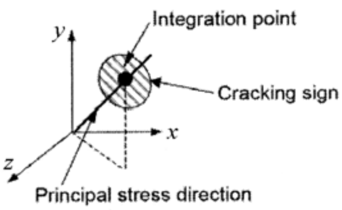


Fig. 9 – Cracking sign.

The ANSYS program records a crack pattern at each applied load step. The cracking signs are circles and appears perpendicular to the direction of the principal stress when the principal tensile stress exceeds the ultimate tensile strength of the concrete (Fig. 9) (Kachlakev *et al.*, 2001; Wolanski, 2004).

4.1. Pushover Analysis of One Level Structural Wall

Fig. 10 shows the load *vs.* lateral displacement curves for the structural walls with one level.

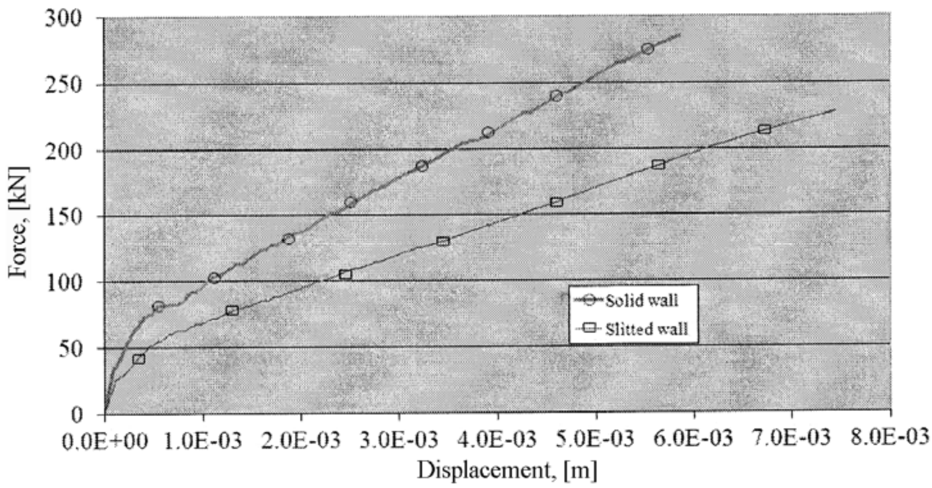


Fig. 10 – Comparison between slitted wall and solid wall with one level (H = 3.3 m).

Table 4
State of Cracking for Slit and Solid Wall with One Level

	Load, [kN]	State of cracking
Slit wall	27	first cracks start to appear at the base of the wall
	56	first cracks start to appear in shear connections
	228	failure of the shear connections
Solid wall	Load, [kN]	State of cracking
	35	first cracks start to appear at the base of the wall
	285	failure of the solid wall

As can be seen in Table 4, at a load of 228 kN the shear connections crush resulting convergence failure for slit wall. Solid wall fails at a load of 285 kN as a result of excessive cracking which occurs at the base. In Fig. 11 one can observe the first cracks and failure of the slit wall. Failure of the model is defined when the solution for a very small load increment still does not converge. The program then gives a message specifying that the model has a significantly large deflection, exceeding the displacement limitation of the program.

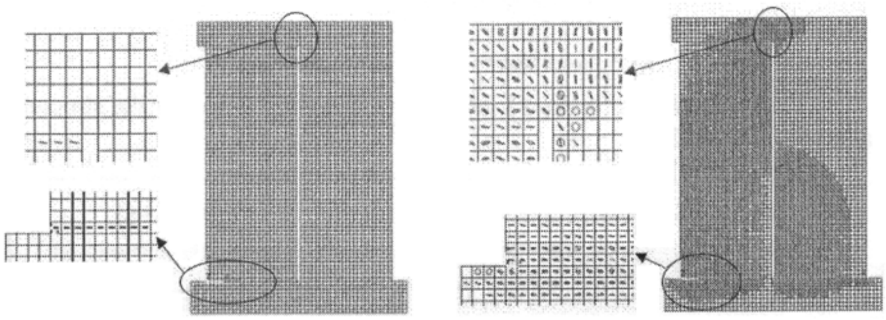


Fig. 11 – Cracking of slit wall: *a* – at 27 kN (first cracks); *b* – at 228 kN (failure of the shear connections).

4.2. Pushover Analysis of Two Levels Structural Wall

Fig. 12 shows the load *vs.* lateral displacement diagram for the structural walls.

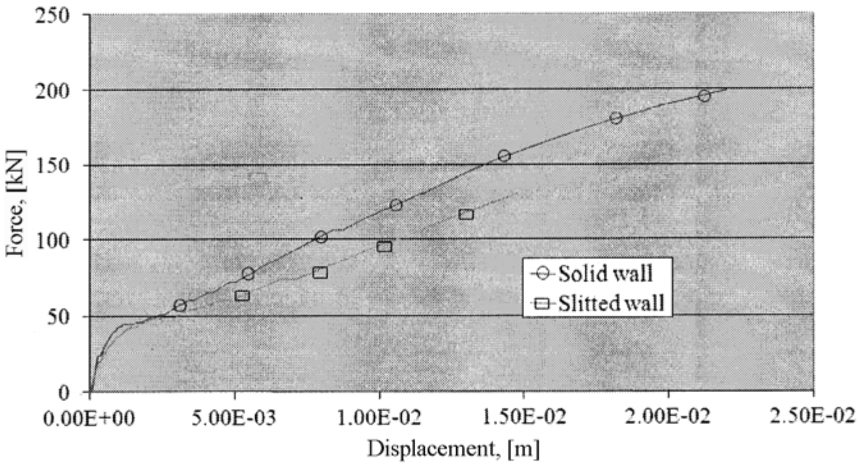


Fig. 12 – Comparison between slited wall and solid wall with two levels (H = 6.6 m).

Table 5
State of Cracking for Slit and Solid Wall with Two Level

	Load, [kN]	State of cracking
Slit wall	20.5	first cracks start to appear at the base of the wall
	50	first cracks start to appear in shear connections
	130.6	failure of the shear connections
Solid wall	Load, [kN]	State of cracking
	24	first cracks start to appear at the base of the wall
	200	failure of the solid wall

As can be seen in Table 5, at a load of 130.6 kN the shear connections crush resulting a convergence failure for slit wall. Solid wall fails at a load of 200 kN as a result of excessive cracking which occurs at the base. In Fig. 13 it can be observed the first cracks and failure of the slit wall.

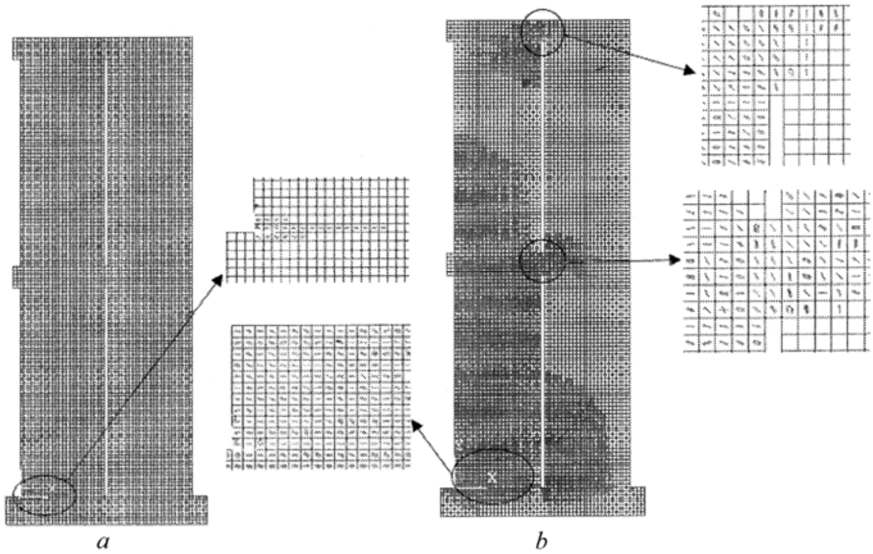


Fig. 13 – Cracking of slit wall: *a* – at 20.5 kN (first cracks); *b* – at 130.6 kN (failure of the shear connections).

4.3. Pushover Analysis of Three Levels Structural Wall

Fig. 14 shows the load *vs.* lateral displacement diagram for the structural walls.

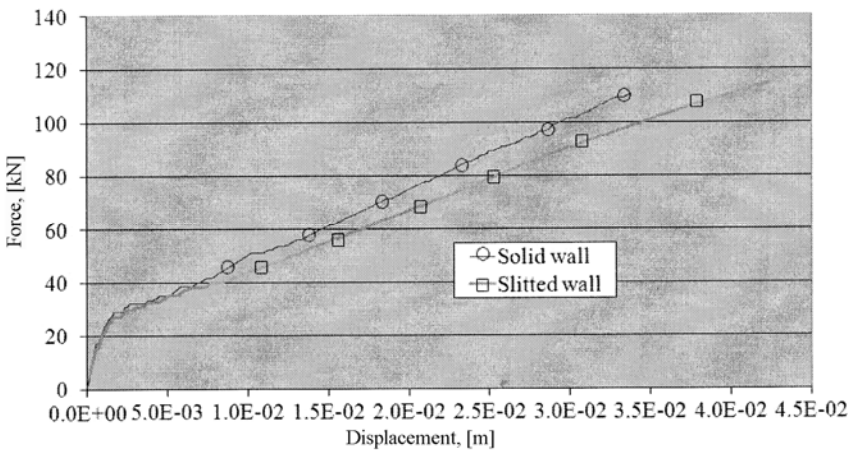
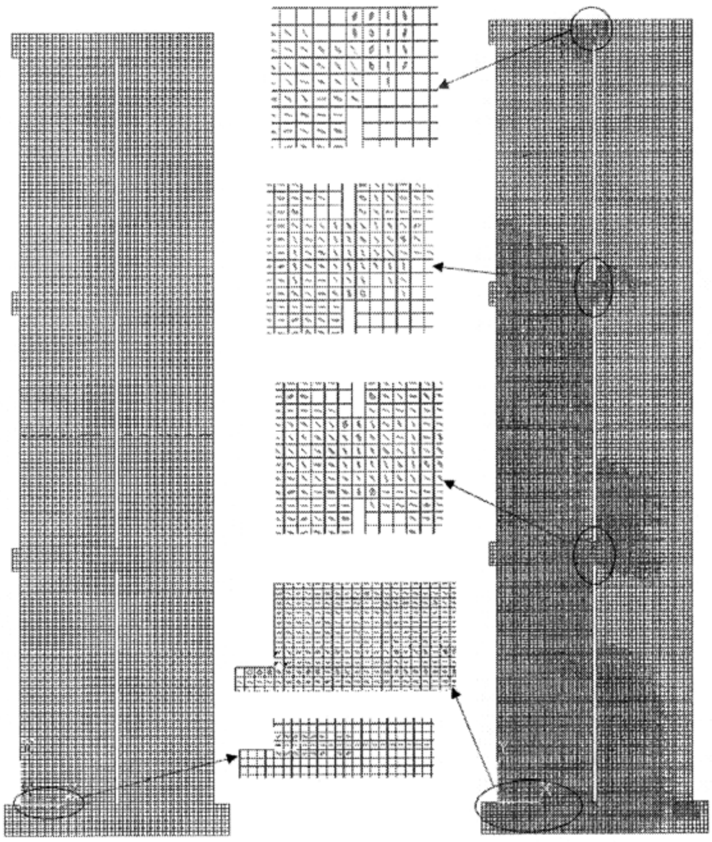


Fig. 14 – Comparison between slited wall and solid wall with three levels (H = 9.9 m).

Table 6
State of Cracking for Slit and Solid Wall with Three Level

	Load, [kN]	State of cracking
Slit wall	16	first cracks start to appear at the base of the wall
	42	first cracks start to appear in shear connections
	115	failure of the shear connections
Solid wall	Load, [kN]	State of cracking
	17	first cracks start to appear at the base of the wall
	111	failure of the solid wall



a *b*

Fig. 15 – Cracking of slit wall: *a* – at 16 kN (first cracks); *b* – at 115 kN (failure of the shear connections).

As can be seen in Table 6, at a load of 115 kN the shear connections crush resulting a convergence failure for slit wall. Solid wall fails at a load of 111 kN as a result of excessive cracking which occurs at the base. In Fig. 15 can be observed the first cracks and failure of the slit wall.

4.4. Results Interpretation

Comparisons between pushover curves shows that slit walls and solid walls have a similar behavior until shear connectors start to crush. As the cracks on shear connections increase we can observe that the rigidity of the wall decreases and the structural walls split in two forming in this way two flexible solid walls in each case analysed. An important observation is that applicability of this solution for energy dissipation is more pronounced for tall structural walls from high-rises buildings, where the predominant effort is a bending one. The wall must be sufficiently slender so that the slipping along the connections zone to appear before cracks from the wall base become dangerous. This slip produces shear cracks in short connections. If the predominant effort is shear the slipping appears after development of large degradations at wall base; this phenomenon must be avoided. In the analysed cases we can observe that proposed walls must be taller for a better behavior of this energy dissipator. Because the model has reached to a very large number of finite elements (over 300,00), with a computation time up to 120 h for the reinforced concrete structural walls with three levels, will be proposed in the second part of this work the reduction in width of the analysed walls and an increase in height. In this way the wall will be more slender and dominated by bending.

5. Conclusions

An economical design of buildings based on performance takes into account the dissipation of seismic energy accumulated in the structure. Reinforced concrete walls are frequently used as strength elements for structures designed in areas with high seismic risk. The fact is, in a tall structural wall, plastic hinge formation happens only at the base of the wall and ductility resources of the rest of the wall remains untapped. If ductility is a major concern, structural walls are not considered as efficient structural component. The main problems of these structural elements – low ductility and redundancy – are removed through the solution presented in this paper. The shear connections prevent collapse of the structure under extreme seismic excitations by dissipating energy through shear yielding. For best performance, the shear connections should maintain their load carrying and energy dissipation capacities until the whole structure fails. Finite element method is frequently used in the design of reinforced concrete structural walls, investigations performed by researchers have shown comparable results with practical experimental models.

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ANALIZA NELINIARĂ A PEREŢILOR ŞLIŢAŢI DIN BETON ARMAT CU PROGRAMUL DE ELEMENT FINIT ANSYS

(Rezumat)

Pereţii structurali din beton armat reprezintă un sistem care oferă unei clădiri rezistenţă laterală şi rigiditate mare. Deoarece disiparea energiei are loc doar prin degradări la baza peretelui, aceştia nu au o ductilitate şi redundanţă bună. Pereţii din beton armat disipatori de energie cu conexiuni scurte, numiţi şi *pereţi şliţaţi*, rezolvă o parte din problemele pereţilor structurali obişnuiţi. Plastificarea conexiunilor scurte la aceşti pereţi creşte disiparea de energie şi formează un amortizor structural care este bazat pe controlul structural pasiv. Datorită acestei soluţii ductilitatea de ansamblu creşte, rezultând un plus de siguranţă pentru construcţie. În acest mod se creează o structură ideală pentru clădirile înalte, cu o comportare rigidă la încărcări seismice reduse care se transformă într-o structură ductilă disipatoare de energie la încărcări seismice ridicate.

Se efectuează analiza statică neliniară a pereţilor structurali disipatori de energie cu conexiuni scurte şi este evaluată comportarea elasto-plastică a conexiunilor. Acest tip de perete este comparat cu unul obişnuit, compact. Soluţiile propuse de ductilizare a pereţilor sunt viabile şi uşor de pus în practică. Pentru analiza pereţilor structurali propuşi a fost folosită metoda elementului finit cu care se poate simula efectiv comportarea la încărcări laterale a pereţilor structurali din beton armat.