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ENGINEERING MODEL FOR ANALYIS OF MASONRY STRUCTURES

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Abstract

This paper presents the methodology for seismic analysis of masonry structures that can be employed in commercial software packages such as SAP2000. The concept of elementary block which combines non-linear spring and linear shell elements is used for discretization of masonry walls. The proposed modelling technique with localized nonlinearity can successfully simulate in-plane wall failure modes induced by compressive or tensile axial force and transverse force. It can also be used to investigate out-of-plane collapse which makes it a good candidate for 3D static and dynamic analysis of buildings. The modelling approach is tested on two examples where pushover analysis was performed: a single slender cantilever masonry wall and a family house. The response was verified against the results delivered by 3MURI and MINEA, and reasonable agreement was obtained. It is demonstrated that the transverse walls have significant contribution to the load bearing capacity of buildings.

Keywords: masonry structures, elementary block, nonlinear spring elements, SAP 2000.

1. Introduction

There are at least two motives for the research presented in this paper. First, during the two previous decades it has become a common practice to upgrade the existing multi-story masonry buildings for one or two new floors. One example is showed on figure 1, presented the former existing masonry building and the upgraded one, situated in the vicinity of the Faculty of Civil Engineering in Sarajevo. Second, a large number of masonry buildings in our country were built before the introduction of the first seismic codes. These buildings don't fulfill most of design requirements set by modern seismic regulations. However, the experiences gained during past earthquakes in the region of the South-East Europe showed us that the existing masonry buildings possess higher resistance than one might conclude from the everyday engineering analysis. It is obvious that the masonry structure should be modelled as a whole.

Figure 1. Masonry building before reconstruction (left), after reconstruction (right).

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This paper presents the concept of seismic analysis based on macro-element named "elementary block" and non-linear link elements. Such model is then implemented in software SAP2000 [4]. Numerical analysis of the building is based on a 3D model with transversal walls that carry a seismic load in their plane and to some extent out-of-plane. Model comprises the interaction of reinforced concrete slab with masonry walls as well as torsional effects. Verification of suggested concept is performed comparing the results of the analysis with results obtained from finite element programs 3MURI [1] and MINEA [3]. Comparing the results, it is shown that masonry building behavior can be successfully described through numerical models, composed of the proposed elementary blocks and nonlinear link elements. The proposed procedure could be implemented into other finite element software, which provides the masonry wall models composed of elementary blocks and nonlinear connection elements [7].

2. Elementary block

It is known from the theory of structures that the most stressed places in structure due to the ordinary loads are in the joints of the structure, such as beam-column connections, wall and slab connections, foots of the walls and columns etc. It is therefore possible to predict the locations in structures where nonlinear deformations would appear. Less stressed structural parts are modelled as linear elastic, while nonlinear behavior is restricted to the most stressed parts. This is the approach applied in this work, where we investigate the behavior of masonry structures under seismic loading. However, in everyday engineering practice non-linear behavior of the structure is assumed only in the case of moderate or stronger earthquake loadings. The structure exposed to gravity loads alone, including combination with usual wind loads, should remain elastic.

The simple engineering model that can simulate the participation of all elements in the structure in taking over the seismic load is presented. Assumed that we know the stress state in a wall (Fig. 2 left). We divide the wall cross-section into segments. The normal force in each of those segments is the product of the mean value of the stress and the cross-sectional area of that segment. The total normal force in the wall is the sum of the normal forces in the segments, and the total moment in a cross-section of the wall is the sum of the product of the distance of an individual force and the intensity of that force.

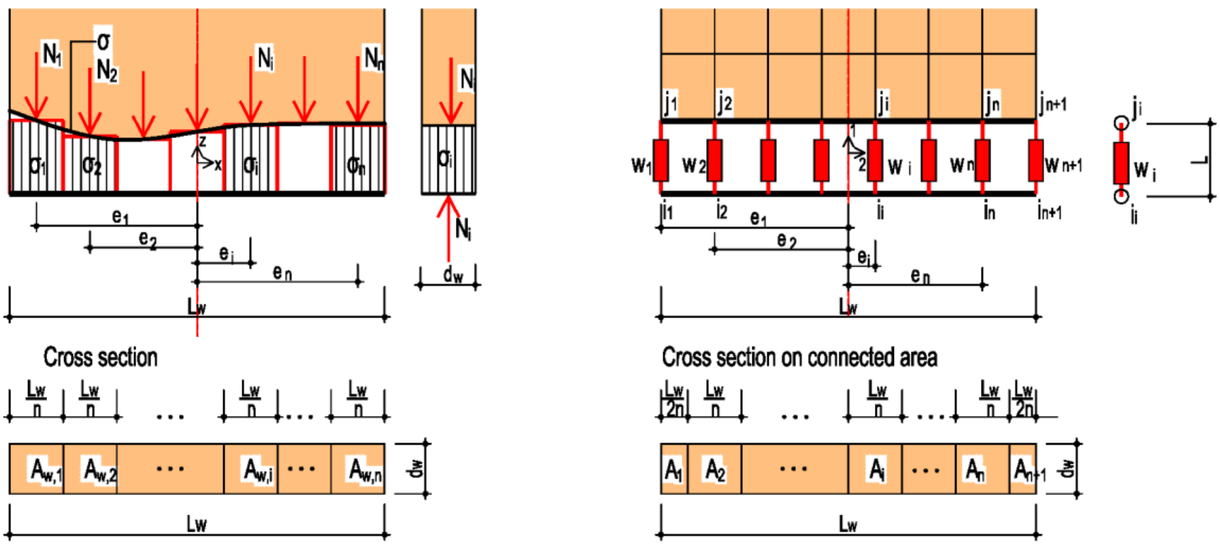

Figure 2. Distribution of normal stresses (left) and finite element discretization (right).

The division of the wall into the segments corresponds with the mesh of finite elements. The position of each finite element in the wall is known (Fig. 2 right). Between the wall and the close surroundings (could be another wall or wall segment), an element with zero-length (L→0) is introduced. Each node of the finite element network above the joint (ji) can be connected to the other part of the structure by nonlinear spring in the nodes (ji). The geometric properties of the spring correspond to the surface of

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the connecting wall, and the material properties of the spring correspond to the material properties of the wall. The calculated load capacities of individual connecting elements N_i, R_d are determined as the product of the area of the wall segment A_i connected by these link elements and the calculated strength of the wall f_d. There are two types of adopted axial connecting elements (Figure 3). Type A (E) is a connecting element on the edge of the coupling. Types of connecting elements B (C, D) are located inside the coupling. Specific role of the connecting element (C, D, E) will be clarified later. For the adopted types of connecting elements, the calculated axial load capacity is:

$$N_{A,Rd} = N_{E,Rd} = \frac{L_w \cdot d_w}{2 \cdot n} \cdot f_d \tag{1}$$

$$N_{B,Rd} = N_{C,Rd} = N_{D,Rd} = \frac{L_w \cdot d_w}{n} \cdot f_d \tag{2}$$

Axial forces in individual connecting elements are a function of the stiffness and displacement of the nodes of the finite elements used to model the wall:

$$N_i = f(k,u) \leq N_{i,Rd} \tag{3}$$

With the gradual increase of the design load, which can be set by the software, the design forces in the connecting elements also increase, that is, the stresses σ_i increase in the part of the wall where the connecting elements are introduced. Upon reaching the given computational strength of each link element, the nonlinear behavior of the model begins. Upon reaching the calculated bearing capacity of all connecting elements in the coupling, the model will yield. The total calculated bearing capacity of the wall under the action of the normal force N_{Rd} represents the sum of the calculated bearing capacities of all connecting elements of the joint:

$$N_{Rd} = \sum_{i=1}^{n+1} N_{i,Rd} = 2 \cdot N_{A,Rd} + (n-2) \cdot N_{B,Rd} + N_{C,Rd} = A \cdot f_d \tag{4}$$

The number of finite elements n along the length of the wall depends on the geometry of the wall and cannot be determined in advance. Therefore, it is not possible to determine the number of connecting elements in the coupling. Two connecting elements of type A and one connecting element of type C represents basic connection model. The number of connecting elements of type B depends on the ratio of height and length of the wall. The minimum number of finite elements per connector is n=4, and the number of connecting elements is n+1.

The model can also simulate the behavior of a wall exposed to normal pressure force with eccentricity. The calculated bearing capacity of the wall under the action of the bending moment in the plane of the wall, expressed through the axial forces in the connecting elements and the position of the connecting elements in the wall, is:

$$M_{Rd} = \sum_{i=1}^{n+1} N_i \cdot e_i \tag{4}$$

The connection element C is used to simulate the behavior of the wall exposed to the transverse loads in the plane of the wall. In the direction of the axis 1 of the n-link element, this connecting (link) element has properties like connection element B. However, in the direction of axis 2, it describes the in-plane shear capacity of the wall. Connecting elements A and B are pure axial elements. Depending on the height the wall could be modelled as an elementary block with connecting elements at the bottom and top of the wall, In the case of relatively higher masonry walls, a few elementary blocks and connection levels could be modelled along the height of the wall. In that case, on possibility is to introduce connecting elements E and D, which are transversely rigid, and axially behave like elements A and B.

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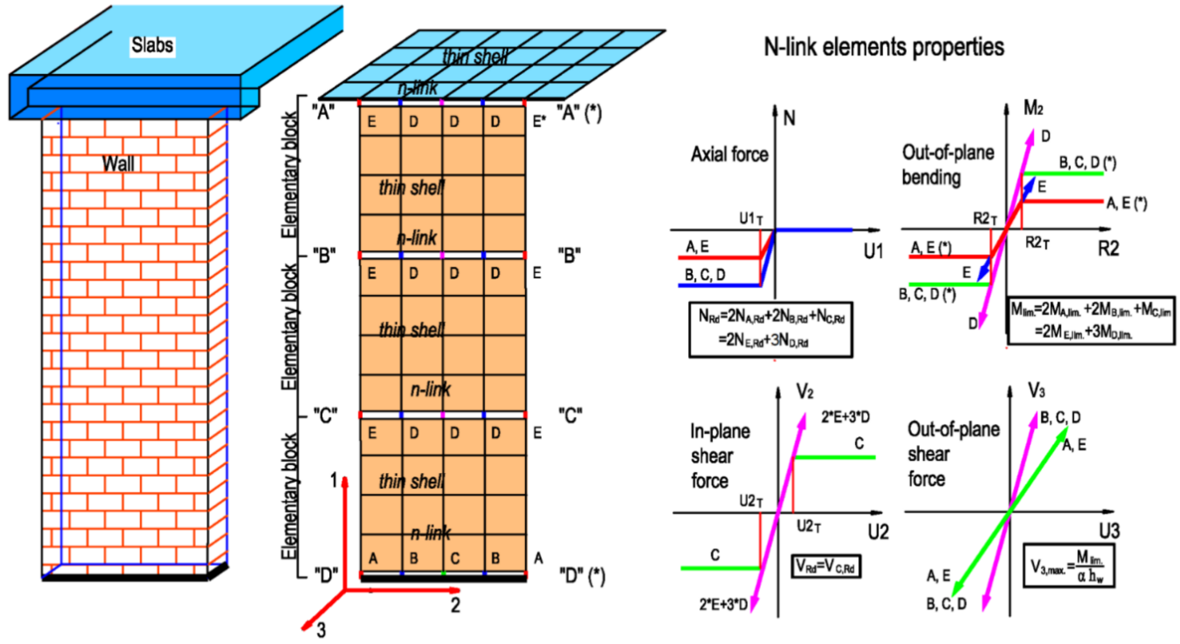

Figure 3. Concept of elementary block.

Simulation of out of plane behavior is possible by coupling elements “A” and “D”. For a known normal force in the wall, the components of M2 can be calculated and limited, so that, for example, the normal force remains in the core of the section. Couplings “A”, “B”, “C” and “D” are axially non-linear. They simulate the collapse of a wall under the action of a centric or eccentric normal force in wall plane. Couplings “A”, “B”, and “C” assume a linear transverse force, while nonlinearity to transverse forces is introduced in coupling “D” with link C. For the case of analysis in and out of plane of the wall, couplings “A” and “D” are rotationally limited in their plane.

One such composition of finite elements and connecting (links) elements is called an elementary block. Finite elements within the elementary block provide linear behavior of the wall, while connecting elements in the joints (connection) provide nonlinear behavior. That means, non-linear behavior is limited to the connection elements.

3. Verification of the modelling approach

3.1 Behavior of the model

The following figure shows the collapse situation of the wall in the plane according to FEMA[5]. The different behavior of the model, on the example of a cantilever wall, each of these responses are illustrated is also shown on the right side of the Figure 4. At the top floors of the building, where the normal and transverse forces in the wall are relatively smaller, we have the behavior of the wall as a rigid body, due to the bending, opening of the joints for tension without crushing at the edges. Similar behavior is also on the floor below, but with crushing at the edge in pressure. If the transverse forces are dominant, then the collapse occurs due to sliding. Diagonal cracks that are typical damages of the masonry wall, appears when the principal stresses reach the tensile strength of the wall cannot be directly simulated by the proposed model because the joints of elementary blocks are introduced horizontally. Nevertheless, indirectly, through the limitation of the transverse force for the known normal force, this fracture can also be simulated, and in the model it manifests itself as sliding.

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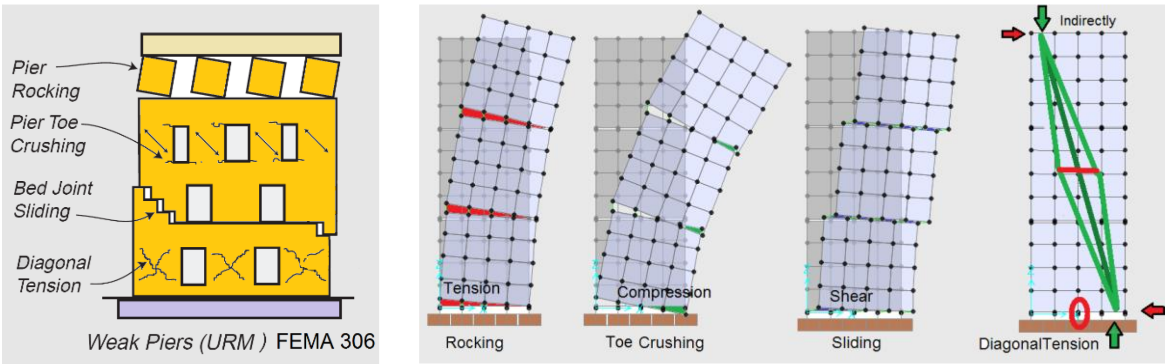


a) b)

Figure 4. a) Failure modes according to FEMA, b) response capacity of the model.

3.2 Analysis of a family house

This example analyses a typical German family house which has a system of bearing walls in two orthogonal directions (x and y). The whole urban settlements are formed by the rows of these simple buildings. Building lies side by side; separated by joints, so each building represent structurally independent unit. At the building lay-out the walls are marked as W1, W2...and W9 and their cross-sections are constant from the ground to the roof of the building (Figure 5). Other walls are partition walls and do not have vertical continuity as well. Seismic load in x direction is resisted mainly by walls W2, W3, W5, W6, W7 and W8. According to building tradition at similar buildings, slabs are monolithic casted in situ and without beams. It is obvious that the building is more vulnerable in a shorter (transverse) direction, view in the plane of the building.

Capacity curves obtained by software 3MURI are a function of effective plate area. Due to the absence of the usual beams, the space frames are modelled with the effective width of the reinforced concrete plate. Capacity curve obtained by software SAP2000 presents the result of the analysis where the walls are modelled in plane with elementary blocks. Limited out-of-plane behavior is included in curve SAP-b. Good matching of results obtained by different analysis is obvious. Capacity curve obtained from software MINEA has almost the same capacity as curve obtained from software SAP2000, but quite different form, because the capacity determined by MINEA software is based on experimental data on different masonry wall's bearing capacity, incorporated in software. Comparing with experimental results of the tests conducted in research project ESECMASE [3], it can be concluded that the proposed model shows significant advantages over traditional modelling of masonry structures in commercial software.

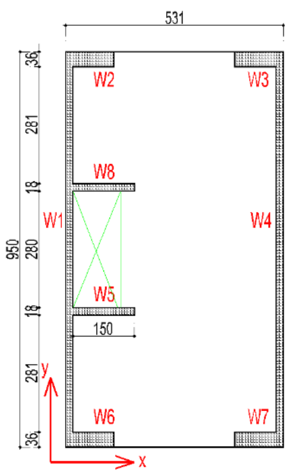
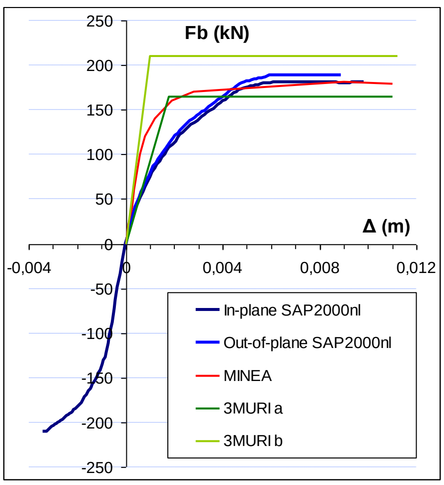
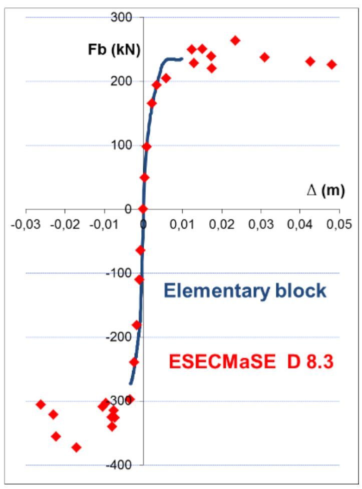


Figure 5. a) Lay-out of the building



b) Capacity curve obtained by different analysis



c) Comparison with experiment

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3.3 Some specifics of the model

Implementing the proposed modeling procedure different performances of masonry walls can be included. On the left side of the Figure 6 the rocking of the masonry wall pier is presented. The uplifting is prevented by floor structure. Due to the restraint an additional normal force could be imposed on the wall structure. The effect can be covered by proposed modelling strategies using commercial software. On the right side of the Figure 6 the shear behavior of the wall is dominant, characterized by typical diagonal cracks mostly through mortar joints. The former effect of uplifting and increased axial force is missing.

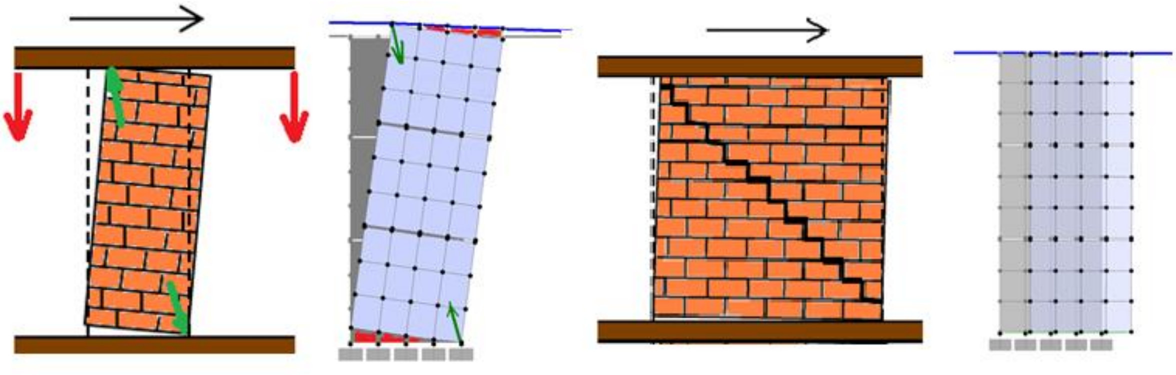

Figure 6. Rocking with increase in normal force (left) and sliding (right).

The response of the walls in the direction of the earthquake can also lead to the collapse of the connecting perpendicular walls due to tension and uplifting. In the Figure 7 below, for the positive direction of the push in wall W1, there is a decrease in the normal (vertical) force, opening of joints in wall W1, strong rocking behavior that detaches the floor structure from wall W2, which results in a tensile fracture of wall W2. For the opposite direction of pushing, in wall W1 there is an increase in the normal force, which results in its higher bearing capacity for the transverse forces. These affects can be successfully modelled by presenting procedure and it is shown on the right side of the Figure 7 through the deformation of the elements mesh.

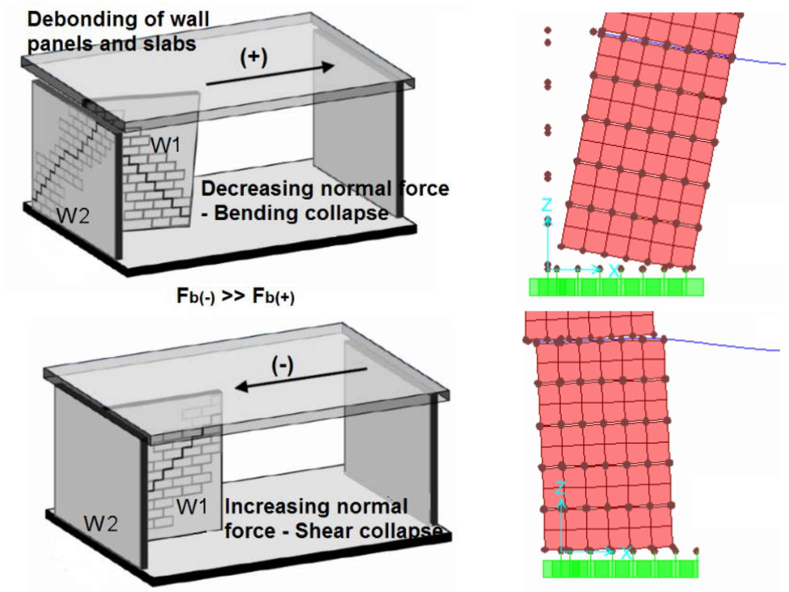

Figure 7. Interaction of longitudinal and transversal wall [6].

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It should be emphasized that the model take in account torsional effects for structures where the center of mass does not coincide with the center of stiffness.

4. Conclusion

With this relatively simple engineering approach, it is possible to successfully simulate the behavior of buildings up to the point of failure implementing finite element software that contains connecting or link element capable to describe non-linear behavior. If the incremental dynamic analysis is included, the history of stresses and crack formations could be followed step by step. The proposed procedure is suitable for every day engineering praxis and more user friendly comparing to highly specialized software. This is even more true if one wants to build three-dimensional model of the masonry structure and wants to include the major effects produced in a masonry building exposed to the stronger earthquake motion. The concept of elementary blocks and reduction of non-linear behavior to the known critical areas facilitates the review and analysis of the usually very extensive results of three-dimensional structural analysis, especially if the non-linear behavior of the structure is included. And, the inclusion of the whole building structure (3-D analysis) gives us better insight into the seismic response and the capacity of the building, if it's exposed to moderate or stronger earthquake.

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