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An Equivalent Truss Model for In-Plane Nonlinear Analysis of Unreinforced Masonry Walls

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Abstract

According to the importance of seismic evaluation of existing unreinforced masonry (URM) buildings, researchers have been interested in numerical modelling of these types of structures and their components. On the other hand, in seismic performance evaluation and retrofitting codes which are mostly based on Performance Based Seismic Design (PBSD), different analysis methods such as linear and nonlinear, static and dynamic analyses are employed. Therefore, simple equivalent frame models with lower computational cost are very useful for modelling and analysis of unreinforced masonry buildings. In this article, a simple equivalent truss model is proposed for modelling and analysis of an unreinforced masonry wall with sliding shear failure as the governing in-plane failure mode. The model is developed according to an analogy between the internal forces in a triangular truss and the Mohr-Coulomb failure criteria. Then, the proposed model is generalized for modelling and push over analysis of combinations of walls. Finally, the modelling procedure is applied for push over analysis of an unreinforced masonry wall consists of some piers and the push over curve of the wall is determined and discussed.

Keywords: Masonry Wall; Nonlinear Analysis; URM Building; Sliding Shear Failure; Truss Model.

1. Introduction

Significant number of unreinforced masonry buildings with different occupancies such as residential or official exist in all over the world. Some of these buildings are historical monuments like churches, mosques and masonry towers. Unreinforced masonry walls are the main load resisting elements in these buildings because they resist lateral loads due to earthquakes as well as the gravity loads from the floors. Therefore, considerable experimental and analytical studies are conducted to investigate the in-plane and out-of-plane behavior of unreinforced masonry walls especially against seismic loads [1-7].

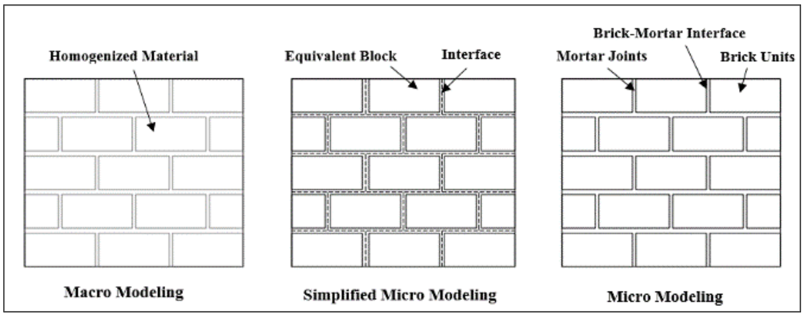

Figure 1. Numerical modeling approaches for masonry

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Recently, different numerical modelling approaches are developed for analysis of unreinforced masonry walls and buildings by application of different numerical tools such as Finite Element Method or Discrete Element method. Modelling approaches that are recommended for unreinforced masonry walls could be categorized into three main groups; micro and simplified micro modelling, macro modelling and equivalent frame modelling. The micro and macro modelling approaches are illustrated in Figure 1, schematically.

In micro modelling approach the masonry units, mortar joints and the unit-mortar bond are modelled separately with appropriate constitutive models for each part. In simplified micro modelling approach, each masonry unit and half of the mortar joints around it are considered as equivalent blocks that are jointed together with appropriate interface models. Various researchers have proposed micro models for analysis of masonry walls [8-9]. Although the results of micro modelling approaches have high accuracy, because of complexity and major computational cost, they are not suitable for large scale modelling and analysis of masonry buildings with several walls and piers. In this regard, macro modelling approach with less computational cost has been introduced. In this method, masonry is assumed as a homogenized isotropic or anisotropic material and the equivalent mechanical properties of masonry assemblage are used in analyses. The results obtained by this method have less precision than the results of micro modelling approaches. Models developed by Lourenco et al. (1997) and Lourenco (2000) are some outstanding examples of this modelling approach [10-11].

The last modelling method is known as equivalent frame method. In this method, one dimensional elements such as frame or truss elements are utilized for simulation and analysis of URM Buildings. Because of simplicity and low computational cost, this method is very attractive for nonlinear static and dynamic analysis of unreinforced masonry buildings. Therefore, considerable investigations have been carried out for development of simple equivalent frame or truss models for analysis of URM buildings. Roca et al. presented an equivalent frame model for nonlinear analysis of URM buildings and historical monuments. They developed the model in three following stages; derivation of appropriate relationships for one dimensional curved elements, simulation of the wall behavior with one dimensional elements and development of appropriate constitutive models based on different failure modes of URM walls. Finally, a building was analyzed with the proposed model and acceptable results were obtained [12]. Roca also developed simple models for prediction of the ultimate in-plane capacity of URM walls based on simple equilibrium relationships. The method was developed for masonry walls with and without opening [13]. Pasticier et al. utilized software SAP2000 for nonlinear analysis and vulnerability assessment of masonry buildings. Standard lumped plastic hinge models recommended in codes with appropriate nonlinear links were utilized to simulate the nonlinear behavior of masonry components for static and dynamic analyses. Finally, the model was employed for seismic performance assessment of an existing building [14]. In another study, Chen et al. introduced an equivalent frame model with appropriate plastic hinges for nonlinear analysis of URM buildings [15]. A pier and spandrel model for frame analysis of masonry and reinforced concrete buildings with openings was proposed by Belmouden and Lestuzzi. The method which was based on smeared cracking theory, showed acceptable results in lateral strength and behavior of the buildings [16]. Among code based models for analysis of masonry buildings, a simple method based on Mexican Building Code (SMSA) has been introduced by Tena-Colunga and Cano-Licona. They modified the existing model by a modification factor applied to shear areas of the walls. It is notable that the existing model, only considered shear deformation of URM walls [17]. Sabatino and Rizzano proposed an equivalent frame model for analysis of masonry buildings. In the proposed method, appropriate models were suggested for masonry piers and spandrels. In this model, different collapse mechanisms in masonry piers and spandrels were considered. Finally, the model was verified by means of some available experimental results and was compared with other simplified models [18]. Recently, Lagomarsino et al. presented a model in software Tremuri for nonlinear analysis of URM buildings to evaluate their seismic performance [19]. Tie and strut models have been used for modelling of masonry walls, too. For instance, Fobaroschi and Vanin developed a new nonlinear static model for determining the lateral capacity of masonry buildings based on this theorem. This model can simulate the behavior of masonry in different stages; uncracked, cracked and softening conditions [20]. Salmanpour et al. through a state of the art article, investigated the deformation capacity of the URM walls. In this research the authors investigated a large number of experimental results and numerical models including macro, micro and equivalent frame models [21]. Addessi et al. [20] developed an equivalent frame model for nonlinear analysis of URM panels subjected to lateral in-plane loads. They developed sectional inelastic response of the masonry piers by assuming an inelastic response for the masonry under monotonic loadings. They also proposed a frame element for modelling of the masonry piers. In-plane flexural and shear failure modes were considered in the model. Finally, the model is employed for numerical modelling of some tests [22]. Recently, Foti presented a new experimental approach for push over analysis of masonry buildings based on an analogy between rock mass and wall mass for the mechanical characterization of the masonry. For this purpose, Hoek–Brown model for determining the nonlinear shear strength of masonry was utilized. Finally a nonlinear push over analysis was carried out on a building with the proposed method [23].

Since most of the existing traditional unreinforced masonry buildings have shown poor performance in earthquakes, seismic evaluation of these buildings and development of appropriate retrofitting methods for them have been taken into consideration by many researchers and engineers. Therefore, seismic evaluation and retrofitting codes such as FEMA356 (2000) [24], which are usually based on Performance Based Seismic Design have been introduced. In these instructions, different analysis methods such as linear and nonlinear, static and dynamic analyses are recommended for seismic evaluation of existing buildings. Consequently, simple equivalent frame models with lower computational cost with compare to the other complex modelling approaches are very useful for large scale modelling and analysis of unreinforced masonry buildings.

In this article, a simple equivalent truss model for nonlinear static analysis of URM walls with sliding shear failure as the governing in-plane failure mode that Mohr-Coulomb constitutive model is used to determine their ultimate in-plane strength, is presented. The proposed model can be used for code based vulnerability assessment of existing URM buildings. First, the model is developed for a single URM shear wall. Then, combination of the piers in plan and elevation of a building with several walls is explained. In addition, a wall consists of some piers is modeled with the proposed modeling technique and the results are discussed. The advantage of the current method with compare to the other equivalent frame models is that only bar elements with axial plastic hinges are employed for modeling of a URM wall. Also, the effect of gravity loads on the in-plane capacity of the masonry piers is considered automatically in the model

2. In-Plane Behavior and Strngth of Urm Walls

In order to perform nonlinear analysis of URM buildings, elastic and inelastic behavior of URM shear walls as the main lateral load resisting elements should be defined. In elastic range, different simplified formulations which are usually based on elementary structural mechanics are recommended in literature and codes. One of the common formulations for this purpose are Equations (1-a) and (1-b) which are suggested in FEMA356 to estimate the in-plane stiffness of URM piers.

a- Piers free to rotate at top (Cantilever walls)

$$k = \frac{1}{\frac{h_{eff}^3}{3E_m I_g} + \frac{h_{eff}}{A_v G_m}} \tag{1}$$

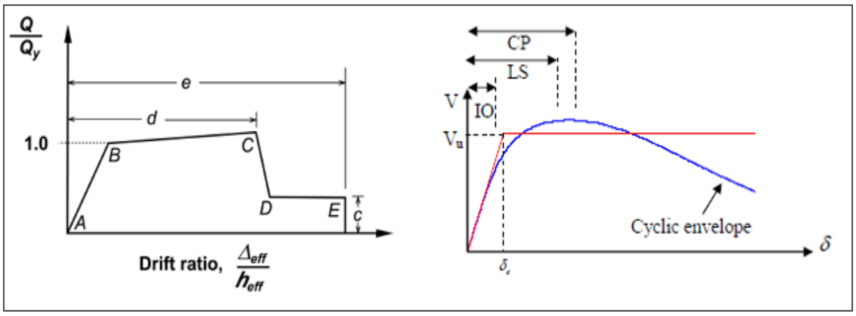
b- Piers are prevented to rotate at top and bottom

$$k = \frac{1}{\frac{h_{eff}^3}{12E_m I_g} + \frac{h_{eff}}{A_v G_m}} \tag{2}$$

Where, k is the in-plane lateral stiffness, h_{eff} is the effective height and A_v is the transverse shear area which all of the parameters belong to the URM piers. I_g is the moment of inertia of the pier section in flexural plane. Also, G_m and E_m are shear and elasticity modulus of the masonry material, respectively.

In inelastic range, the in-plane shear strength and post yield force-displacement relationship of the URM piers are needed. The general Mohr-coulomb relationship (Equation 2) is recommended by different codes like Eurocode6 [23] and FEMA356 (2000) to calculate the in-plane shear capacity of URM walls.

$$\tau = C + \sigma\mu \tag{3}$$



(a) (b)

Figure 2. a) Standard plastic hinge for URM piers in FEMA 356; b) Elastoplastic load-displacement curve used by Akhaveissy (2012) [26]

In Equation 2, τ is the shear stress which causes sliding failure in mortar bedjoints, C is the brick-mortar bond shear cohesion, σ is the normal stress on mortar bedjoints and μ is the brick-mortar interface coefficient of friction.

In Eurocode 6 the in-plane shear capacity of URM walls with filled and unfilled mortar head-joints is calculated by Equations (3-a) and (3-b), respectively.

$$f_{vk} = f_{vko} + 0.4\sigma_d \tag{4}$$

$$f_{vk} = 0.5f_{vko} + 0.4\sigma_d \tag{5}$$

Where f_{vk} is the characteristic shear strength of masonry, f_{vko} is the primary characteristic shear strength of masonry without compressive normal stress on bedjoints and σ_d stands for the compressive normal stress on bedjoints.

In FEMA356 the in-plane shear strength of a URM wall corresponding to the sliding shear failure mode through mortar bedjoints is calculated by Equation 4.

$$v_{me} = \frac{0.75\left[0.75v_{te} + \frac{P_{CE}}{A_n}\right]}{1.5} \quad (6)$$

Where, v_{me} is the expected shear strength of masonry, P_{CE} is the expected gravity load on the masonry pier, A_n is the net transverse area of the wall and v_{te} is the shear cohesion strength of the mortar bedjoints.

According to Equations 2 through 4, the in-plane shear capacity depends directly on the gravity loads on the wall.

Elastoplastic force-displacement relationship with or without hardening is recommended in literature and standards for inelastic lateral behavior of the URM piers. For example the standard backbone curve for the plastic hinges in FEMA356 is illustrated in Figure (2-a). Some other researchers apply elastic-perfectly plastic relationship for the inelastic behavior of URM piers. Akhaveissy [24] used elastic-perfectly plastic relationship for post yield behavior of URM piers, Figure (2-b). In current research, elastoplastic behavior is assumed for the post yield behavior of URM piers.

3. Equivalent Truss Model for a Single Urm Wall

In this study, an analogy is defined between a URM wall with a bi-element truss which is illustrated in Figure 3. The truss consists of a vertical and an inclined element which makes angle θ with horizontal direction. The vertical element is assumed rigid and the incline element is a compression-only element. Whole of the gravity and lateral loads are applied at joint b of the truss, separately.

Since the truss is a determinate structure, the internal forces do not depend on stiffness of the elements. In small deformation, lateral stiffness of the truss only depends on stiffness of the inclined element as follows:

$$K_t = \left(\frac{AE}{l}\right)_t cos^2\theta \quad (7)$$

Where K_t is the truss lateral stiffness, E is the modulus of elasticity corresponding to the truss material, θ is the angle between inclined truss element and the horizontal direction and A and I are cross sectional area and length of the inclined truss element, respectively. Therefore, by using Equations 1 and 5, cross sectional area of the inclined element of the equivalent truss can be determined.

Mechanical and geometrical properties of the equivalent truss can be achieved by comparison of the internal forces in truss elements with Equation 2 which estimates the in-plane shear capacity of URM walls. Equation 6 is the relation between external loads and internal force in element#1 of the truss which can be obtained by a simple equilibrium equation as follows:

$$V = \frac{F_1}{\tan\theta} + \frac{P}{\tan\theta} \quad (8)$$

In Equation 8, F_1 is the axial force in element 1, P is the gravity load and V is the in-plane lateral load.

On the other hand, Equation 2 which defines the constitutive equation for the in-plane shear strength of the URM walls can be rewritten as follows:

$$V_r = A_n\tau = A_nC + A_n\mu\sigma = A_nC + \mu P \quad (9)$$

Where V_r is the in-plane shear capacity of a URM wall and A_n is the transverse net area of the wall.

Comparison of Equations 8 and 9 confirms the similarity of these two equations. Consequently, the parameters θ and F_1 corresponding to the truss model are determined as follows by means of a simple analogy:

$$F_1 = A_nC\tan\theta \quad (10)$$

$$\theta = Arc\tan\left(\frac{1}{\mu}\right) \quad (11)$$

In this way, the failure of the wall can be simulated by a plastic hinge in vertical element of the truss with ultimate strength equal to F1 obtained from Equation 10. In summary, the truss model with three characteristic properties which are appropriate tensile load-deflection relationship for the vertical element and cross sectional area and inclination angle of the inclined element, can simulate the nonlinear behavior of a URM wall, Figure 3.

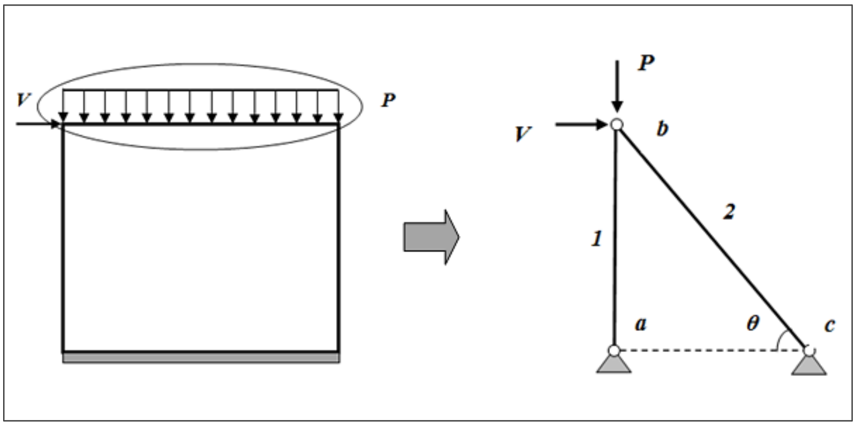


Figure 3. Definition of characteristic properties of equivalent truss model

4. Equivalent Truss Model for Combination of Urm Piers

To perform a nonlinear static analysis on URM buildings, piers in plan and elevation should be combined. For this purpose, first, each pier in plan is modelled with an equivalent truss as explained previously. The portion of the gravity load on each pier is calculated and inserted as a point load to the corresponding truss. For buildings with rigid diaphragms, top nodes of the trusses are constrained to displace equally in horizontal direction. This modeling procedure is illustrated in Figure 4. Finally, a nonlinear push over analysis can be performed on combination of piers and the push over curves are obtained. Performance of the building can be evaluated easily by comparison of the lateral deformation of each pier by permissible values corresponding to each performance level.

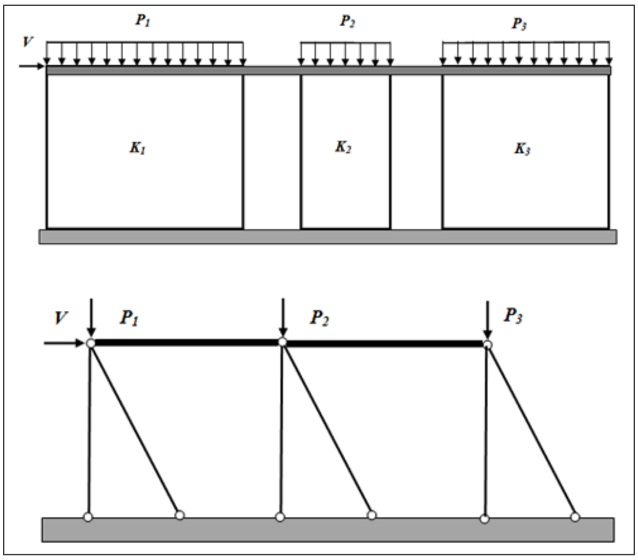


Figure 4. Equivalent truss model for combination of piers in plan

5. Equivalent Truss Model for Multi-Story Urm Walls

In multi-story buildings, the piers of the first story can be modelled following the procedure explained before. For these piers the gravity loads which are inserted to the equivalent trusses are sum of the total gravity loads from above floors and walls. The gravity loads are applied as a point load on the top node of each equivalent truss.

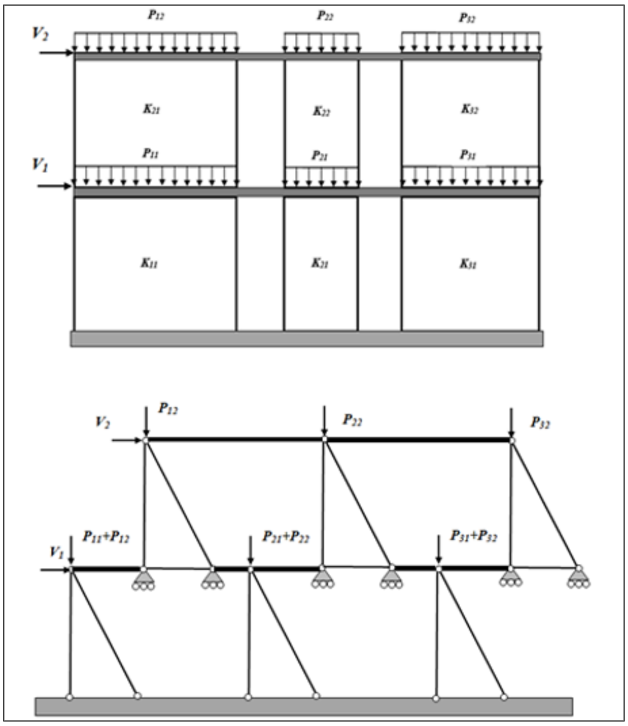


Figure 5. Equivalent truss model for multi-story buildings

The walls of other stories are simulated with tri-element trusses as illustrated in Figure 5. In each truss the vertical and inclined elements are the same as the truss model presented in previous section. Also, the mechanical and geometrical properties of these elements are determined in a similar way. The horizontal element which is rigid, constraints the bottom nodes of each truss to displace equally in horizontal direction. Since the equivalent trusses of upper stories can displace horizontally, roller supports are utilized for them. To enforce each floor level to move equally due to rigid diaphragm action, the nodes of the trusses in any floor level are constrained to displace equally. Similar to the first story, the gravity loads from above floors and walls are applied as a point load on the top node of each equivalent truss.

Finally, a nonlinear push over analysis with any lateral load pattern can be performed on combination of the walls. Also, the performance of the building can be evaluated simply by comparison of the lateral deformation of each pier by the permissible values for each performance level

6. Application of the Proposed Method in a Practical Case

The equivalent truss modeling procedure is used here to simulate a wall that consists of three piers with different dimensions and boundary conditions. For this purpose, a URM wall with opening pattern as illustrated in Figure 6 is considered. It consists of three piers with different lengths and effective heights. The boundary conditions of the piers are assumed according to the instructions in FEMA 356. The assumed mechanical properties of the masonry material are tabulated in Table 1.

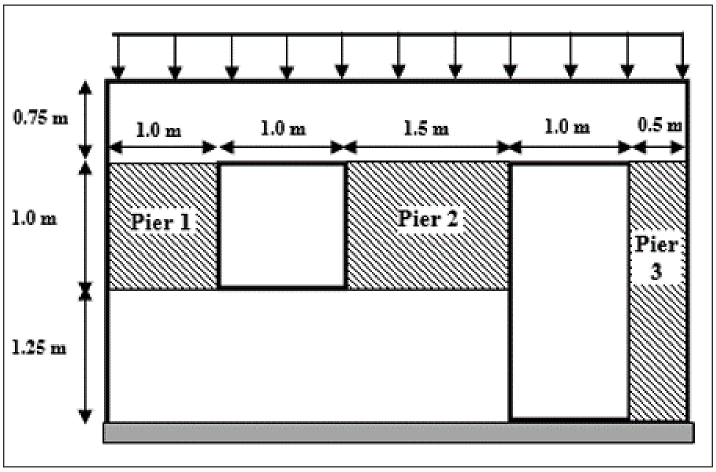


Figure 6. A Wall with three piers

Table 1. Material properties of masonry used for numerical studies of brick panels

Masonry Compressive Strength (MPa)	10.0
Modulus of Elasticity of Masonry (MPa)	5500.0
Shear Modulus of Masonry (MPa)	2200.0
Shear Cohesion of Brick-Mortar Bond (MPa)	0.1
Coefficient of Friction of Brick-Mortar Bond	0.75
Material Density (kN/m^3)	18.5

Table 2. Characteristic properties of equivalent trusses

	Pier 1	Pier 2	Pier 3
Inclination Angle (θ) (degree)	53.13	53.13	53.13
Vertical Element Length (mm)	3000.0	3000.0	3000.0
Inclined Element Cross Sectional Area (mm^2)	832278.0	1248423.0	208038.0
Modulus of Elasticity of Truss Material (MPa)	5500.0	5500.0	5500.0
Tensile Strength of Vertical Element (kN)	26.64	39.96	13.32

At first stage, the stiffness of each pier is calculated with appropriate formulations (Equation 1-b) according to their boundary conditions. Then, characteristic properties of the equivalent trusses such as plastic hinge capacity of the vertical element, cross sectional area and inclination angle of the inclined element for each equivalent truss are determined as explained before, see Table 2. Finally, a nonlinear static analysis is performed on combination of the piers and the push over curve is obtained, see Figure 8. The results of the push over analysis indicate that the ultimate capacity of the wall is equal to 111.47. It is similar to the value which can be estimated by hand calculation. The push over curve also shows that the first yielding occurs in piers 1 and 3 simultaneously and finally the pier 2 yields. The performance of the piers can also be evaluated with comparison of the top lateral displacement of each truss with allowable values suggested in different codes.

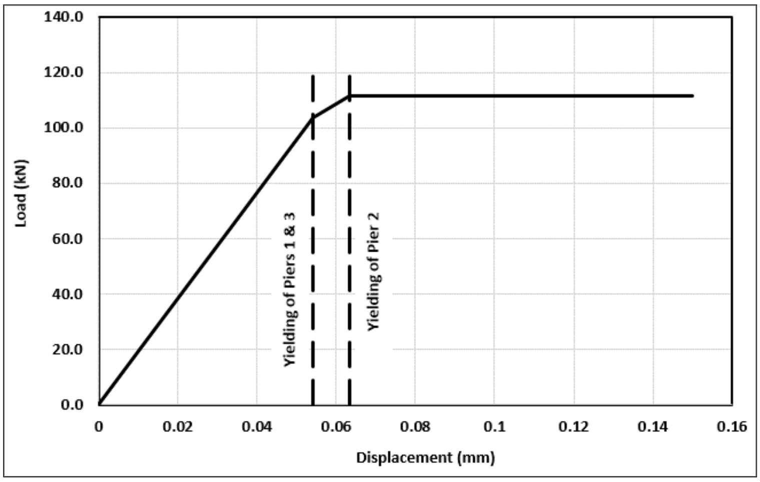


Figure 8. Push over curve of the wall obtained with equivalent truss method

7. Conclusion

An equivalent truss model for nonlinear static analysis of unreinforced masonry buildings is presented in this article. The model can be used if sliding shear failure with Mohr-Coulomb failure criteria is the governing in-plane failure mode in the piers. In this method, only truss elements with axial force and plastic hinges are used. This property makes the proposed model a simple modelling technique and suitable for seismic evaluation of existing URM buildings. The main advantage of the current model to the other similar methods is that the effects of the gravity load on the in-plane capacity of the piers are considered automatically and there is no need to calculate the capacity of each pier separately for definition of the plastic hinges. Also, results of the example was solved by this method show that this modelling approach gives results with good precision.

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