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Equivalent frame model and shell element for modeling of in-plane behavior of Unreinforced Brick Masonry buildings

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Abstract. Although performance based assessment procedures are mainly developed for reinforced concrete and steel buildings, URM (Unreinforced Masonry) buildings occupy significant portion of buildings in earthquake prone areas of the world as well as in IRAN. Variability of material properties, non-engineered nature of the construction and difficulties in structural analysis of masonry walls make analysis of URM buildings challenging. Despite sophisticated finite element models satisfy the modeling requirements, extensive experimental data for definition of material behavior and high computational resources are needed. Recently, nonlinear equivalent frame models which are developed assigning lumped plastic hinges to isotropic and homogenous equivalent frame elements are used for nonlinear modeling of URM buildings. The equivalent frame models are not novel for the analysis of masonry structures, but the actual potentialities have not yet been completely studied, particularly for non-linear applications. In the present paper an effective tool for the non-linear static analysis of 2D masonry walls is presented. The work presented in this study is about performance assessment of unreinforced brick masonry buildings through nonlinear equivalent frame modeling technique. Reliability of the proposed models is tested with a reversed cyclic experiment conducted on a full scale, two-story URM building at the University of Pavia .The pushover curves were found to provide good agreement with the experimental backbone curves. Furthermore, the results of analysis show that EFM (Equivalent Frame Model) with Dolce RO (rigid offset zone) and shell element have good agreement with finite element software and experimental results.

Keywords: Unreinforced Masonry buildings; equivalent frame modeling; pushover analysis; performance assessment

1. Introduction

In recent decades, numerical modeling of masonry structures behavior has been widely considered. Numerical methods used for continuum modeling like finite element methods are not able to simulate the behavior of such structures accurately, because they cannot estimate the dynamic behavior of separated elements and interactions between them (Abrams 1997). A certain number of methods have been utilized for the study of URM buildings so far. Due to the diversity and high level of complexity inherent to masonry, the approach towards the analytical modeling has led researchers to seek for several constitutive models characterized by different levels of complexity. From sophisticated finite element micro models to limit analysis approaches, a wide

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range of numerical methods are available in literature. Equivalent frame models and limit analysis methods are user friendly and require lesser amount of data. However compared to FEM models both of them is limited in terms of simulating distribution of nonlinearity, force redistribution, coupling effect between orthogonal walls, mode of failure prediction and so on. Although finite element models are the most reliable among all, the best method might be defined as “the method that provides the sought information in a reliable manner, i.e., within an acceptable error, with the least cost” (Oliviera 2003). Lourencho (1996) summaries finite element modeling strategies defined in literature depending on the level of refinement used for the structural analysis as below (see Fig. 1):

• Detailed micro-modeling – requires discrete modeling of mortar, brick units with continuum elements and unit mortar interface with discontinuous elements.

• Simplified micro-modeling – brick units are modeled with continuum elements whereas the behavior of the mortar joints and unit-mortar interface is lumped in discontinuous elements;

• Macro-modeling - units, mortar and unit-mortar interface are smeared out in the continuum.

2. Equivalent frame model

Equivalent frame method is a simple way to conduct nonlinear analyses on URM structures. Least amount of data is required to describe material property among other modeling strategies since homogenous, isotropic material idealization is made. Local nonlinear behavior of each wall is described with nonlinear hinges whose force displacement properties are usually defined from experimental test results. Being both simple and effective, a wide range of studies to improve the reliability of the EFM is found in the literature. Attempts to simulate nonlinear behavior of URM with equivalent frame models are summarized below: Gilmore *et al*. (2009) proposed an equivalent frame model to perform pushover analysis of confined masonry buildings. Structural degradation of confined masonry walls is associated with shear behavior and a rotational shear spring to idealize nonlinear response of masonry walls is proposed. Rotational spring is used to relate shear force on the wall with inter-story drift due to shear deformation. For this purpose hinge is located at the bottom of the wall (see Fig. 2).

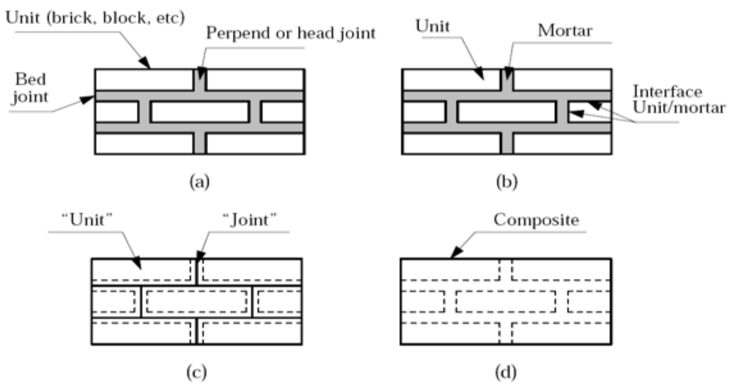

Fig.1 Modeling strategies for Brick Masonry: (a) typical masonry sample, (b) detailed micro modeling, (c) simplified micro-modeling, (d) macro-modeling (Lourencho 1996)

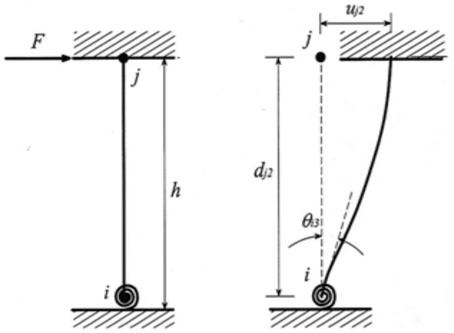

Fig. 2 Modified wide column model for PO analysis (Gilmore e*t al*. 2009)

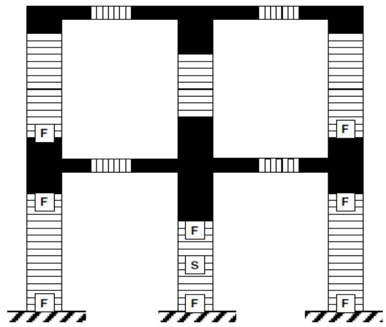

Fig. 3 Spread nonlinearity approach in EFM (Belmouden and Lestuzzi 2007)

Kappos *et al*. (2002) conducted elastic and plastic comparative analyses on two and three dimensional masonry structures aiming to evaluate accuracy of equivalent frame modeling technique. Salonikios *et al*. (2003) conducted comparative inelastic analyses on nonlinear equivalent frames and finite element models of 2D masonry frames. Influence of different lateral force distributions on pushover analysis of masonry frames is investigated due to the fact that important fraction of the total mass is distributed along the wall height in masonry buildings which makes it harder to determine load distribution during pushover analyses. Pasticier *et al*. (2007) aimed to utilize SAP2000 for seismic analyses of masonry buildings using EFM. In nonlinear modeling of masonry piers, two rocking hinges at the end of the rigid offsets and one shear hinge at the middle of the pier is used. On the other hand, only one shear hinge was introduced for nonlinear modeling of spandrels. Belmouden and Lestuzzi (2007) come up with and equivalent frame model for seismic analysis of masonry buildings. Unlike other proposed models up to the present, analytical model is based on smeared crack and distributed plasticity approach. Moreover interaction between both axial force-bending moment and axial force shear force are considered. Inelastic flexural as well as inelastic shear deformations are allowed for piers and spandrels. Translational shear springs are added at the middle of the span and flexural hinges are added at the ends of the span. However piers and spandrels are discretized into series of slices, nonlinearity is distributed along the length of the spans (see Fig. 3).

Roca *et al*. (2005) studied 2D wall panels as equivalent systems of one-dimensional members, namely equivalent frames. Force deformation characteristic of masonry in compression is modeled with Kent and Park model. Axial force-shear force interaction is considered through use of Mohr-Coulomb criterion as biaxial stress envelope. Penelis (2006) developed a method for pushover analysis of URM buildings using EFM. Rotational hinges using lumped plasticity approach are utilized at the ends of structural elements for nonlinear action. Magenes and Fontana (1998) proposed a method named as SAM (simplified analysis of masonry buildings) for simplified non-linear seismic analysis of masonry buildings through equivalent frame idealization of URM walls subjected to in-plane loadings. Constitutive relation of structural members is idealized as elastic-perfectly plastic where shear strength of members are calculated from simple strength equations in literature. A limit to total chord rotation (i.e., flexural rotation plus shear rotation) is assigned as 0.5% for shear failures and 1% for flexural failures. An effective height is used for structural elements in terms of rigid end offsets proposed by Dolce (1989) for the definition of the stiffness matrix in the elastic range.

3. Shell element method

This method was presented by Sweeny (2004) in modeling a firehouse. In this method masonry walls were modeled using shell elements. Shell elements have linear behavior in Sap2000 and the nonlinear behavior of shell elements is not defined in Sap2000; therefore, the nonlinear behavior of masonry walls was defined using frame elements in the critical place of masonry wall based on failure modes of masonry walls.

4. Shell element and equivalent frame modeling of URM walls

Masonry buildings are composed of internal and external walls. Internal walls are usually solid but in most cases peripheral walls are perforated as a result of both door and window openings. Structural components on perforated masonry walls are named as piers or spandrels due to their orientation (see Fig. 4).

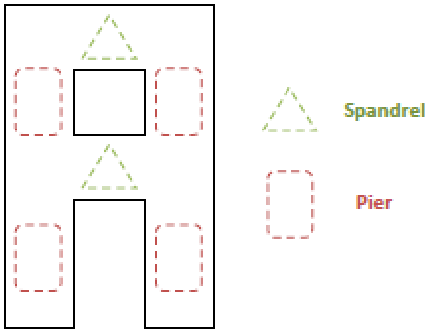

Fig. 4 Spandrels and piers on a perforated wall

Among the different modeling approaches to model masonry walls, equivalent frame modeling will be investigated in detail. In equivalent frame modeling method, each pier and spandrel is modeled with frame elements passing through their centerline (see Fig. 6(a)). Since cross sectional and mechanical properties of each member is squeezed to a line element, we expect each equivalent frame element to reflect similar structural behavior with their counterpart at perforated wall. As it enables to implement displacement based concepts, equivalent frame method is frequently used for the modeling of masonry buildings in the literature. Compared to more sophisticated finite element models, equivalent frame models are simple and easy to apply. Besides, according to Magenes and Fontana (1998), “equivalent frame idealization of masonry structures are effective for; good prediction of strength of a building subjected to a pattern of increasing horizontal forces, good prediction of the failure mechanism in the single sub elements and good prediction of the overall deformation of the building particularly at the ultimate state.”

5. Determination of effective height for Masonry piers and spandrels

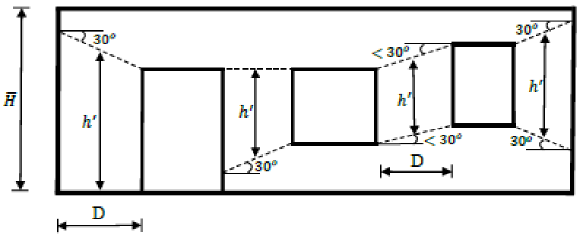

Fig. 5 Effective height determination offered by Dolce

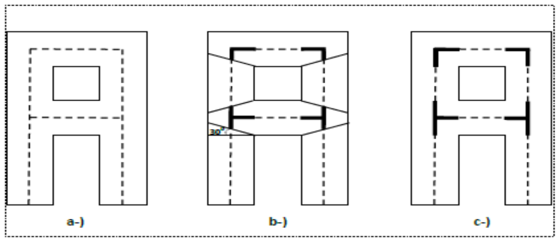

Fig. 6 (a) Equivalent frame model, (b) EFM with Dolce RO, (c) EFM with Full RO

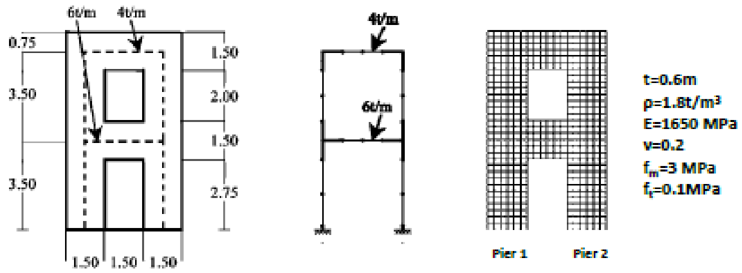

Fig. 7 1B2S Masonry frame investigated by Salonikios *et al.* (2003)

Although it is easy to idealize each pier and spandrel as equivalent frames with their cross section dimensions, height and mechanical properties, defining connection between them is challenging. In order to take coupling effect between piers and spandrels into account rigid end offsets are assigned at the ends of frame elements. Assigning full RO for spandrels is a widely used assumption. However, being the most important element of in plane load carrying mechanism, RO length of piers should be carefully assigned. Different methods for assigning rigidity at pier-spandrel interaction are found in the literature. One method proposed by Dolce (1989) is to take a portion of pier-spandrel interaction as rigid (see Fig. 5) whereas another approach is to take pier - spandrel interaction as fully rigid (see Fig. 6). In order to decide which approach to use, a comparative study will be performed. The aim is to determine the closest approximation to finite element results of a perforated frame by equivalent frame models whose rigid end offset patterns are variable. For this purpose, 4 different perforated frames are modeled with different modeling approaches and results are compared. Three criteria will be checked for comparison. Namely, story displacements, axial force on base piers and shear force on base piers. ABAQUS (V.6.9.1) is utilized for finite element modeling and SAP2000 (v14) is utilized for equivalent frame modeling and shell element.

6. Comparative elastic analysis of 1 bay 2 story (1B2S) perforated masonry frame

1 bay 2 story masonry frame whose nonlinear behavior is investigated by Salonikios *et al.* (2003) is chosen for linear comparative analysis (see Fig. 7).

Plane stress assumption is made for finite element modeling in ABAQUS. The results of analysis were shown in Table1.Linear beam element is used for equivalent frame modeling (EFM) in SAP2000 and shell thin element for modeling masonry wall using shell element method. Analyses are conducted on three different rigid end offset (RO) alternatives (see Figure 8a, 8b and 8c). Loading on each model is imposed in a two-step sequence. First, dead load plus distributed slab loading on spandrels are imposed. Second, lateral loads at story levels which sum up to 15% of total weight is applied in proportion to first mode story displacements calculated using EFM with Dolce offset. Floors are assumed to be rigid so diaphragm constraints are assigned at floor levels.

Results of the analyses are summarized in Table 1. As it is clearly seen, RO proposed by Dolce (1989) gives the best approximation to finite element analyses considering deflected shape, axial force and shear force on base piers. Not assigning any rigid end zone results in a more flexible behavior compared to finite element analysis. On the contrary assigning full rigid end offset results in a stiffer behavior. For a better comparison between FEM and its best estimator EFM with Dolce offset, lateral story displacements are plotted below (see Fig. 9).

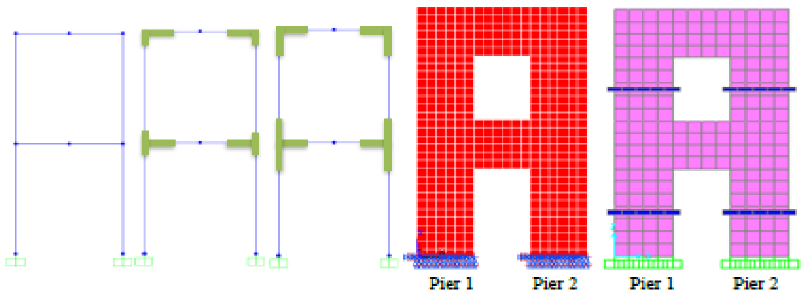

Fig. 8 2B3S Frame models (a) EFM without RO, (b) EFM with Dolce RO, (c) EFM with full RO, (d) FEM coarse mesh, (e) shell element method

Table 1 Analysis Results of FEM and EFM for 1B2S Masonry frame

		ABAQUS (FEM)	SAP(EFM) No RO	SAP(EFM) Dolce RO	SAP(EFM) Full RO	Difference (%) No RO	Difference (%) Dolce RO	Difference (%) Full RO
Axial Force(KN)	Pier1	240	250	238	230	4.2	-0.83	-4.2
	Pier2	510	489	505	513	-4.1	-0.98	0.588
Base shear(KN)	Pier1	50	45	47	43	-10	-6	-14
	Pier2	78	70	74	76	-10.25	-5.128	-2.56
Lateral Roof Displacement	1st Floor	1.8	2.04	1.74	1.6	13.33	-3.33	-11.1
	2nd Floor	3.8	4.05	3.65	2.45	6.57	-3.947	-35.52

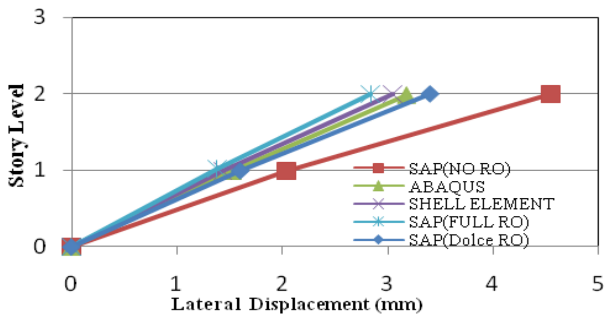

Fig. 9 Lateral story displacements of FEM and EFM on 1B2S Masonry frame

7. Comparative elastic analyses of 2 bay 2 story (2B2S) perforated Masonry frame with strong spandrels

Same analyses that were conducted on 1B2S masonry frame are also conducted for 2B2S masonry frame (see Fig. 10). Meshing of the finite element model and rigid end offsets of equivalent frame models are drawn in Fig. 11.

Results of the analyses are summarized in Table 2. Considering the shear and axial force on base piers, both Dolce and Full rigid end offsets give satisfactory results. Lateral displacement of first story which is most critical story under lateral loads is best approximated by Dolce offset. Again FEM results are in between full RO and Dolce RO. For a visual comparison between FEM and its best estimator EFM with Dolce RO, lateral displacement at the base is drawn below (see Fig. 12).

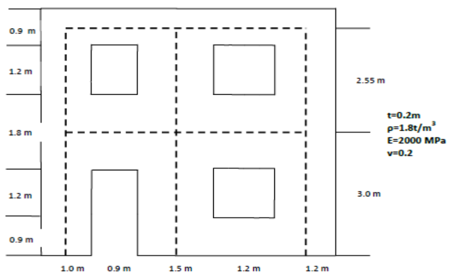

Fig. 10 2B2S Masonry frame with strong spandrels

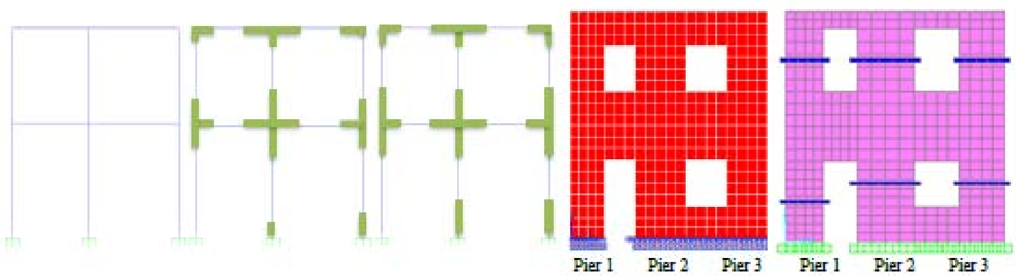

Fig. 11 Models 2B2S Masonry frame (a) EFM without RO, (b) EFM with Dolce RO, (c) EFM with full RO, (d) FEM, (e) shell element method

Table 2 Analyses results of FEM and EFM for 2B2S Masonry frame

		ABAQUS	SAP(EFM)			Difference (%)		
		(FEM)	No RO	Dolce RO	Full RO	No RO	Dolce RO	Full RO
Axial Force(KN)	Pier1	240	250	238	230	4.2	-0.83	-4.2
	Pier2	510	489	505	513	-4.1	-0.98	0.588
Base shear(KN)	Pier1	50	45	47	43	-10	-6	-14
	Pier2	78	70	74	76	-10.25	-5.128	-2.56
Lateral Roof Displacement	1st Floor	1.8	2.04	1.74	1.6	13.33	-3.33	-11.1
	2nd Floor	3.8	4.05	3.65	2.45	6.57	-3.947	-35.52

Results are summarized in Table 2. Similar to previous analyses considering the finite element model, SAP2000 model without RO results in larger story displacements indicating that equivalent frame without RO is flexible. Full RO model result in smaller displacements indicating that model is stiffer. Dolce RO is the best approximation to story displacements. It also approximates base shear and axial force in base piers satisfactorily.

8. Comparative elastic analyses of 2 bay 3 story (2B3S) perforated Masonry frame with weak spandrels

2 bay 3 story masonry frame whose nonlinear behavior is investigated by Roca *et al*. (2005) is chosen for comparative linear analyses (see Fig. 13). Same analyses that were conducted on 1B1S and 1B2S frames are conducted again. Meshing of the finite element model and RO patterns of equivalent frame model are drawn below (see Fig. 14).

Results are summarized in Table 3. Similar to previous analyses considering the finite element model, SAP2000 model without RO results in larger story displacements indicating that equivalent frame without RO is flexible. Full RO model result in smaller displacements indicating that model is stiffer. Dolce RO is the best approximation to story displacements. It also approximates base shear and axial force in base piers satisfactorily (see Fig.15).

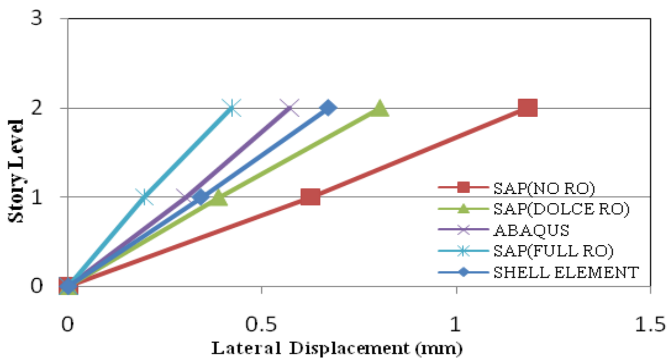

Fig. 12 Lateral story displacements of FEM and EFM on 2B2S frame

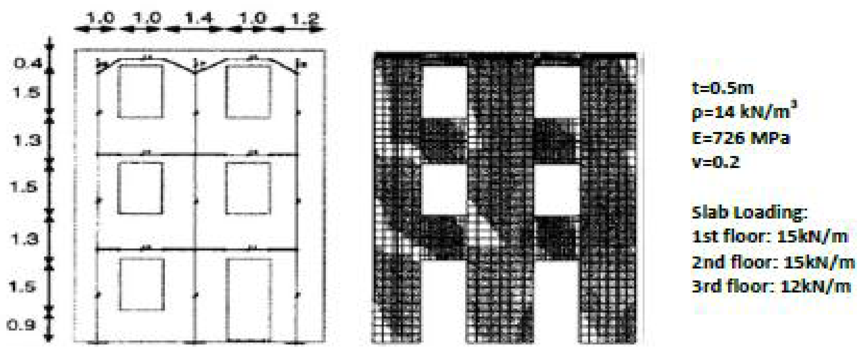

Fig. 13 2B3S Masonry frame investigated by Roca *et al*. (2005)

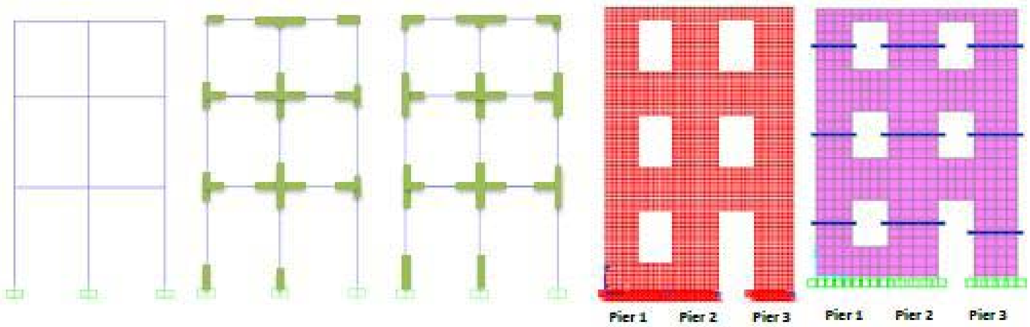

Fig. 14 2B3S Frame models (a) EFM without RO, (b) EFM with Dolce RO, (c) EFM with Full RO, (d) FEM, (e) shell element method

Table 3 Analyses results of FEM and EFM for 2B3S Masonry frame

		ABAQUS (FEM)	SAP(EFM)			Difference (%)		
			No RO	Dolce RO	Full RO	No RO	Dolce RO	Full RO
Axial Force(KN)	Pier1	53	71.24	71.4	74.8	34.41509	34.71698	41.13208
	Pier2	211.5	206.76	200.3	195.7	-2.24113	-5.29551	-7.47045
	Pier3	234	218.5	228.4	225.8	-6.62393	-2.39316	-3.50427
Base shear(KN)	Pier1	18.9	14.2	16.9	18.4	-24.8677	-10.582	-2.6455
	Pier2	32.5	35.3	40.3	43.5	8.615385	24	33.84615
	Pier3	23.4	24.8	17.2	12.4	5.982906	-26.4957	-47.0085
Lateral Roof Displacement	1st Floor	1.51	2.57	1.28	0.98	70.19868	-15.2318	-35.0993
	2nd Floor	3.08	5.41	2.71	2.22	75.64935	-12.013	-27.9221
	3rd Floor	4.34	7.6	3.83	3.27	75.11521	-11.7512	-24.6544

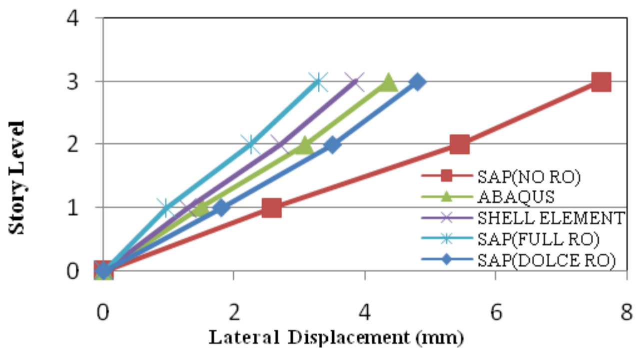

Fig. 15 Lateral story displacements for FEM and EFM analyses on 2B3S Masonry frame

As a result of elastic linear analyses on perforated frames with different RO patterns, it might be concluded that considering story displacements, axial load on base piers and shear force on base piers, best approximation to finite element model is SAP2000 model with Dolce RO. Comparison of shear force (see Fig. 16) and axial load at base piers (see Fig. 17) between finite element method and its best approximation; EFM with Dolce RO are plotted below.

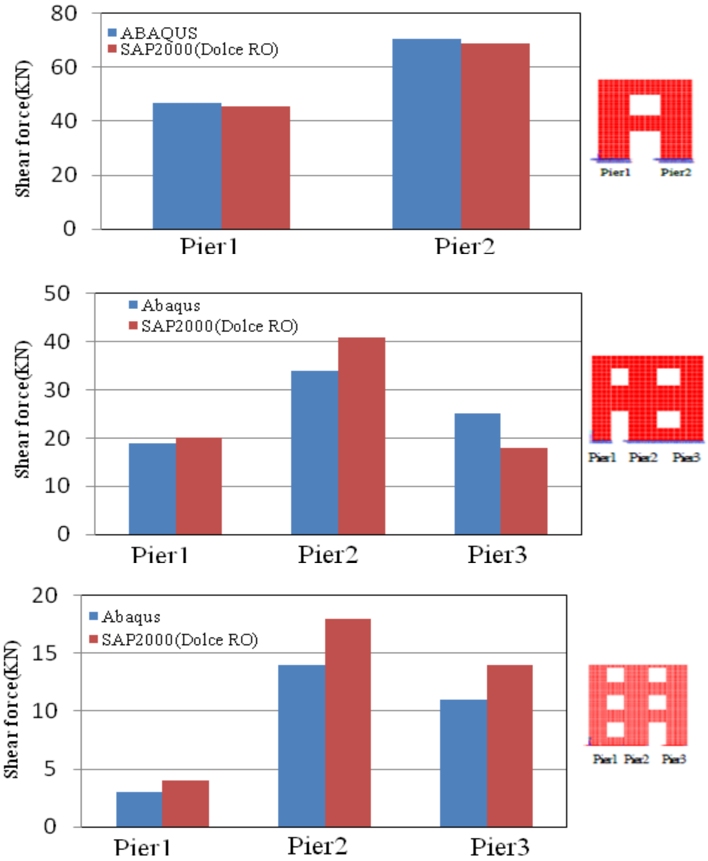

Fig. 16 Comparison of shear forces on base piers of Masonry frames

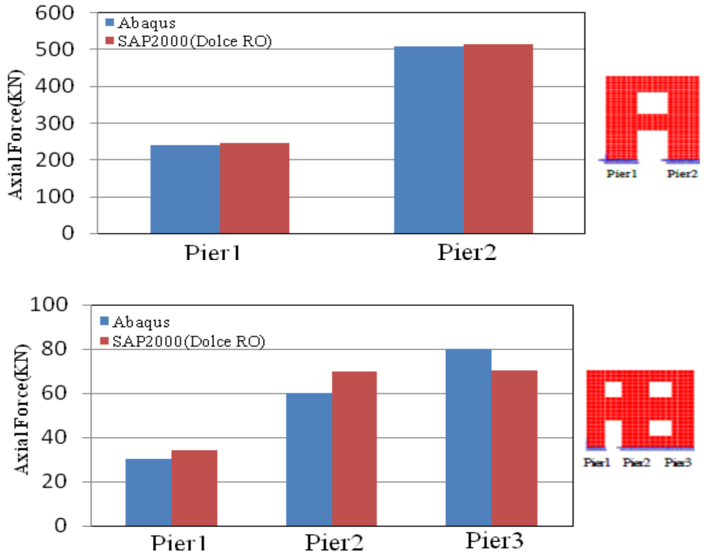

Fig. 17 Comparison of axial load on base piers of Masonry frames

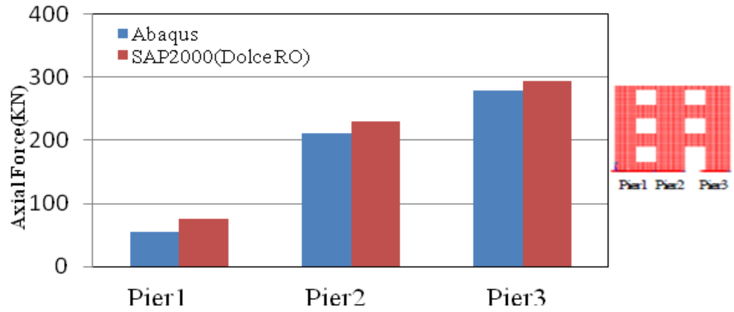

Fig. 17 Continued

Table 4 Mechanical properties adopted in the numerical analyses

		Pavia	
		masonry	
Young's modulus	E	[Mpa]	1400
Shear modulus	G	[Mpa]	480
copmpressive strength	f_{wc}	[Mpa]	6.2
Shear Strength	f_{wd0}	[Mpa]	0.18
		Brick Units	
Tensile strength	f_{bt}	[Mpa]	1.22
		Mortar joints	
Cohesion	c	[Mpa]	0.23
Friction coefficient	μ		0.58

8.1 Nonlinear equivalent frame modeling approach for Masonry buildings

For the equivalent frame modeling of masonry buildings, well known computer software SAP2000 (2009) will be utilized. Nonlinear material behavior is available through the use of frame hinges which might be defined manually. Software is capable of conducting nonlinear static analyses through these hinges where all plastic deformation occurs within the point hinge (i.e., lumped plasticity). Assumption made for the modeling is; nonlinearity is restricted to masonry piers only. Spandrels remain elastic through analysis but piers pass into nonlinear range when they are pushed above the elastic limit. This is because very little experimental information is available on cyclic behavior of unreinforced masonry beams, especially regarding the deformational behavior (Magenes and Fontana 1998) and ultimate failure of the masonry buildings is controlled by piers.

8.2 Verification of proposed computer model

Test conducted on masonry specimen at the University of Pavia is the most preferred reference for the verification of different URM modeling techniques in the literature. After definition of experimental work, results of proposed nonlinear equivalent frame model will be compared with the results of these experiments for the purpose of verification. For all the comparisons described in this section, the mechanical properties adopted in the analyses are summarized in Table 4.

A very detailed experimental test has been carried at the University of Pavia, Italy by Magnes *et al.* (1995). A full scale, two-storey URM building prototype (plan dimension 6.00 × 4.40 m) has been tested by applying cyclic displacements at floor levels (see Fig. 18), such to obtain a distribution of lateral forces proportional to seismic weights (in addition to gravity loads: 248.8 kN at first floor, 236.8 kN at second floor). The prototype contains an almost independent shear wall ("Pavia Door Wall") which has been an interesting benchmark for many authors. Pushover analysis was used for comparing results. Nonlinear behavior of masonry walls were defined according to failure modes. Plastic hinges were used for modeling of nonlinear behavior of masonry walls based on FEMA356. In Figs. 19-21 the comparison between the results of analysis and the experimental test is depicted. The comparison shows a satisfactory agreement between the experimental test and the proposed code; moreover, a general agreement with all the models is present.

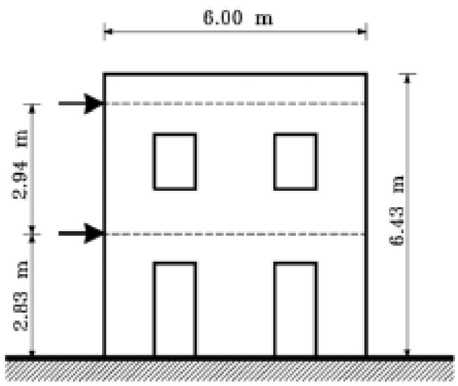

Fig. 18 Pavia door wall testing scheme

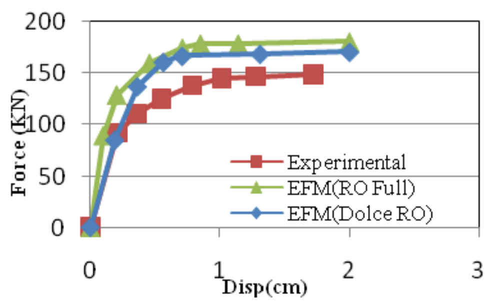

Fig. 19 Total base shear vs. top displacement curves: Pavia door wall

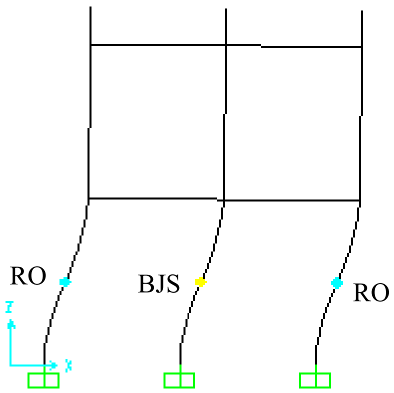

Fig. 20 Crack patterns from numerical results at 17 mm

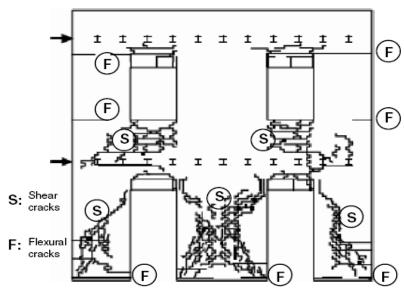

Fig. 21Crack patterns from the experimental test of the URM building (at failure state (top displacement equal to 24 mm))

9. Modeling of masonry structures using shell elements and EFM

In this study, masonry structures were modeled using shell elements and EFM (Dolce Rigid offset zone) in SAP2000.ver14.0.0. Nonlinear static analysis has been used and plastic hinges were used for defining nonlinear behavior of masonry walls based on failure modes. Shear plastic hinges had been assigned to masonry walls for nonlinear static analysis in the middle of the masonry walls. In this study proposed method by Dolce was used in modeling masonry structures using EFM. The structure which was modeled in this study was depicted in Figs. 20 and 21.

To perform a pushover analysis, the nonlinear load-deformation response must be specified for each structural element that could potentially yield. The nonlinear behavior of individual piers was found based on FEMA 356, and this information was used to assign hinge properties for models created in SAP2000. Force-deformation relations shall be based on experimental evidence or the generalized force-deformation relation shown in Fig. 22. The lateral pushover loads applied was proportional to the fundamental mode of the structure in each horizontal direction for the 3D cases. In addition, the SAP pushovers include the dead load of the test model and P-delta effects. The lateral strength of the piers in URM walls depends on the mode of failure. FEMA 356 recognizes four different types of failure. The four primary in-plane failure modes of URM walls such as rocking, bed joint sliding, diagonal tension failure along masonry units or along head and bed joints in a stair stepped fashion and toe crushing are identified in these studies. Rocking and sliding govern the response under low levels of axial force and at high aspect ratios. These failure modes are capable of exhibiting large ultimate drifts. At higher levels of axial force and low aspect ratios, toe-crushing and diagonal tension failures are more common. According to FEMA356,

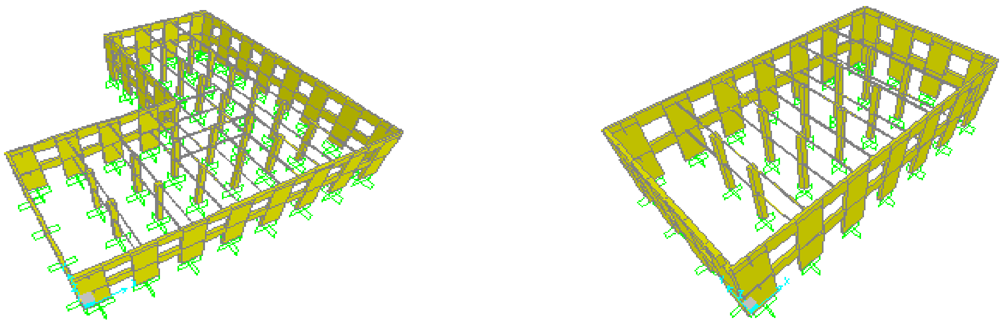

Fig. 20 Equivalent frame model for masonry walls (L & regular shape)

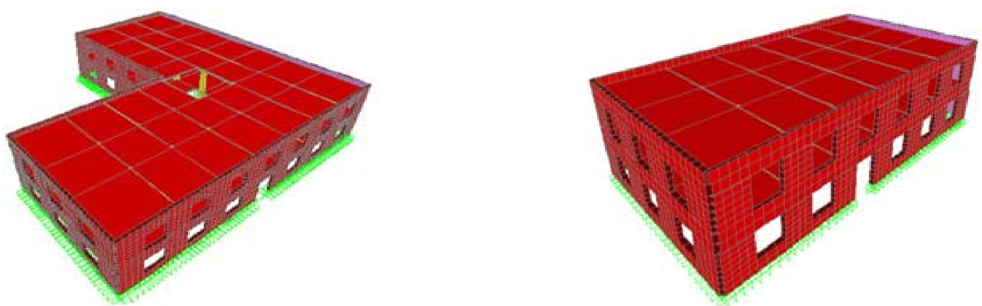

Fig. 21 Shell element model for masonry walls (L & regular shape)

rocking and sliding are described as displacement-based; therefore, theses failure modes were assigned to the masonry wall. Figs. 23 through 28 show analysis results for both triangle and uniform load pattern.

Table 4 shows the mechanical properties used in the structures. According to the material properties, the nonlinear behavior of masonry walls was determined and assigned to structures using plastic hinges. V2 hinges were assigned to the structure based on the failure modes in the masonry walls. Table 6 shows the nonlinear behavior of masonry structure which has been calculated based on failure modes.

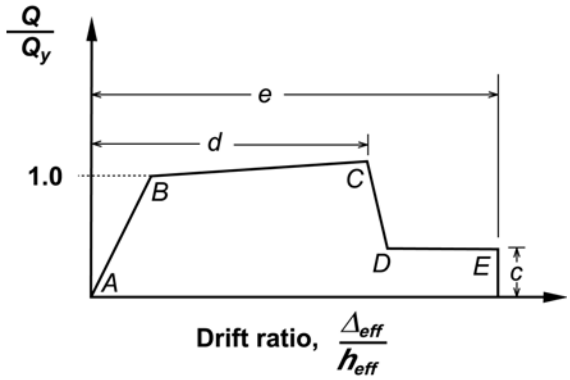

Fig. 22 Generalized force-deformation relation for Masonry elements or components

Table 5 Mechanical properties

		Pavia Masonry wall	
Young 's modulus	E	[Mpa]	3410
Shear modulus	G	[Mpa]	1364
copmpressive strength	f_{wc}	[Mpa]	6.2
Shear Strength	f_{wd0}	[Mpa]	0.18

Table 6 The property of nonlinear behavior in regular masonry structure

wall	Failure Mode	Disp SF(m)	Force SF(kgf)	C.P	L.S	I.O	Point E	Point D	Point C
1	Bed Joint Sliding	3	2271	3.73E-03	2.73E-03	7.29E-04	7.73E-03	3.73E-03	0.006
2	Bed Joint Sliding	3	3679.5	0.00356	0.00256	5.61E-04	0.00756	0.00356	0.006
3	Diagonal tension		3148						
4	Diagonal tension		3148						
5	Bed Joint Sliding	3	3639.6	0.00356	0.00256	5.61E-04	0.00756	0.00356	0.006
6	Bed Joint Sliding	3	3664	0.00356	0.00256	5.61E-04	0.00756	0.00356	0.006
7	Bed Joint Sliding	3	2272	3.73E-03	2.73E-03	7.29E-04	7.73E-03	3.73E-03	0.006

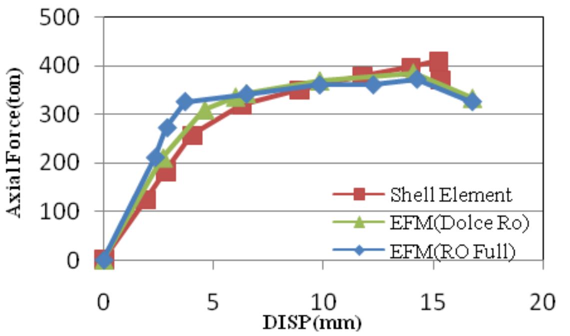

Fig. 23 Pushover curves for the building under 0.9dead (Ex) in the +X direction based on TLP

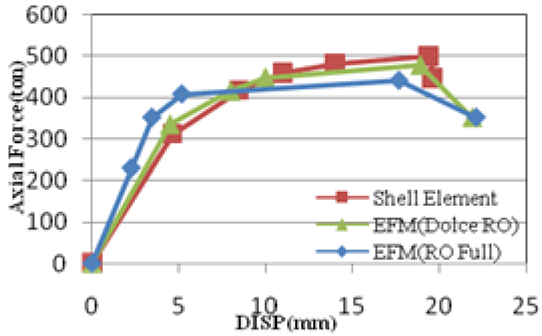

Fig. 24 Pushover curves for the building under 0.9dead (Ex) in the +X direction based on ULP

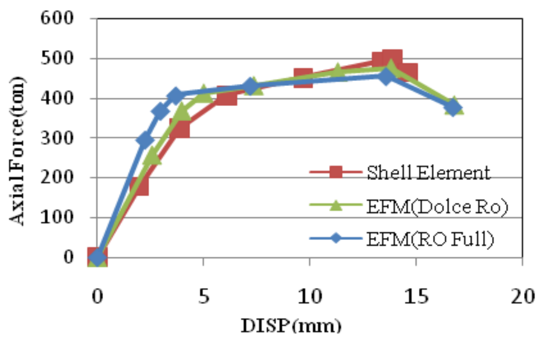

Fig. 25 Pushover curves for the building under 0.9dead (Ex+0.3Ey) in the +X direction based on TLP

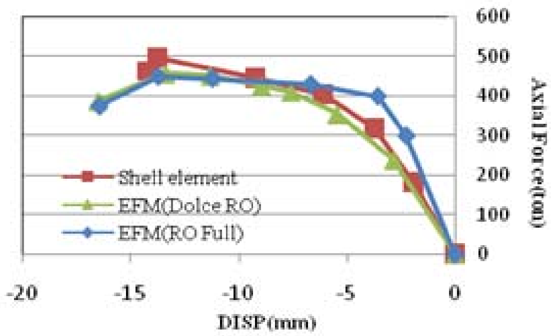

Fig. 26 Pushover curves for the building under 0.9dead (Ex+0.3Ey) in the -X direction based on TLP

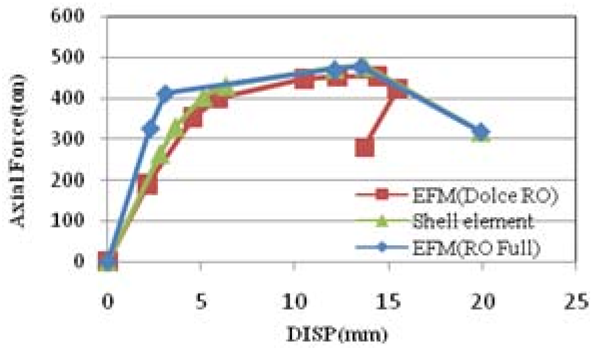

Fig. 27 Pushover curves for the building under 0.9dead (Ey+0.3Ex) in the +y direction based on TLP

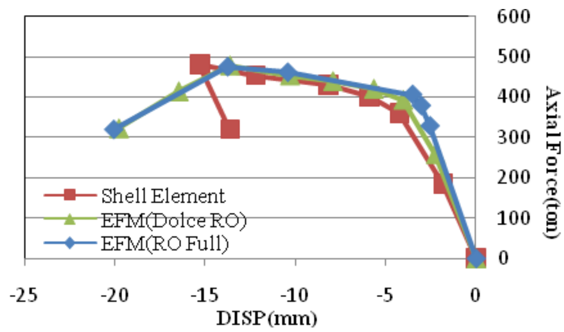

Fig. 28 Pushover curves for the building under 0.9dead (Ey+0.3Ex) in the -y direction based on TLP

The results of the analysis show that Equivalent frame method (proposed by Dolce) and shell element do not have any difference in failure modes , the initial stiffness of structural and yield strength because failure modes in masonry walls are determined based on the lowest strength but modeled structures using EFM(RO Full) has a significance difference in initial stiffness. Also modeled masonry structures using EFM (proposed by Dolce) and Sweeney method have significance difference in the position of the plastic hinges.

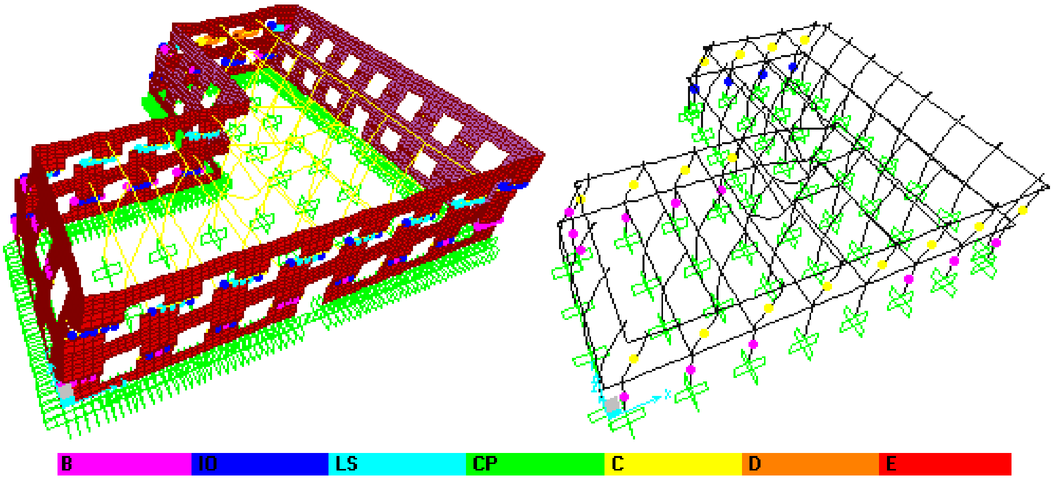

Fig. 29 Position of the plastic hinges at a target displacement of 15 mm (Ex+0.3Ey) in the +X direction for the BSE-1

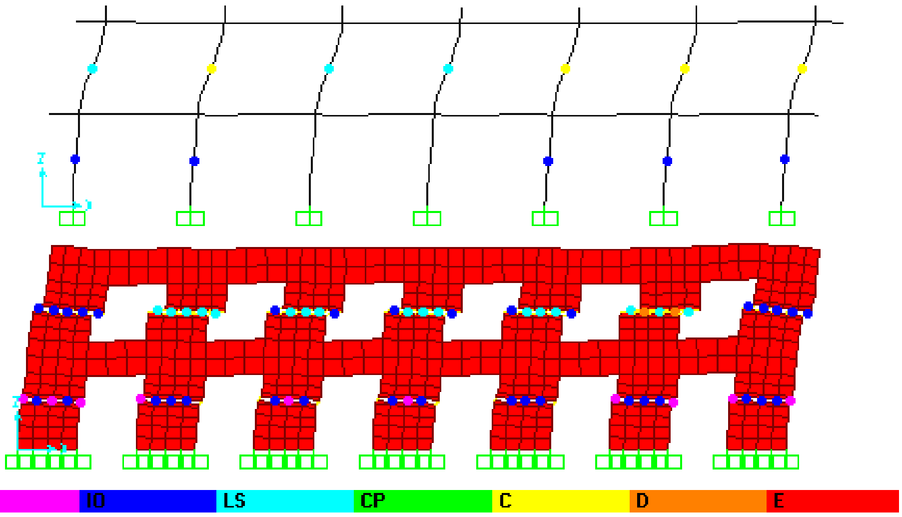

Fig. 30 Plastic hinges at a target displacement of 15 mm (Ex) in the +X direction for the BSE-1 based on TLP

10. Conclusions

In this study the modeling techniques of masonry structures were presented. Masonry structures were modeled using EFM and shell element. Modeling of masonry structures using EFM was done in two states:1) Equivalent frame method (Dolce RO) 2) Equivalent frame method (RO Full). Based on the analysis conducted in this study, the following conclusions can be drawn:

1) Due to the high complexity and time-consuming nature of finite element methods for modeling masonry structures, shell element method is a suitable method for the evaluation and retrofit ting masonry structures. In addition, the analysis is highly time efficient, taking less than

two minutes to run, although construction of the model requires some technical skill and time input. The advantage of accuracy outweighs the simplicity of the current simplified methods, whilst still being superior in efficiency and time over finite element modeling.

2) Analyses show that the use of rigid offsets is a crucial issue in equivalent frame modeling. The dimensions of rigid offsets in piers are calculated based on an empirical approach proposed by Dolce. As displayed in the rigid offsets have a significant effect on the global response not only on stiffness, but also on strength capacity of the structure. This study shows that the results of analysis using approach proposed by Dolce have good agreement with shell element in initial stiffness, yield and ultimate strength.

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