



Equivalent Frame Modelling of an Unreinforced Masonry Building in Finite Element Environment

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Abstract. Equivalent frame method (EFM) is a viable modelling option for global seismic analysis of masonry buildings in comparison to more refined techniques, such as finite elements (FE), especially in professional practice [1–3]. EFM takes advantage of a building’s geometric regularity, both in plan and elevation, as well as of the good quality of masonry and floors stiffness, as required for the activation of box-like behaviour under seismic loads. However, typical vulnerabilities in existing unreinforced masonry (URM) buildings, e.g. highly flexible floors, openings too close one another, poor quality masonry, isolated pillars or non-vertically aligned walls, limit the effectiveness of EFM application. Recently, many studies have been devoted to expanding the possibilities of applying EFM to buildings which do not meet box-behaviour hypotheses [4–6].

The paper describes the procedure for implementing an EF model of an existing URM building in Midas GEN, a FE software commonly used for design of steel and r.c. structures. The equivalent frame (piers and spandrels) consists of a system of mono-dimensional, lumped plasticity beam elements [7]. In MIDAS, the frame is defined by the user, who also have to control the modelling process, by using the theoretical criteria available [8–10] and adapting their results to a building’s characteristics. Therefore, some peculiar vulnerabilities of the original building may be specifically implemented, thus obtaining a more refined model.

The case study is Palazzo Carraro, a cultural heritage building, located close to the old town of Noale (Venezia). The palace complies with the main requirements of EFM except for floors stiffness, although horizontal connections may be considered sufficient at this level of analysis.

Pro and cons of the specific procedure are here discussed, also referring to other state-of-the-art techniques, such as continuum models [11] implemented through the DIANA FEA code. Finally, the work explores the response of the EF model to different modelling choices, but also its reliability in overall analyses.

Keywords: Equivalent frame model · Unreinforced masonry · Cultural heritage

1 Introduction

The equivalent Frame Model (EFM) method refers to the set of simplified approaches used for the seismic analysis of masonry building. Use of these methods is permitted provided the following conditions are met [12]: (a) geometric regularity, both in plan and in elevation of the building; (b) good quality of the masonry; (c) rigid diaphragms; (d) concentration of masses at floors height.

These conditions imply that: (i) the box-like behaviour is obtained; (ii) the macro-element discretization of the walls is possible; (iii) the dynamic properties depend mainly on the first mode.

In the following, the generalities of the geometrical procedure needed to define the equivalent frame of a masonry structure are given and then used to implement the model in MIDAS GEN software. Then equivalent mechanical properties of timber floor diaphragms under different structural conditions are defined from data available in literature. The parameters obtained are also used to implement the same model in TREMURI, another equivalent frame-based software, and DIANA, a state-of-the-art finite element tool, to give a complete comparison, in both, the same and a different field of modelling.

The paper aims to give an insight into the quality of the model defined in MIDAS GEN in comparison to more widespread options, through the means of the model's overall behavior, evaluated through linear modal analysis and non-linear static analysis (pushover).

1.1 Equivalent Frame Modelling of Unreinforced Masonry Walls

In a good quality masonry building, that is fulfilling the above-mentioned criteria, usually cracks appear in specific areas of masonry walls.

According to their behaviour these areas are identified as follows (Fig. 1):

- (a) *Piers*, which are the main vertical resistant elements, carrying both vertical and horizontal loads;
- (b) *Spandrels*—the parts of wall between two vertically aligned openings—which couple the response of adjacent piers in the case of lateral loads;
- (c) *Rigid nodes*, parts of the wall that correspond to the intersection between piers and spandrels which are not usually subject to damage.

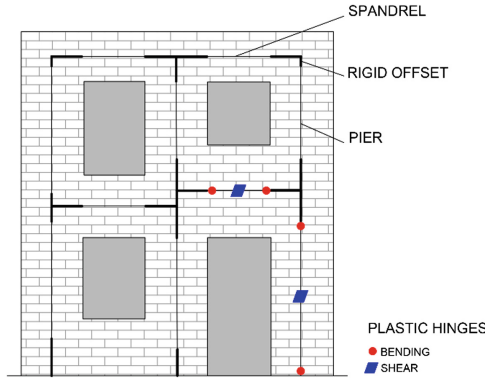


Fig. 1. Nomenclature of equivalent frame elements. Hinges are reported for example only

Piers play the most important role in the structural behaviour of the EF since they are the main seismic resistant elements of the structural system. Therefore, the definition of the *effective height* of the piers is the key to the correct discretization of the equivalent frame representing the building. Different criteria are available in order to convert the structure into the equivalent frame, according to masonry quality, openings layout etc. The so-called ‘metodo Dolce’ [9, 10] is one of the most common ones. This method is used for quite flexible structures since it limits the dimension of the rigid nodes, thus lengthening the piers. In fact, considering that cracks develop starting from the corners of the openings, this criterion empirically defines an effective height (H_{eff}) as a function of a geometrical parameter (h'), that is defined as the distance between the midpoints of the line connecting the vertices of two consecutive openings (Fig. 2):

$$H_{eff} = h' + \frac{1}{3}D \frac{(\widehat{H} - h')}{h'} \tag{1}$$

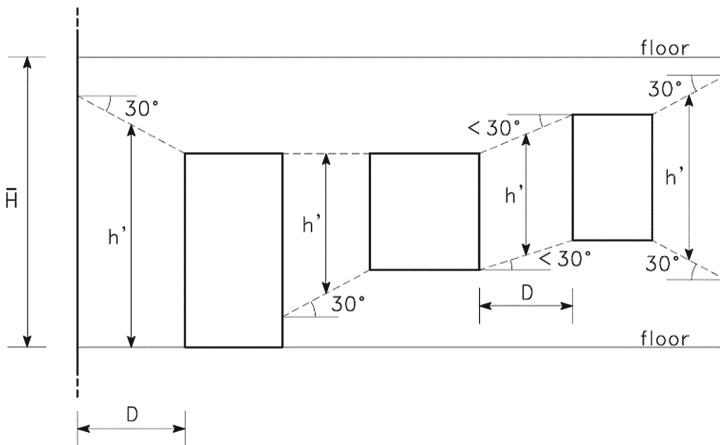


Fig. 2. Definition of effective height of piers with ‘metodo Dolce’ [9].

The identification of the geometry of the piers and spandrel elements can be easily done in the case of walls perforated by regularly distributed openings, but it becomes more ambiguous when either the openings are irregularly arranged or when infilled windows exist. In this case, they could be idealized—on the safe side—as openings, hence neglecting the contribution of added masonry [13]. Other elements that adversely influence the definition of the EF are, among others, chimneys, niches and other ancillary components, which reduce the resistant section of the walls.

Independently from the discretization approach, equivalent frame models can be distinguished in:

- (a) 1-D macro-elements models, in which piers and spandrels are simulated with columns and beams, respectively. Rigid nodes are modelled as rigid offsets starting from the extremities of both piers and spandrels. BIBLIO DEI DUE METODI?
- (b) 2-D macro-elements models, in which piers and spandrels are represented by 2D elements with the same dimensions as the correspondent portion of the wall.

1.2 EF Modelling State of Art

Equivalent frame modelling has received, in recent years, increasing attention from professional practice due to the reduced computational effort it requires, if compared to more classical FE approaches.

In the last decade, many EFM implementations have been developed.

Among others, the 3D-MACRO approach [14] and the application of a force-based model in the OpenSees platform [15], opened the road for other force-based models [17, 18].

The simulation of the application of Fiber Reinforced Polymer (FRP) to masonry in EF modelling has been studied in [19, 20]. Further applications and comparisons are reported in [16] where traditional lumped plasticity models and new EF proposals are tested in the MIDAS GEN environment. A New fiber EF model, called Fiber Flexural Model (FFM) have been tested in TU Delft and partially described in [21].

This paper follows what may be considered a traditional approach to EF modelling, where plasticity is lumped at plastic hinges at pier-rigid node and spandrel-rigid node interfaces.

2 The Case Study

Palazzo Carraro is a medium sized unreinforced masonry (URM) building, dating back to the XVII century, located in Noale (Venice). The geological and seismic characteristics of the site, as defined by the Italian technical regulation 17/01/2018 [22], are shown in Fig. 3.

Likewise the traditional historical building on the plains of the Veneto, the bearing structure is built with clay-brick-and-lime-mortar masonry while floors and roof are made of timber. The 30 cm-thick pavement finishing slabs are made of traditional *cocciopesto*, a technique where tiny brick fragments are mixed with lime mortar; the resulting paving slab is then polished with the help of flax seed oil. In roofs, thin clay

Coordinates	Latitude	45.5500	
	Longitude	12.0720	
Seismic zone	4		
Soil type	C		
Topography	T1		
a_g	SLD	SLV	SLC
	0.042	0.112	0.148
F_0	2.56	2.52	2.51
T^*c	0.259	0.330	0.351

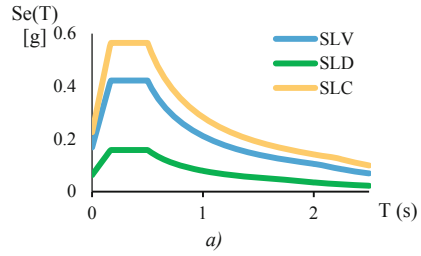


Fig. 3. Fundamental parameters, soil data and acceleration spectra at various limit states for the site, tabular form and (a) graph for elastic conditions



Fig. 4. Plans and main elevations of Palazzo Carraro [23].

bricks (*pianelle*) are laid underneath the covering tiles to provide a waterproofing layer. Therefore, the stiffness of the horizontal structure is not relevant if compared to that of masonry. Simple joist floors are parallel to the main façade (X direction) while roof rafters are perpendicular to it (Y direction). The internal plan layout is shared by the two residential storeys (ground floor and first) but in the loft, which was used as a barn in the past, transversal walls are reduced to simple masonry pillars serving as the king post of a false-truss timber structure. Windows exist only on the longest facades and their layout, which is quite regular, follows the rhythm of internal walls. However, the overall structure may be considered regular, in both plan and elevation (Fig. 4), thus fulfilling the main requirements to be modelled according to the EF approach.

2.1 Creation of the Equivalent Frame Model

An EF model of Palazzo Carraro has been created using Dolce’s method. As for the horizontal beams: the spandrels’ lengths have been defined considering the vertical alignment of the openings; the remaining elements have been defined as the horizontal rigid part of node panels. Table 1 lists the values, used for the definition of the EF of the south elevation—the most regular one.

Table 1. Geometrical parameters for ‘metodo Dolce’ application.

	\hat{H} [m]	h' [m]	D [m]	H_{eff} [m]		\hat{H} [m]	h' [m]	D [m]	H_{eff} [m]
1	3,3	2,14	0,17	2,170717	13	3,3	1,75	2,2	2,399524
2	3,3	2,07	2,07	2,48	14	3,3	1,75	1,47	2,184
3	3,3	2,23	1,22	2,425127	15	3,3	1,75	2,18	2,393619
4	3,3	2,23	1,19	2,420329	16	3,3	2,49	1,22	2,622289
5	3,3	2,07	2,12	2,489903	17	2,2	1,3	0,17	1,339231
6	3,3	2,06	1,47	2,354951	18	2,2	1,175	2,15	1,800177
7	3,3	2,49	2,18	2,726386	19	2,2	1,18	0,99	1,465254
8	3,3	3,27	1,07	3,273272	20	2,2	1,19	0,94	1,455938
9	3,3	2,02	0,17	2,055908	21	2,2	1,175	2,2	1,814716
10	3,3	1,75	2,15	2,384762	22	2,2	1,175	1,47	1,602447
11	3,3	1,75	1,05	2,06	23	2,2	1,175	2,18	1,808901
12	3,3	1,75	1,14	2,086571	24	2,2	1,8	1,08	1,88

The process worked best for six of the seven walls, where the “frame-like” structure could be easily recognized (Fig. 5a). On the other hand, the small and narrow windows present at the lower part of the north elevation create anomalies in the geometrical properties of piers and spandrels. This scheme appears in contrast to the frame idealization and leads to the extreme situation of a ‘non-frame-like’ wall (Fig. 5b).



Fig. 5. (a) EFM of main elevation (frame-like); (b) EFM of rear elevation (non-frame-like).

2.2 The Equivalent Frame Model in FEM Environment

MIDAS GEN accepts directly the 3D model of the discretized structure, as created by a CAD tool, except for floor planes, which must be redefined through finite elements in the software's pre-processor. As the frame contains information only about the axis of each structural element, any other property (section, angle, materials, etc.) must be defined in the FEM software. Two different materials [24] have been used:

- Masonry material (Table 2), assigned to piers and spandrels, whose mechanical properties conform to the Italian Regulation 14/01/2008 [22];
- Rigid material (Table 2), assigned to the rigid nodes, whose elasticity modulus (E) is 10^6 times higher than masonry one, while the mass density is a half. In fact, horizontal and vertical rigid offsets are partially overlapping, thus implying a greater structural weight on foundations than the real one if exact mass density is considered; in other models, rigid nodes mass density is set to zero [24].

Table 2. Material properties used in the model.

		Piers and spandrels	Rigid nodes
E	[MPa]	1500	$1,0 \times 10^9$
G	[MPa]	500	500
f_m	[MPa]	3,2	3,2
τ_0	[MPa]	0,076	0,076
f_t	[MPa]	0,114	0,114
w	[kN/m ³]	18	9

As for the in-plane behaviour, plastic hinges with multilinear behaviour are defined according to FEMA codes [25] at both ends of each pier and each spandrel (Fig. 6). Out-of-plane rotations are restricted only by spandrel elements of the internal walls reaching piers in the facades: no flange-effect is considered at walls' intersections (tees and corners).

In EF models, horizontal structures can be modelled by equivalent plates, 2D elements which permit in-plane and out-of-plane flexural deformations and which can match all the frame's node. Since the existing horizontal diaphragms are made of single planking timber floors, the main issue has been the definition of their equivalent mechanical properties. In particular, the shear modulus G_{XY} is the key factor, as it affects the coupling degree between perpendicular walls of the equivalent frame. Data obtained from experimental campaigns [26, 27] on in-plane behaviour of timber joist floor samples, with or without stiffening interventions, have been used to define the equivalent properties entered in present structural models. Furthermore, a full continuity between vertical and horizontal structures has been considered. This assumption may not always be appropriate for existing historical buildings, where floor joists may simply rest within a recess created in the walls without having any deep connection. However, this assumption can be considered an appropriate simplification, at least for very closely spaced joist floors as happens in traditional Veneto buildings.

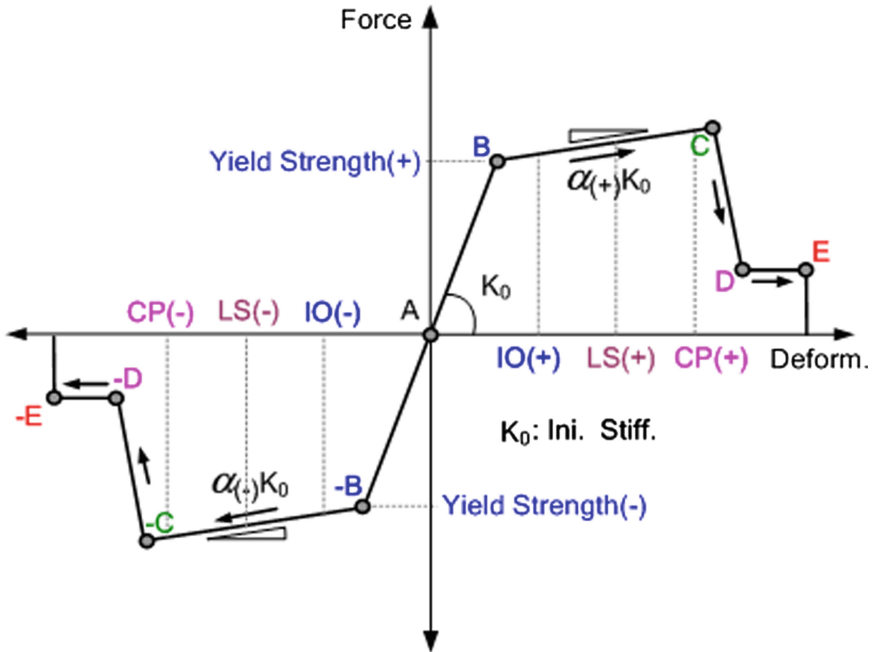


Fig. 6. FEMA hinge [25].

In order to study the effective role played by diaphragm stiffness of the in the overall behaviour of the building, three models were developed, each one differing from the others just for the G_{XY} modulus. This may be considered as an ‘incremental’ approach in respect of the philosophy of interventions according to Italian technical standards, from ‘improvement’ to ‘seismic retrofit’. The following diaphragm types were considered in the study (Table 3):

- (a) Single planking timber floor (SPT), composed of unidirectional timber joists and a simple timber board nailed to them;
- (b) Double planking timber floor (DPT), similar to type (a) but with the addition of one more planking layer, orthogonally disposed;
- (c) Triple planking timber floor (TPT), which has two planking layers crossed at 45° in addition to the basic structure described in type (a);
- (d) Reinforced concrete diaphragm (RCD).

Table 3. Values of in-plane shear modulus (G_{XY}) and tensile moduli (E) in the main directions for each simulated floor type [23].

Floor type	G_{xy} [MPa]	E_x [MPa]	E_y [MPa]
Single planking timber floor (SPT)	4,05	7400	370
Double planking timber floor (DPT)	15,8	7400	7400
Triple planking timber floor (TPT)	24,6	7400	7400
Reinforced concrete diaphragm (RCD)	295,3	300000	30000

It is worth noting that the very low values of the shear moduli make the modelling of horizontal elements more realistic, thus compensating for the overestimated connection level guaranteed by plate elements. Moreover, the great difference in the longitudinal Young’s moduli (E) between the two main directions in the SPT is used to simulate the joist floor, whose unidirectional disposition may also influence the distribution of horizontal loads on masonry structures. This difference disappears when the additional planking provides some transversal stiffness.

2.3 Considerations on the EF Modelling Method

With the aim of evaluating the efficiency and the limits of the ‘metodo Dolce’ here used to idealize the EF model, the same case study has been developed also using TREMURI. In contrast to MIDAS GEN, TREMURI automatically produces the frame of the 3D model just starting from the geometrical characteristics. Comparing the ‘meshes’ obtained from the two software codes for the south and north façades, the elevation’s regularity appears as the main factor on which the similarity between the two methods primarily depends. In case of a wall characterized by an ordered disposition of the openings, the automatic ‘mesh’ produced by TREMURI (Fig. 7a) is fully comparable with the one created in MIDAS GEN (Fig. 7b), except for the dimensions

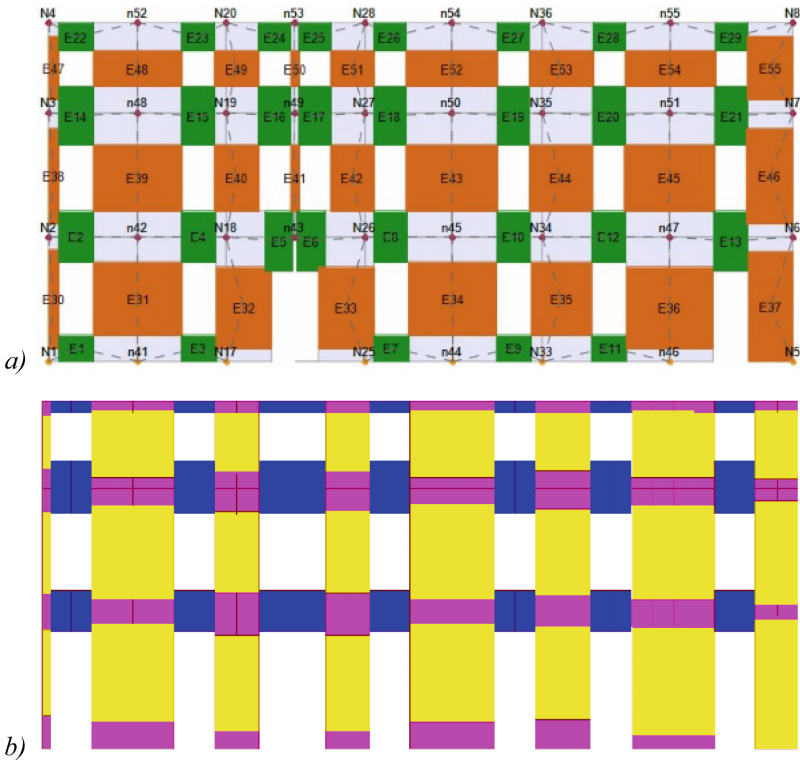


Fig. 7. EF models of south elevation: (a) TREMURI; (b) MIDAS GEN.

of the windows, which are a little bigger in MIDAS GEN, since they corresponds to the H_{eff} value and not to the real size. In case of a wall characterized by geometrical irregularity, a less comparable situation becomes evident. Taking into account the left portion of the north elevation, significant differences between the two models—both in terms of size and disposition—are found (Fig. 8).

One of the more complex aspects of URM buildings’ modelling with EF approach is the definition of horizontal diaphragms when their stiffness is not enough to ensure the box-behaviour of the structure. The possibility to control mechanical parameters was investigated using TREMURI. In modelling the diaphragms, the user can choose the correct scheme of the real structural system from a list, but the mechanical parameters for deformable floors suggested by the software differ greatly from the ones manually calculated. G_{XY} results equal 8333 MPa which is more than 20000% extra if compared with the corresponding shear modulus of the SPT case. The possibility to set the value of G_{XY} manually is also given to the user, but the difference is too high to infer that the results are comparable.

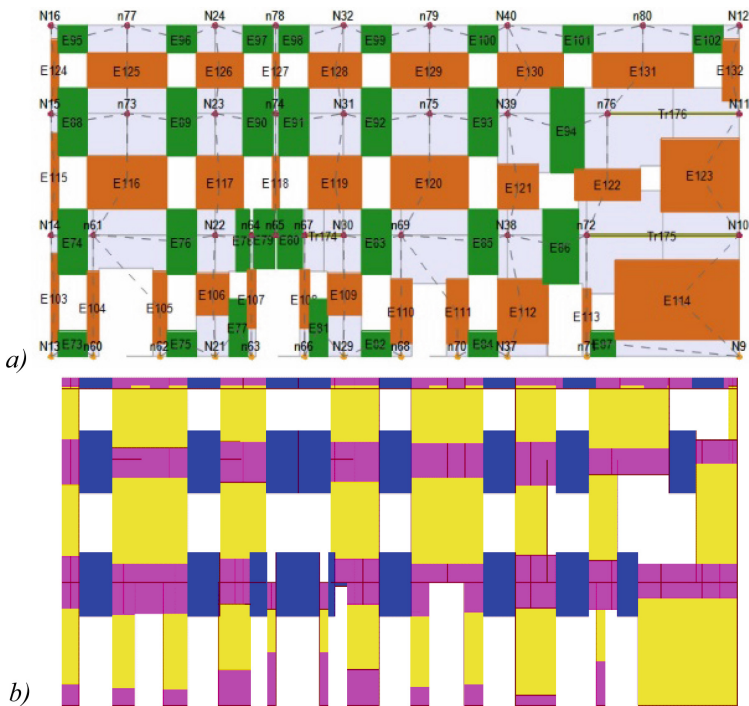


Fig. 8. EF models of the north elevation (a) TREMURI; (b) MIDAS GEN.

3 Results of Overall Analyses

Each model has been tested with linear modal and nonlinear static analyses with uniform mass load distribution, as suggested in [28] for URM buildings with flexible diaphragms. These analyses are able to describe the overall inelastic response of the structure under horizontal seismic loads. They also provide essential information about idealized behaviour in terms of stiffness, overall strength and ultimate displacement capacity. In this section the main results—in terms of frequencies, participating mass, modal shapes, capacity curves and distribution of damage—are shown and compared among the four numerical EF models.

3.1 The Modal Analysis

Table 4 lists the results of the modal analysis, in function of floors typology.

The modal analysis returns the frequencies (f) and the participant masses on the two main directions in which the structure is tested (M_x and M_y). Concerning the frequencies, it is worth noting that, as diaphragm stiffness increases, the model’s overall stiffness increases too. The first mode—the fundamental one—shows an increment from the 3.14 Hz in the SPT model, to the 4.77 Hz in the RCD one. The same happens with the participating mass: from a value of 19.4% it goes up to 81.0%—for the X direction—and from 21.6% up to 69.9%—for the Y direction—. One may observe—for both the modal parameters—that the stiffness of the double planking timber floor is already enough to make the building’s parts work together, at least in X direction, where the participating mass increases from 19.4% to 69.0%.

Table 4. Modal frequencies (f) and participating mass (M_x and M_y) for each model according to MIDAS GEN.

Mode	SPT			DPT			TPT			RCD		
	f [Hz]	M_x [%]	M_y [%]	f [Hz]	M_x [%]	M_y [%]	f [Hz]	M_x [%]	M_y [%]	f [Hz]	M_x [%]	M_y [%]
1	3,14	19,4	0,02	4,39	69,0	0,12	4,48	74,8	0,08	4,77	81,0	0,00
2	4,44	0,4	21,6	5,40	0,00	0,27	5,40	0,00	0,02	5,41	0,00	0,00
3	4,75	67,7	0,09	5,43	0,44	21,3	5,45	0,44	0,68	5,47	6,17	0,00
4	5,37	0,00	0,00	5,46	6,12	20,5	5,55	4,11	45,1	6,47	0,04	69,9
5	5,41	0,28	0,00	6,29	13,0	1,46	6,82	8,06	0,44	7,55	0,04	10,4
6	6,17	0,32	1,86	6,98	0,11	3,95	7,04	0,83	5,36	9,64	1,32	0,00
7	6,32	0,02	2,85	7,71	0,18	13,9	7,86	0,10	11,5	11,64	3,23	0,42
8	6,40	0,60	0,05	8,08	0,60	0,19	8,16	0,69	0,65	12,29	1,07	0,00
9	6,81	0,16	5,06	8,31	0,11	15,3	8,50	0,04	14,3	12,71	0,33	6,60
10	6,86	0,02	34,0	8,52	2,84	4,66	8,70	2,57	4,63	13,22	0,17	2,15

In Y direction, by comparison, the participant mass is much more scattered and only the r.c. slab is able to provide enough coupling to make almost 70% of the mass converge in one mode (the 4th, last column in Table 4). This can be considered a result

of the strong directionality of the joist floor which is here simulated by the different tensile moduli in the two directions. Further considerations are reported in [23].

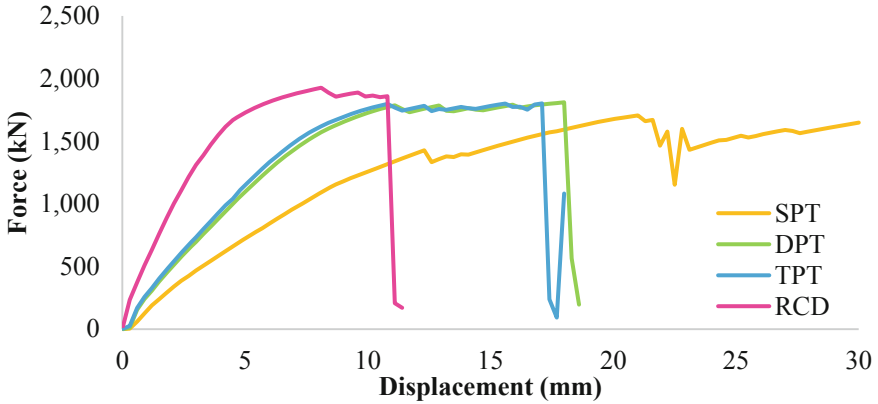


Fig. 9. Capacity curves along Y direction according to MIDAS GEN.

3.2 The Pushover Analysis

Figure 9 shows the capacity curves of the pushover analysis in the Y direction. The SPT curve (in yellow) is the longest and the least steep, representing a more flexible structure. Increasing the stiffness of horizontal structures, the slope gradually becomes steeper while the length of the curves decreases. Both these results signal a progressive increment in the stiffness of the entire building, and therefore of the gradual activation of the box-behaviour, that MIDAS GEN is able to capture.

4 Discussion of Results

In order to evaluate the efficiency of the Midas GEN EF models, they have also been implemented in the continuous approach through the DIANA FEA and in a different EF code, TREMURI. Material properties and mechanical features of the horizontal structures are the same, except for the particular differences of each code. For a complete comparison between the approaches and the results in relation to the variation of floor stiffness, see [23].

As for the modal analysis, the results (Table 5)—both in terms of frequency and participating mass—become gradually more similar by increasing the stiffness of the diaphragms, reaching a value of participating mass equal to $M_X = 81.0\%$ and $M_Y = 69.9\%$ for the Midas GEN EF model, and $M_X = 80.5\%$ and $M_Y = 67.9\%$ for the DIANA FE model, in the RCD one.

The capacity curves obtained from the three pieces of software in the SPT case (Fig. 10a) show that the models MIDAS GEN and DIANA FEA express very similar behaviour in terms of overall stiffness—visible in the curve's slope—, shear values—

Table 5. Modal frequencies (f) and participating mass (M_x and M_y) for each model according to DIANA FEA.

Mode	SPT			DPT			TPT			RCD		
	f [Hz]	M_x [%]	M_y [%]	f [Hz]	M_x [%]	M_y [%]	f [Hz]	M_x [%]	M_y [%]	f [Hz]	M_x [%]	M_y [%]
1	2,80	29,22	0,02	4,48	55,93	0,49	4,80	68,0	0,35	5,52	80,5	0,02
2	3,52	0,09	9,90	4,69	1,31	14,54	4,98	0,96	19,6	7,39	0,02	67,9
3	4,14	22,53	0,03	6,31	18,0	0,00	6,84	11,0	0,00	8,47	0,24	0,12
4	4,95	0,00	0,01	7,07	0,00	15,73	7,32	0,01	23,3	12,20	2,67	0,01
5	5,71	0,06	13,84	7,21	0,03	1,15	7,43	0,02	3,94	12,55	0,00	6,59
6	5,96	0,08	3,01	7,70	0,00	17,81	8,22	0,00	15,7	13,01	0,05	2,42
7	7,06	1,87	1,07	8,69	0,34	18,19	9,19	0,24	14,7	13,82	0,00	0,30
8	7,50	0,10	18,22	9,65	0,32	0,31	10,01	0,97	0,21	13,83	0,01	1,47
9	7,83	27,44	0,00	9,90	2,67	0,92	10,26	2,39	0,66	14,56	0,15	0,00
10	7,97	0,34	0,01	10,29	5,55	0,06	10,59	4,37	0,01	16,40	0,13	0,00

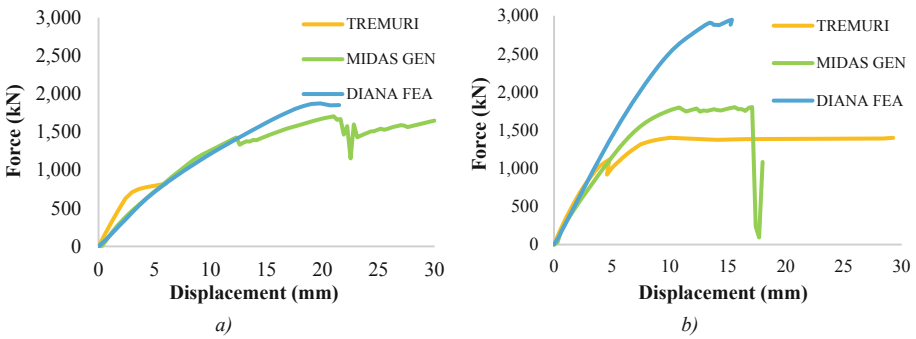


Fig. 10. Capacity curves along Y direction (a) in SPT conditions; (b) in TPT conditions.

visible in the Y-axis—and ultimate displacement—visible in the X-axis—. The TREMURI curve is noticeably different from the others, thus revealing a more rigid behaviour.

A different scenario appears when a higher diaphragm stiffness is considered. Figure 10b shows the capacity curves obtained by each software, in the case of TPT floors. The evolution of the three curves—in terms of steepness—becomes almost comparable, showing more similarities between TREMURI and MIDAS GEN models.

The damage state at failure gives another insight into the behaviour of the three approaches with respect to the diaphragms’ stiffness. As with the discussion on capacity curves, in the SPT condition it seems difficult to find a correlation between the three models [23] and the building’s collapse is reached with the crack of the most vulnerable elements. With the improvement of slabs, the interaction between all the macro-elements becomes evident, and the forces are redistributed though all the vertical structures of the building. Figure 11—which presents the last step of the pushover analysis in X direction—shows how, once diaphragm stiffness increases, failure in each model occurs with the achievement of the limit states of the building’s shearing

capacity, as should happen in a good quality masonry building under horizontal actions. The diagram of the tensile strain in the principal direction (E1, Fig. 11c) obtained with the FE model shows a realistic collapse configuration, since tensile stresses are located in areas where, in the real building, cracks are already present.

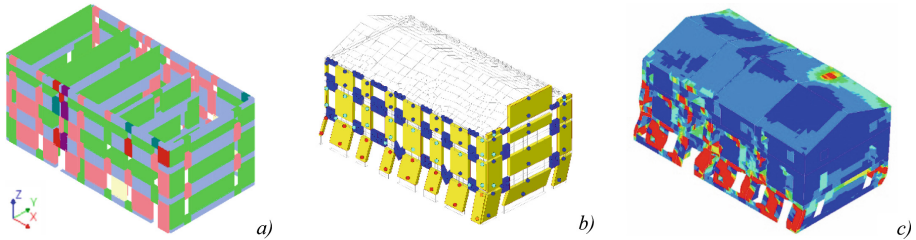


Fig. 11. Damage state at failure: TREMURI (a), MIDAS GEN (b), DIANA FEA (c) [23].

5 Conclusions

The paper investigates the possibility of modelling an URM building according to the equivalent frame approach in a FEM software, MIDAS GEN.

This allows the user to have complete control over the resulting ‘mesh’ by the means of the geometric procedure (the so-called ‘*metodo Dolce*’) used to convert the masonry structure into the equivalent frame, what may be an advantage in cases of strong geometric irregularities. The authors chose a slightly different modelling approach for i) rigid nodes, whose structural weight is considered, although halved in respect to masonry; ii) attachment between perpendicular walls, where no flange-effect is considered. Notwithstanding these characteristics, the model shows a good comparability with the more widespread approaches of TREMURI (EFM) and DIANA (FEM) software, where palazzo Carraro has also been modelled. It appears that overall analyses, both eigenvalue and non-linear static, in MIDAS GEN are closer to the ones obtained by DIANA rather than TREMURI, even though some discrepancies exist in mass distribution among the modes. As for the pushover analyses MIDAS follows the tendencies highlighted by the two other tools.

From a structural point of view the paper deals with the definition of equivalent mechanical properties of non-rigid floor slabs. These are obtained from a study of literature on experimental tests on joist floors samples, with and without reinforcement, from which equivalent elastic moduli are obtained. The key factor is, as expected, the in-plane shear modulus G_{xy} but also the unidirectionality of the existing floor system (simple joists) played an important role in the building’s global behaviour. To simulate this particular effect longitudinal elastic moduli in the two directions (E_x , E_y) are considered to have different values in unreinforced conditions. They become equal when interventions are used. Moreover, the higher the G_{xy} modulus, the more pronounced and the stiffer the building’s global behaviour become. Despite this, X direction (parallel to the main facades) has modes with higher participating mass than Y component, linked to the fact that global behaviour is more significant for the former

than the latter. Pushover curves reveals that one or two additional planking above the existing one have comparable effects, which may be considered as intermediate between the unreinforced conditions and the additional integral r.c. slab.

As far as damage prediction is concerned, it is possible to notice a strong dependence on the distribution of seismic forces (whether considered uniform or ‘modal’) but there is some agreement on the fact that collapse is reached by a ‘soft storey’ mechanism on the ground floor. It is worth nothing that only TREMURI stresses the vulnerability of the upper corners of the building.

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