Seismic evaluation of URM buildings with flexible diaphragms. Proposal of a simplified "ENT" method

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SUMMARY:
The structural behaviour of an existing masonry building, when subjected to seismic action, is affected by the in-plane mechanical properties of the floors and roofs. The goal of the present paper is to investigate the influence of the diaphragms properties (and therefore the influence of the different refurbishment techniques) on the global seismic behaviour of simple e regular masonry buildings. For this purpose a simplified elastic no-tension (ENT) method for modelling masonry structures have been proposed and adopted in order to perform push-over non linear analysis in function of different parameters of the building. According to the ENT method, at every step all the elements which are outside a Rankine failure surface are eliminated, and the analysis is repeated with an updated geometry of the model: a "globally nonlinear" behaviour is therefore determined through a series of linear analyses. The results of the push-over analyses show a lesser influence on the on the maximum level of the load and on the maximum displacement of the floor stiffness in case of URM buildings endowed with regularity and symmetry of the geometry.

Keywords: unreinforced masonry buildings, in plane-behaviour of timber diaphragms, elastic no-tension model

1. INTRODUCTION

The main purpose of this paper is to deepen the understanding of the importance of modelling the real in-plane floor stiffness when evaluating the seismic response of URM buildings. Another issue is to determine whether the wood diaphragms (both as built and refurbished ones) are to be treated as linear materials or not. Several studies have shown that timber floors, when subjected to significant lateral loads, exhibit a highly nonlinear behaviour. Since a yielding point is not always clearly identifiable [Piazza et al. (2011)], one cannot easily fit the experimental data with a bilinear curve nor can define, *a priori* a target displacement in which determining an equivalent secant stiffness. As a matter of fact, the diaphragm requirements in terms of displacement are related to the masonry skeleton the floor is connected to. In order to sort all these issues out, a simplified elastic no-tension (ENT) method for modelling masonry structures have been proposed.

2. MODELLING OF MASONRY

Masonry is known for its low tensile strength and therefore a numerical model based on plane, linear elastic finite elements (the simplest choice) could not be able to reproduce the real behaviour of a historical building. On the other hand, employing refined constitutive laws could be very time consuming and not easily manageable in case of large structures. Elastic no tension models (ENT) represent a first step towards finer modelling approaches and could be considered a reasonable compromise between accuracy and feasibility. Unfortunately ENT materials are highly sensitive to boundary conditions and prone to lack of solution and excessive displacements. Hence a simplified method has been formulated in order to take into account a very limited tensile strength and avoid the typical problems related to ENT models. So as to achieve this, a "global" Rankine failure criterion (with no limits in compression, Fig. 1) has been adopted, maintaining though an infinite linear elastic behaviour throughout all the various steps the analysis is comprised of. To make it clearer, let's consider a displacement controlled analysis on a simple masonry pier, modelled with planar linear elastic finite elements, as in Fig. 2a. The pushover analysis has been divided into five steps (from *A* to

E). After the first step a check on the principal stresses has to be made: if one of the principal stresses (σ_I, σ_{II}) of a generic element exceeds the masonry tensile strength, then the element is eliminated and the external force needed to maintain the structure at a displacement equal to ΔA decreases (Fig. 2b). Thanks to the linear elastic behaviour of the material, it is possible to stop the analysis right after the first step, do the stress check, unload the structure, eliminate the elements that are outside the failure surface and then reload up to the ΔA displacement being confident to reach the A' point. Repeating this procedure for every step and connecting the points A',B'…E' (Fig. 2d,e), one obtains the capacity curve of the structures. A "globally nonlinear" behaviour has thus been depicted through a series of linear analyses. The level of accuracy is related to the number of steps the analysis has been divided into. The greater the number of steps, the lower the probability that some elements, at the time of the stress check, are far beyond the failure surface keeping the adjacent elements from being eliminated. Consequently, in case of coarse steps, the structure response tends to be stiffer. Assuming an infinite resistance in compression, is quite a strong hypothesis (not on the safe side) borrowed from the limit analysis [Heyman, 1995] so as to keep the method as much easy to handle as possible.
The method implementation has been accomplished by means of SAP2000 and the CSI's Open Application Programming Interface that guarantees the complete automation of the procedure.

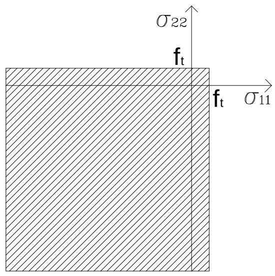

Figure 1. Failure criterion in terms of principal stresses (f_t = tensile strength of masonry)

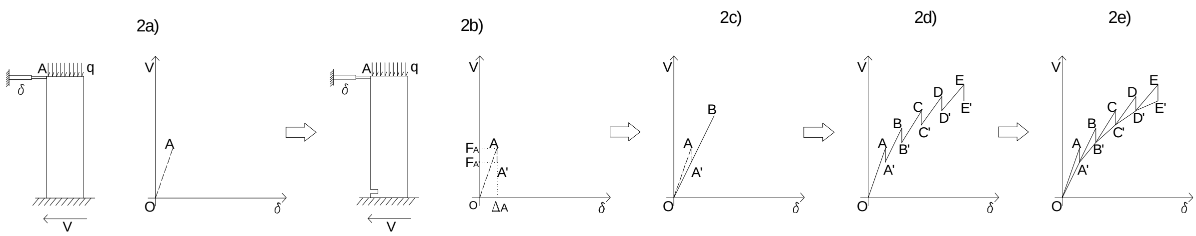

Figure 2. Simplified ENT procedure

2.1. Method validation

In order to validate the proposed method a case study has been selected from literature. The choice has fallen on the "Catania Project" [Liberatore 2000], an Italian research project involving several research groups, proposed by the National Group for Earthquake Defence. In particular the attention has been focused on the internal wall of the building sited in *Via Martoglio* (Catania, Italy, Fig. 3), whose mechanical parameters are reported in Table 1. According to the Italian Standards [C.M.617 (2009)], a value equal to 1.5 times the shear strength, has been assumed for the tensile strength of masonry ($f_t = 1.5\tau_k = 0.24\ MPa$).

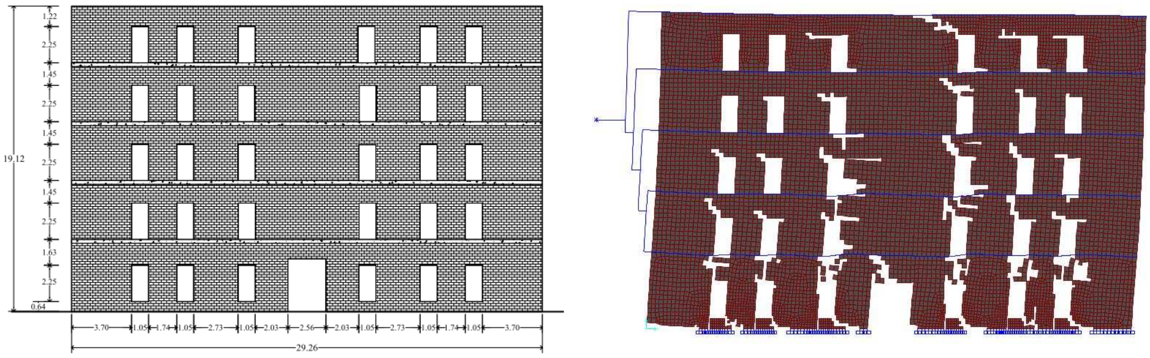

Figure 3. Via Martoglio wall – Unloaded condition (on the left), ultimate condition (on the right)

With the aim of applying a prescribed load distribution (e.g. mass proportional, first mode proportional) in a displacement controlled analysis, an equivalent isostatic loading system has been adopted [Anthoine (2006)]. The horizontal forces are introduced into the model at the storey level, in correspondence with the concrete curbs, together with the vertical loads (so as to avoid any mass loss when an element is deleted due to excessive traction). The meshing of the wall has been performed through four-node (2x2 Gauss points), two-dimensional finite elements (with just membrane behaviour) whose maximum size (0.2x0.2 m^2) has been determined after a sensitivity analysis. It should be noted that the mesh dependence is related to the analysis step dimension.

Table 1. Mechanical properties of masonry

Weight density of masonry	γ_m	17	kN/m^3
Tensile strength of a brick	f_{bt}	1	MPa
Compressive strength of masonry	f_u	6.0	MPa
Shear strength of masonry	τ_k	0.16	MPa
Elastic modulus of masonry	E	1600	MPa
Shear modulus of masonry	G	300	MPa
Cohesion	c	0.15	MPa
Friction parameter	μ	0.5	
Elastic modulus of concrete curbs	E_c	20000	MPa

From the study of the damage evolution it can be stated that the first cracks appear on the lintel above the main door at the ground floor. Then, a progressive reduction of the coupling effect offered by the spandrels has been observed (starting from the lower storeys) and consequently the formation of rocking mechanisms at the base of the ground storey piers. The shear resistance of the wall is given in Fig. 4 ($V = 1002\ kN$). With respect to the data reported in Table 2 (there is a significant scatter in the results of the different research groups) the shear resistance obtained through the proposed method is somewhat on the safe side. It should be underlined that the ultimate load is strictly related to the f_t value. If $f_t = 2\tau_k$ is used, a shear resistance close to $1300\ kN$ is obtained. As far as displacements are concerned, the proposed method exhibits the collapse point at 1.96 cm, very close to when the research group of Pavia detected the formation of a soft-storey (Fig. 4). On the other hand, as expected, it is quite distant from the ultimate displacement shown by the POR based methods (L'Aquila research group) which do not consider any damages of the spandrels.

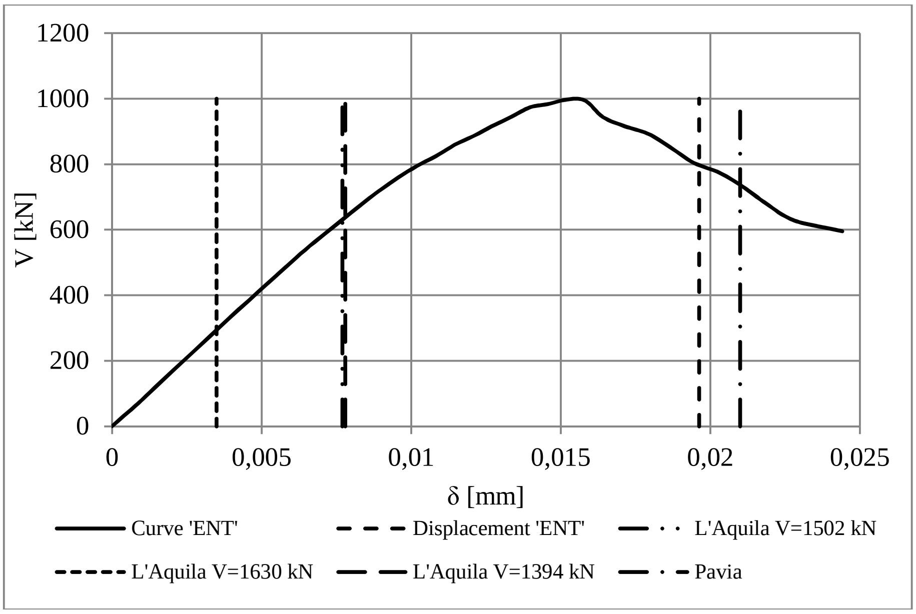

Figure 4. Capacity curve of the Via Martoglio wall

Table 2. Catania Project results

Model	**Research Units**	**V (kN)**
	Basilicata	2050
Elastic curbs E = 20000MPa	Genova	1492
	Pavia	1227
Elastic curbs E = 20000MPa (rigid offsets)	Basilicata	2226
	Basilicata	2050
Elastic curbs E = 4000MPa	Genova	1263
	Pavia	848
POR, piers’ height = interstorey height	L’Aquila	1502
POR, piers’ height = openings’ height	L’Aquila	1630
POR90, piers’ height = interstorey height	L’Aquila	1394

2.2 Case study building

Figure 5 shows the structure selected for the analyses regarding the in-plane behaviour of timber diaphragms. It is a four storeys building (15.60 m high) with a rough size of 10.60x15.60 m^2. The thickness of the walls is 0.6 m for the first two storeys and 0.5 m for the others. There is also an internal spine wall whose thickness is equal to 0.3 m. As already mentioned, the loading system is able to maintain a prescribed load pattern throughout the displacement controlled analysis, required to depict the post peak phase. In other words, at the “actuator”, the analysis is a proper displacement controlled analysis, while on the building it becomes a force-controlled one. This means that the nodal displacement of the frame representing the actuator, is an increasing monotonic function. On the other hand, some points of the building could show a reduction in displacement in order to counterbalance (due to the isostatic loading system) the decreased stiffness of part of the structure. The ratio between the forces acting at the same level has been worked out thanks to a force-controlled elastic analysis in which, all the inertial forces have been applied exactly where they are. That is to say, for example, that the forces generated by the floor mass are introduced at the nodes of the cells modelling the diaphragms. It should be noted that this distribution, representative of the undamaged condition, is kept unchanged for the entire analysis. To determine whether this aspect yields remarkable effects on the determination of the peak point, some force-controlled analyses have been performed following a procedure similar to that exposed in paragraph 2 (no effects have been registered).

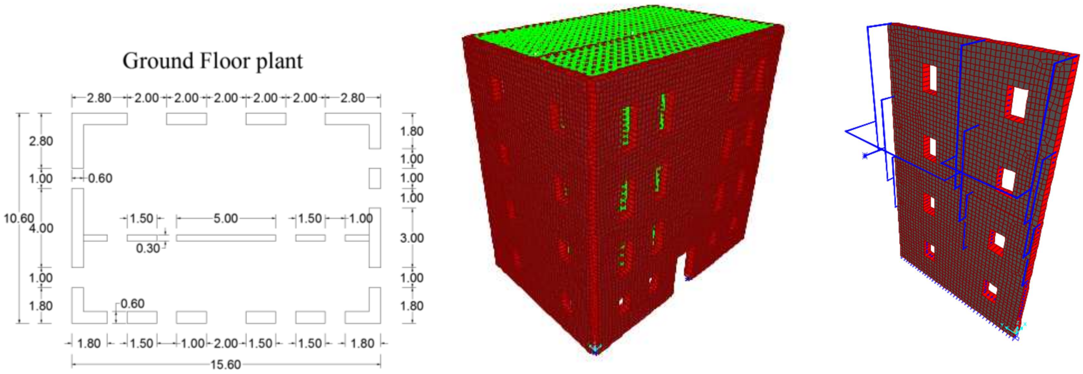

Figure 5. Case study building (on the left and in the middle) and the isostatic loading system (on the right)

It is known that the choice of the control point has a great influence on the determination of the capacity curve. In addition, owing to the features of the loading system, it is not rare to observe a decrease (from a certain time onwards) in the displacement of the monitored point. Therefore it has been chosen of monitoring the building displacement in correspondence with the frame element representing the actuator. Considering that this element is positioned at about two third of the building height, the resulting capacity curves will be on the safe side in terms of ultimate displacement.

3. MODELLING OF WOOD DIAPHRAGMS

Data pertaining to wood diaphragms are taken from [Baldessari et al. (2009)] where an extensive experimental campaign on *5x4* m^2 timber floors is presented. A fitting of the backbone force-displacement curves has been carried out following the procedure proposed by [ABK (1984)]:

$$F(\delta) = \frac{F_u \cdot \delta}{\dfrac{F_u}{k_i} + \delta} \tag{3.1}$$

where δ is the midspan displacement, $F(\delta)$ is the force at the diaphragm's end, k_i is the initial stiffness and F_u is the ultimate force. F_u is obtained multiplying the unit shear strength of the diaphragm ν_u by its width [Paquette & Bruneau (2006)]. With reference to every floor typology tested by Baldessari et al., all the parameters required for determining the backbone curves are given in Table 3.

Table 3. Parameters for ABK formula

	ν_u [kN/m]	F_u [kN]	k_i [kN/mm]
Single Straight Sheathing	52.0	208.0	1.1
Double Sheathing	67.6	270.4	11.2
Steel Plates	59.8	234.4	23.2
FRP Laminae	51.8	207.2	45.1
Concrete Slab	85.4	341.6	60.0
Plywood Layers	64.8	259.2	106.1

Both the experimental tests and the parametric analyses performed on FEM models (Fig. 5) have shown that the deformed shape of the diaphragms are extremely close to that of a uniformly loaded shear beam. Consequently an equivalent shear stiffness G_{eq} has been calculated regarding the diaphragm deformation as equal to the shear deformation of a simply supported beam under a uniform load distribution.

$$G_{eq}(\delta) = \frac{2F(\delta)\cdot L}{8\cdot B\cdot t\cdot \delta} \tag{3.2}$$

where L = floor span perpendicular to the load direction, B = floor span parallel to the load direction, t = floor (membrane) thickness, $2F(\delta)$ = lateral load applied, δ = mid span deflection. It is worth noting that the secant stiffness curve calculated in (3.2), is a function of the midspan displacement of the specimen and therefore could not be representative of floors with different geometries. To solve this problem, might be useful referring to a non-dimensional quantity such as the shear strain γ. On the other hand the shear strain is not uniform and varies along the equivalent-beam axis. Since the diaphragms have been modelled with a series of reference cells (Fig.8) consisting of an external frame of rigid rods and two internal diagonal rods whose stiffness is equal to G_{eq} multiplied by the floor thickness, a mean value of shear strain γ^* has been calculated for every δ (Fig. 6). So as to take into account the nonlinear behaviour of the floors, the following iterative procedure has been developed. The analysis begins with the shear stiffness of the floors equal to G_1 (Fig. 6). At the end of the first step, the angular deformation of each cell is calculated: if the maximum γ is equal or smaller than γ^*_1, it is possible to proceed with the stress check and the element deletion, otherwise the stiffness has to be changed and the step rerun. This process must be repeated after each step.

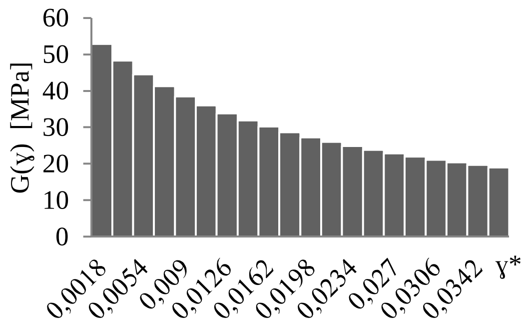

Figure 6. Equivalent shear stiffness (Double sheathing)

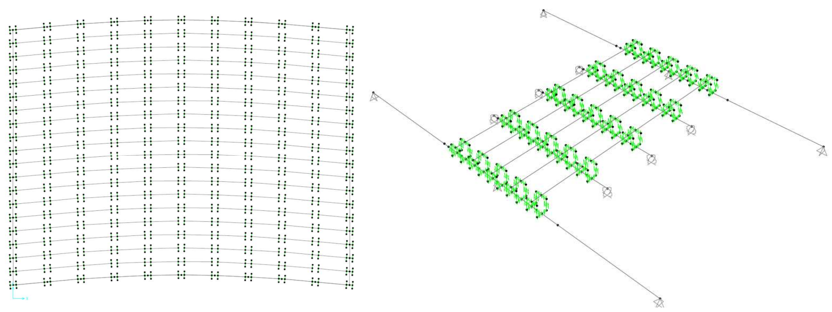

Figure 7. Numerical model employed in the parametric study of the deformed shape

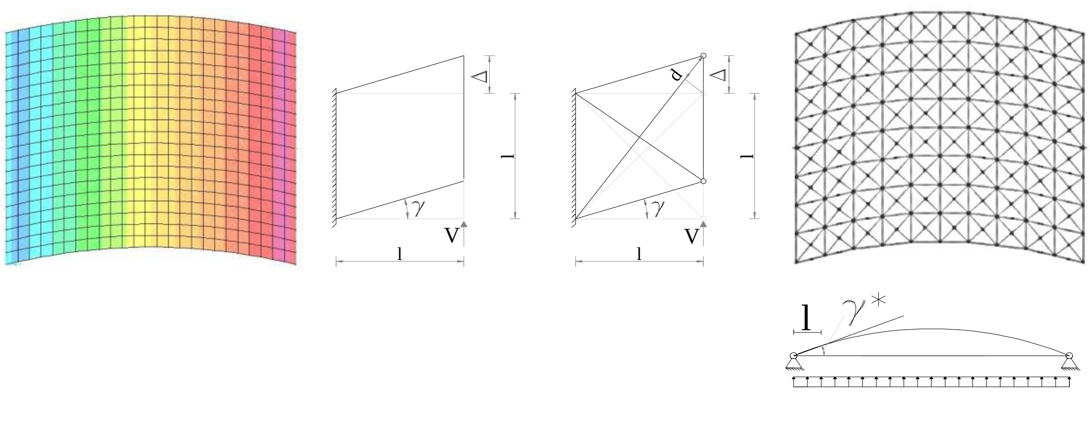

Figure 8. Diaphragm modeling

For the analyses where the diaphragm behaviour is considered linear elastic, the target point needed to determine the secant shear stiffness, has been chosen in accordance with the results presented by Paquette & Bruneau which carried out pseudo-dynamic tests on a URM building with flexible floor of size very similar to the specimens tested by Baldessari et al.

4. ANALYSIS RESULTS

Figure 9 presents the capacity curves of the case study building for the analysed diaphragms. It can be seen that the in-plane stiffness of floors plays a negligible role in determining the global response of the structure (all the curves are practically the same). A probable reason can be found in the distance between the mass centre and the centre of stiffness which is smaller than *0.5 m*. In order to increase the stress state of the diaphragms, the model has been modified by halving the thickness of the north wall (moving therefore the centre of stiffness). As a result, a very slight difference is registered, denoting an increase in the performance as the floor stiffness grows (Fig. 10). It should be noted that in masonry buildings the bulk of the structure is represented by the walls. Consequently, the north-wall's stiffness-variation generated by the halving of the thickness, is somehow counterbalanced by the reduction in horizontal force (acting on the north wall) due to the mass diminishing. Therefore it has been decided to apply an additional eccentricity of *2 m* to the mass centre, even if that is not consistent with the building geometry. From Fig. 11 it is possible to observe that the higher the floor stiffness, the greater the shear resistance and the ultimate displacement. This result seems not to be in good agreement with [Giongo et al. (2011)] where, apart from the single straight sheathing, it appears not to be any significant variations in the pushover curves between the different floor typologies. The causes might be found in the different method adopted for modelling masonry (equivalent frame method) and in the building characteristics.
With reference to the issue of assuming for diaphragms a linear behaviour rather than a nonlinear one, many analyses have been carried out: no appreciable differences have been observed. The only small difference has been registered for single square sheathing solution when the aforementioned additional eccentricity is considered (Fig. 12). So it seems that a linear elastic behaviour could be adequate to reproduce the global seismic response of a URM building with timber floors. Further analysis is however recommended.

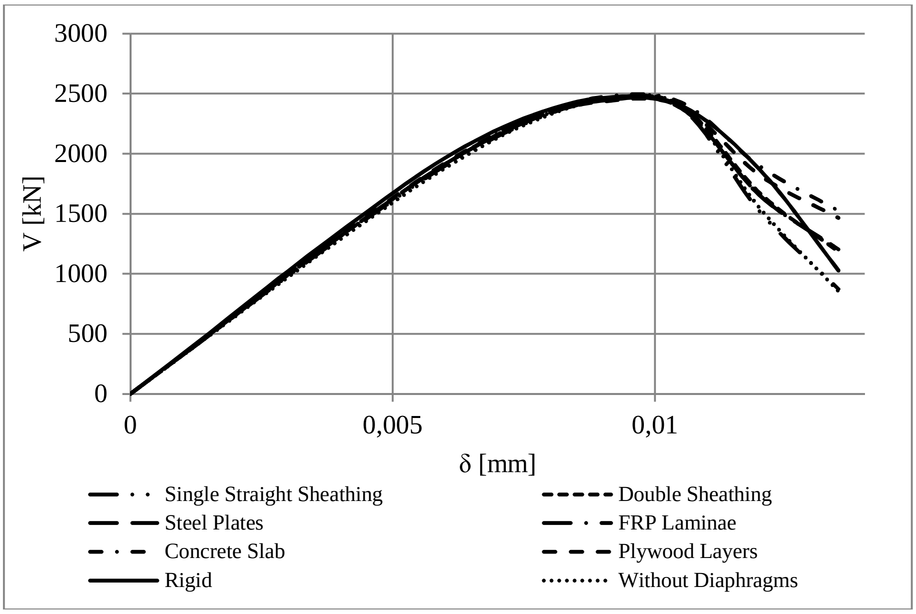

Figure 9. Capacity curves (different floor-typologies)

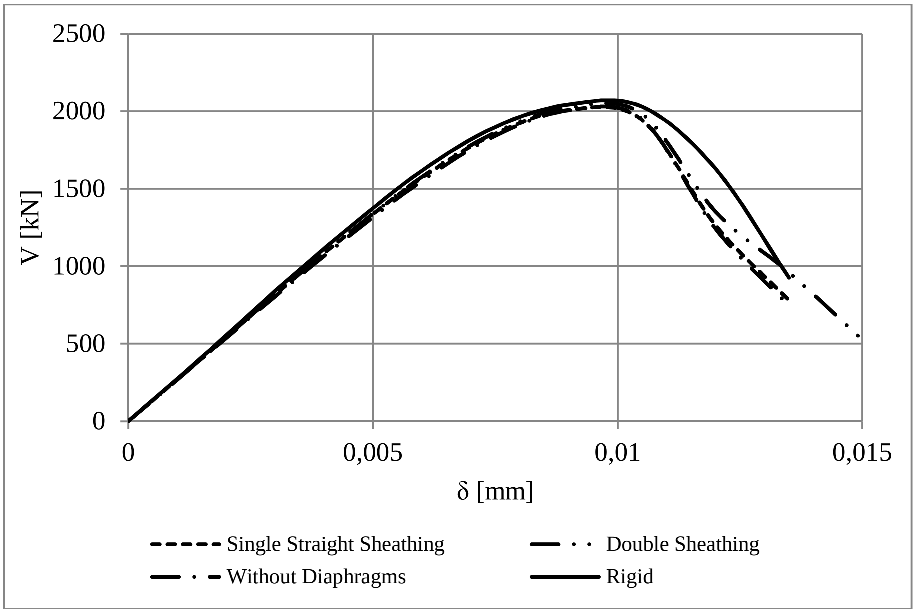

Figure 10. Capacity curves (North wall with halved thickness)

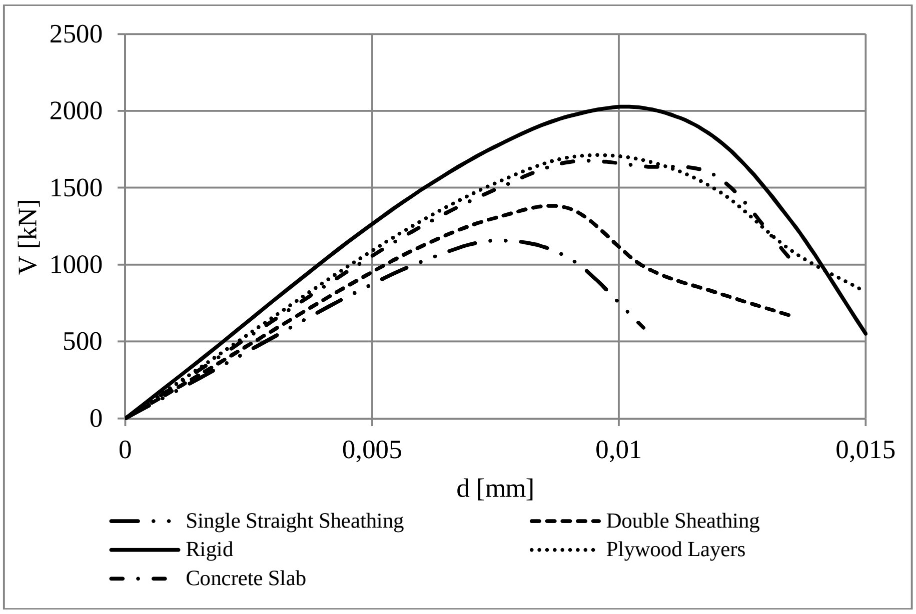

Figure 11. Capacity curves (*2 m* of additional eccentricity to the mass centre)

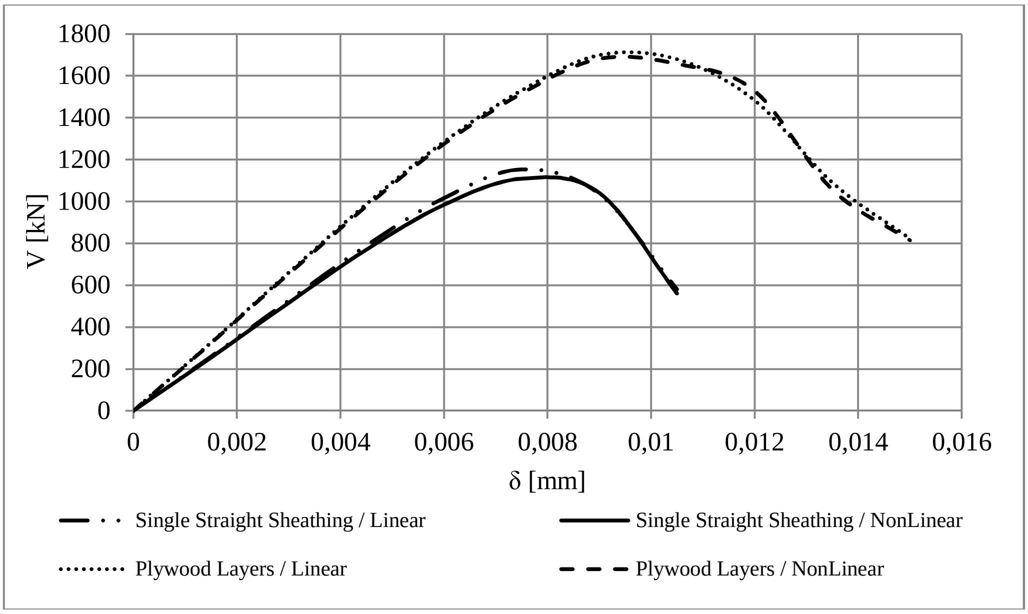

Figure 12. Floors with linear constitutive law Vs. Floors with nonlinear constitutive law

5. CONCLUSIONS

From the presented results it would appear that modelling the real in-plane stiffness of diaphragms becomes quite important only in presence of remarkable eccentricity between the mass centre and the centre of stiffness. However, it should be taken into account that in URM buildings, the seismic mass associated with floors is very small in comparison with the mass of the walls. Therefore the position of the centre of mass is related to that one of the centre of stiffness.

In addition, it seems that modelling wood diaphragms with a linear elastic in-plane behaviour is sufficient to describe the global seismic response of URM buildings.

As far as the proposed simplified ENT method is concerned, it has shown to be quite easy to handle and able to follow the damage evolution.

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