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PRECEEDING STUDIES ON ASSESSMENT PERFORMANCE OF THE UNREINFORCED MASONRY LOW-RISE BUILDING

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ملخص البحث

تعتبر مباني الطوب من المنشآت واسعة الإنتشار على مر التاريخ القديم , و حديثا تمّ إستخدامها في بعض الأحيان بديلا عن المنشآت الخرسانية و المعدنية , حيث تميزت مباني الطوب بقدرتها العالية على مقاومة العوامل الجوية و ما تتعرض له من أحمال حرارية و زلزالية على مر العصور. يحتوي البحث على ما تمت دراسته في سلوك حوائط مباني الطوب منخفضة الأدوار سواءا في بعض المنشآت التاريخية أو المعاصرة و تشمل هذه الدراسة حالات وجود أسقف في صورة قبو نفقي (فولت) و بعض طرق التحليل الإنشائي التي تمّ تبنيها في هذه الأبحاث. و يقوم هذا البحث أيضا بإستعراض ما توصلت إليه الأبحاث السابقة من نتائج عملية و نتائج نظرية بناءا على التحليل الإنشائي لهذا النوع من المنشآت.

Abstract

This paper demonstrates preceding researches related to the performance assessment of the low-rise building composed of the unreinforced masonry URM used in the contemporary and historic ages either its roof was vaulted or flat. The research focused on the analytical approaches used in these studies and both the results and conclusions conducted. Also, these studied discussed the types of the applied loads on the vaulted or non-vaulted URM structures.

Keywords: Barrel Vaults, Unreinforced Masonry, URM, Seismic Behavior, Stone Masonry, historic structures.

1. Introduction:

In the past, the use of unreinforced stone masonry structures (URM) was widespread as a good alternative in constructing many low-rise building. The historic buildings, heritages, castles and temples belong to the URM masonry. The masonry material is an environmental resource can be used without causing any pollution or consuming large energy. That leads to a sustainable and economic building material can be adopted in constructing contemporary structures. The cost of these types of stone URM is noticeably less than both the reinforced concrete or steel elements for constructing low-rise buildings. Some previous studies related to the URM would be explained in the coming sections below.

2. REDUCED-SCALE DYNAMIC TEST ON URM BUILDINGS

Based on Reduced-scale dynamic tests [1&2], pseudo-dynamic tests (Paquette and Bruneau, 2000 and 2003) [3,4], and large-scale quasi-static tests (Magenes et al. 1995) [5] have been conducted on URM structures. The first dynamic tests on a URM structure were conducted by Clough et al. (1979) [6]. Four one-story masonry houses, with both unreinforced and partially reinforced masonry wall panels, were tested on a shake table. The objectives of this experiment were to determine the maximum earthquake intensity that could be resisted by a typical URM house, and to evaluate the additional resistance that would be provided to the structure by partial reinforcement.

In this test, the masonry units, the size of the wall components, and the roof-to wall connections were full-scale to represent the behavior of a real masonry building. On the other hand, the plan areas of the building were one-ninth those of a reasonable prototype due to the capacity of the shake table. To represent the realistic gravity stresses in the masonry pier, weights were added at the roof level. The first specimen was designed with a panel in the middle of each four sides, and with a corner component located at each corner (Fig. 1). The other three specimens were designed with four perforated walls and no direct connections between adjacent wall panels (Fig. 2). All four specimens were made from standard two-core hollow concrete block or two-core hollow clay brick and type S mortar. A typical timber truss roof system was used for all the four specimens.

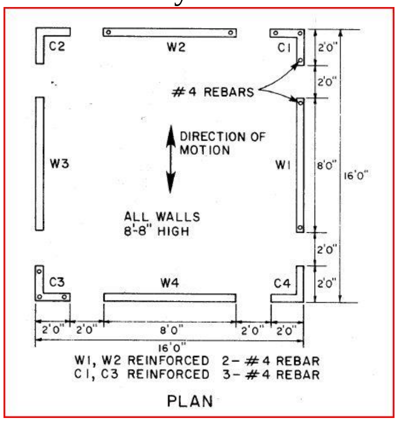

Fig. 1: Crack observed in the spandrel of Wall B (Magenes et al. 1995)

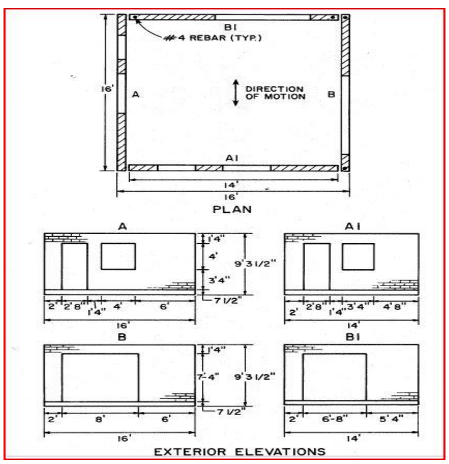

Fig. 2: Crack observed in the spandrel of Wall B (Magenes et al. 1995)+

The following phenomena were observed in these tests (Clough et al. 1979):

- Since the stiffness of the in-plane walls was much larger than that of the out of plane walls, the in-plane walls resisted the majority of the seismic forces.
- The masonry structure was so stiff that the motions of the test structures followed the shake table motions very closely, with the deformation of the structure generally being proportional to, and in phase with the base accelerations. The amplification of ground motion due to the flexibility of structure was rather small.

As a result, the peak acceleration, instead of the frequency characteristics, was a major factor to be considered when assessing the damage of a URM building.

- If one in-plane wall was stiffer than the other, the two in-plane walls might develop different lateral displacements under lateral earthquake excitation, with a resulting tendency to cause rotation of the roof. If the roof had sufficient membrane rigidity, it would rotate as a rigid unit, and consequently induced out of-plane deformations in the in-plane walls, and in-plane deformations in the out of-plane walls. However, if the stiffness of the roof diaphragm was much smaller than that of the masonry walls, the masonry walls would resist this tendency and forced the roof to develop shear distortions to accommodate the unequal displacements at the top of the in-plane walls.

Based on the prototypes of old urban masonry residential houses in the earthquake-prone areas of central Europe and Mediterranean, four 1:4 scale simplified two-story URM models were constructed and tested in a one-degree vibration shake table by Tomazevic et al. (1993) [7]. The URM structures were composed of stone and cement mortar (cement: lime: sand in the proportion of 0.5:4:12). The structural configurations of the masonry walls in all the four models were identical: the in-plane walls oriented in the direction of the shake table motion were solid loading-bearing walls, while the out-of plane walls were perforated walls with window and door openings (Fig. 3). The diaphragms were different for the four models (Fig. 4). Model A had wooden floors made of freely supported wood joists without steel ties. The diaphragms of Model C were identical to those of Model A, except that the masonry walls were tied with steel ties at the floor and roof levels. The diaphragms of Model D were similar to those of Model C, except that a brick vault replaced the wooden roof. The diaphragms in Model B consisted of RC slabs with bond-beams along the walls.

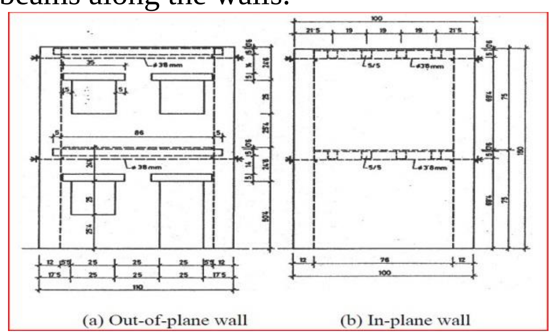

Fig. 3: Layout and dimensions of the tested models (units in cm) (Tomazevic et al. 1993)

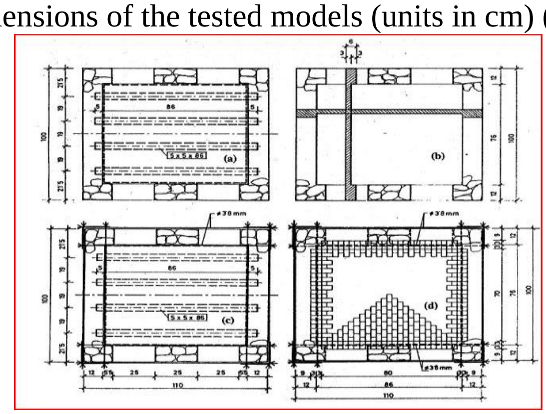

Fig. 4: Floor and Roof systems of the tested models (units in cm) (Tomazevic et al.1993)

The behavior of Model A was as follows. At the beginning of the test, rocking was observed along the cracks at the joints between the walls and the foundation slab. With increasing ground motion, more and more horizontal and diagonal cracks developed in the first floor walls. After that, the walls in the second story disintegrated, and all the upper corner walls separated. Vertical cracks and horizontal cracks were also observed in the second-story out-of-plane walls. Masonry units began to fall off. Meanwhile, the cracks in the first floor continued to propagate. The test was stopped when one of the corner walls at the second floor collapsed. The behavior of Models B, C, and D were similar. All of them collapsed because of the severe damage developed in the walls in the first story, whereas no significant damages to the second story walls were observed. At the beginning of the test, the models were observed rocking and vibrating along the crack at the joints between the walls and the foundation slab. Then horizontal cracks developed all around the models just below the floor diaphragm. With increasing ground motion, the damage accumulated in the first floor walls, while the second story walls vibrated like a monolithic box placed on the top of the first floor walls with little damage. Finally, severe diagonal cracks developed in the first-story in-plane walls. Also, vertical cracks developed at the corners of the first-story in-plane walls because of the sliding and rocking of the upper second-story box.

The lateral deformation shapes were also obtained in this experiment. (Fig. 5) shows the distribution of the displacements at three locations along the roof. The displacements of the in-plane walls and the out-of-plane walls were almost the same in the elastic range for the different diaphragms, possibly due to the large thickness of the masonry walls. However, with increasing ground motion, the differences between the lateral displacements of the in-plane walls and that of the out-of-plane wall increased. As observed in the experiment, there was out-of-plane failure in Model A, but not in Model B, C, and D. It indicates a rigid diaphragm or simply tying the masonry walls at the floor and roof levels can prevent the out-of-plane damage of masonry walls.

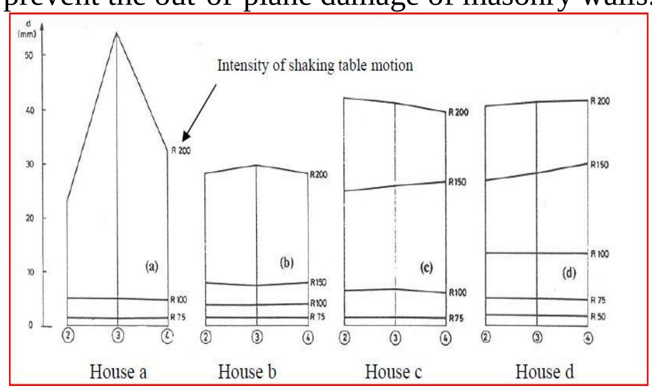

Fig. 5: Distribution of displacements along the top floor (Tomazevic 1993)
(Locations: 2, 4 the in-plane walls, 3 center of the out-of-plane wall)

The other important conclusions also obtained from this test are (Tomazevic 1993):

- The structural characteristics of the floor and roof diaphragms and the tying of structural walls represented decisive parameters to the seismic resistance of masonry walls.
- For a URM structure without ties to prevent the separation of the walls, the out of-plane walls cracked easily. As a result, the out-of–plane walls might collapse

before severe damage developed in other parts of the structure. In addition, the failure of out-of-plane walls was easy to develop in the upper story.

- If the failure of the out-of-plane walls were prevented by a strong floor system, the damage would concentrate on the first story in-plane walls. When the upper structure rocked and slid on the top of the first floor, the corner of the first floor failed early in the tests.

More recently, two reduced-scale URM buildings were constructed and tested at the University of Illinois by Costley and Abrams (1996). The box-type structures had two perforated shear/bearing in plane walls (window wall and door wall), and two solid out of- plane walls (Fig. 6 and 7). For both test structures S1 and S2, the two out-of plane walls and the window wall were continuous, forming a C-shape, while the door wall was separated by a full-height gap with the width of one mortar joint. A steel diaphragm with attached additional weights was used to represent the flexible wood diaphragm. The diaphragm was simply supported on the in-plane walls through special details so that it could transfer the shear forces as well as the vertical forces. The floor system was also tied to the out-of-plane walls by rods and nuts. Only the first building S1 is discussed here, since the second building was rebuilt from the first one and exhibited similar behavior.

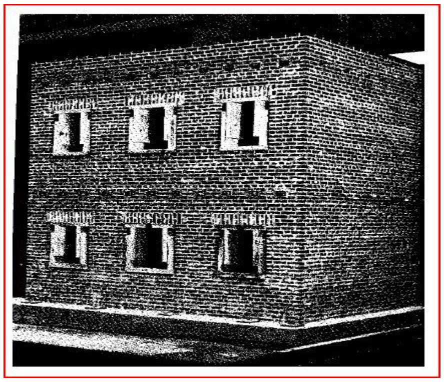

Fig. 6: Window wall and out-of-plane wall of the tested structure S1 (Costley and Abrams 1996)

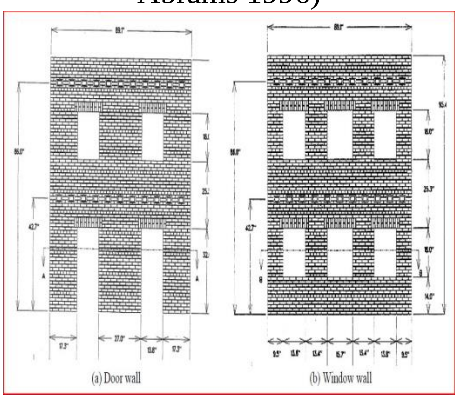

Fig. 7: Configuration of perforated in-plane walls (Costley and Abrams 1996)

The first cracks observed in this building were the debonding cracks between two out-of-plane walls and the concrete foundation. With increasing base acceleration, more and more cracks developed in both the in-plane walls and the out-of-plane walls. In the door wall, the outside piers rocked, and the central pier slid. In the window wall, some cracks were observed initiating from the corner of the window opening, and propagating as diagonal cracks into the piers. The entire top portion of this test structure appeared to be fixed in space as the first-story walls moved back and forth below with the base excitation.

As expected for a flexible diaphragm system, little coupling was observed between the parallel shear walls. Individual walls vibrated independently of each other with no torsion induced by the diaphragm. In some cases, the deflection of the door wall was two times larger than that of the window wall. The acceleration ratios for the model structure were also interesting. Prior to cracking, both the ratio between the wall acceleration and the base acceleration and the ratio between the diaphragm acceleration and the wall acceleration were appreciable, on the order of 1.2-1.7 and 1.7-2.5, respectively. After substantial cracks developed in the walls, both of the two ratios decreased to almost 1:1, which means little amplitude existed.

The test also showed that the equivalent roof level seismic force was almost the same as that at the floor level. For the structure in elastic range, the phenomenon could be explained by the fact that the masonry walls might be very stiff. After cracks developed in the structure, these results might also be expected since the upper portion (including both diaphragms) of the structure remained intact and moved as a rigid body on the top of the first floor.

Compared to the reduced-scale dynamic experiments, full-scale tests of URM structures are seldom conducted due to the cost and test capacity demands. Recently, a research program was conducted at the University of Ottawa to investigate the flexible floor- rigid wall interaction in old URM buildings (Paquette and Bruneau, 2002). A test of a single-story full-scale URM building was conducted. This building was composed of two symmetric perforated in-plane walls and two solid out-of-plane walls, which were constructed from solid bricks and Type O mortar (Figure 2.14). The two out-of-plane walls and the east in-plane wall were continuous, forming a C-shape, while the west in plane wall was separated from the out-of-plane walls by a gap. This was used to investigate the effect of out-of-plane walls on the in-plane walls. The flexible diaphragm of this building was constructed with wood joists and covered with diagonal boards with a straight board overlay. The diaphragm was also anchored to the wall with through-wall bolts in accordance with UCBC (ICBO, 1997). The building was tested in a pseudo dynamic fashion by using one actuator to apply pseudo-dynamic force at the center of the diaphragm. One interesting finding in this test is that during the initial low intensity seismic motion, different stiffness for the east and west walls were observed. However, after the cracks fully developed in the building, the hysteretic curves for these two shear walls during a higher intensity seismic motion became very similar. This suggests that the effect of continuous or discontinuous corners becomes less significant during high intensity seismic motion.

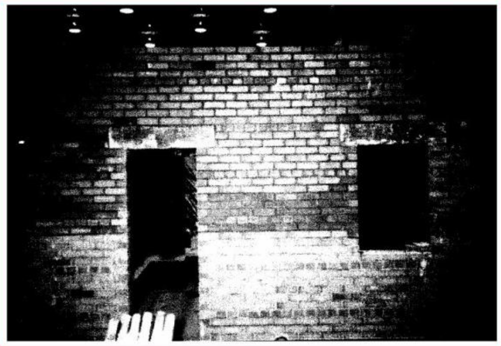

Fig. 8: Tested single-story URM building (Paquette and Bruneau 2002)

2.0. VAULTED SYSTEMS IN HISTORIC CHURCHES

The seismic behavior of churches may be investigated with both global analyses [1] and local analyses [2]. The second approach is supposed to be more capable to correctly interpret the churches response if compared to the first one [3]. Nevertheless, a global analysis is needed to collect information about the dynamic behavior of the whole building, which is justified especially in presence of a single constructive phase for the church. Therefore, global analyses can be useful to understand the role of macro-elements once their dynamic characterization has been evaluated [4].The proposed method was applied to the St. Frediano's church, a Romanesque church located in Pisa (Fig. 9a). The case study was chosen being a typical three naves basilica church characterized by the presence of different typologies of vaulted systems (Fig. 9b).

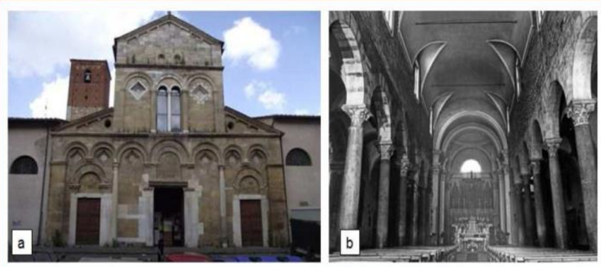

Fig. 9: Eastern main façade (a) and internal view (b) of St Frediano's church.

That constructive solution is very widespread in Italy, and since the geometric ratio between structural elements dimensions are recurring for such churches, the procedure can be easily applied to similar cases. The global dimensions are 41x15x16 m (length x width x maximum height). The church is adjacent to masonry buildings but it will be considered structurally isolated for the purposes of the work. The church is made up by ashlar stone (façade and walls), granite (columns), bricks and mortar (vaults). For linear analyses the following mechanical properties [8-9] have been considered: Young modulus 50000 MPa (granite), 1500 MPa (bricks and mortar), 2800 MPa (stone); Poisson coefficient 0.15 for all materials; specific weight 27 kN/m^3 (granite), 18 kN/m^3 (bricks and mortar), 22 kN/m^3 (stone).

2.1. MODAL AND SENSITIVITY ANALYSIS

A first comparison between actual (church with full modeled vaults) and simplified model was made in terms of natural frequencies and activated masses [10-11]. Two VETs (Vaults Equivalent Trusses) simplified models were considered: one with additional masses representing the removed masses of the vaults, and a second one were the additional masses are not computed. In the first case the inertia is recreated by applying concentrated masses at the supports of the vaults; for the central nave barrel vault at the extremities of VETs. In both models material density was calibrated to keep the same rate of self-weight for structural elements, in particular the same rate of vertical reactions forces at columns with respect to the total value, namely about 20%. 230 modes were necessary to reach about 85% of the total mass for the two main horizontal directions of the building.

Modal analyses showed a good correspondence for the simplified model without additional masses, having a difference lower than 8% of the first natural frequency along the transverse direction and the corresponding modal mass. In the model with additional masses the natural frequency variation is 18% and the variation of the corresponding mass 13% (Table 1). Moreover, also in the longitudinal direction the correspondence is better in case of simplified model without additional masses (Tables 2&3). The mode shapes between actual and simplified model are quite similar, so the VET model without masses does not filter the individuation of macro-elements, especially the relative transverse deformation of the longitudinal central walls which affects the mode in the longitudinal direction (Fig. 10).

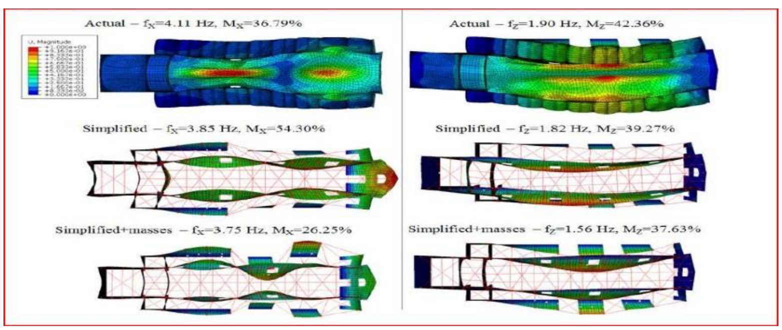

Fig. 10: Comparisons between the main mode shapes in X (longitudinal) and Z (transverse direction).

A sensitivity analysis on models with horizontal slabs was also performed to investigate the dynamic behavior in case of vaults as flat panels. The horizontal thrust is calculated for each vault and applied as distributed force; the density was calibrated not to alter the initial mass distribution. By assuming the same thickness of the vaults (6 cm), the buildings stiffness is slightly lower than the VET for transverse direction, but stiffer in the longitudinal one. Consequently, for transverse direction the VET model is in favor of safety. In addition, the mentioned effect of relative displacement in the central nave for the longitudinal mode is not visible in case of horizontal slabs (Fig. 11), thus they are not able to properly describe the overall behavior. Generally, the VET model is more deformable then the actual one, as expected, since this is not able to recreate totally arch stiffening effects. The thrusts are indeed idealized in concentrate points and not along the arches profiles.

Table 1: VET for the St. Frediano's church.

		Vault	K_{eq} (daN/cm)	d_{eq} (mm)	$K_{eq}/2$	d_{eq} ($K_{eq}/2$)
Lateral naves	DV1	diag	15552	22.2	7776	15.7
		long	12555	16.8	6278	11.9
		transv	12555	16.8	6278	11.9
	DV2	diag	10060	18.6	5030	13.1
		long	9882	14.9	4941	10.5
		transv	13663	17.5	6832	12.4
Apse	-	diag	56445	59.7	28223	42.2
		long	138719	74.8	69360	52.9
		transv	54208	51.3	27104	36.3
Central nave	800X370	Barrel_lun_diag	62206	57.7	31103	40.8
		Barrel_lun_long	172806	62.3	86403	44.0
		Barrel_lun_transv	15240	27.2	7620	19.2
		Barrel_diag	81209	65.9	40604	46.6
		Barrel_long	248612	74.7	124306	52.8
		Barrel_transv	9626	21.6	4813	15.3
	800X430	Barrel_lun_diag	54977	55.0	27488	38.9
		Barrel_lun_long	152189	63.0	76095	44.5
		Barrel_lun_transv	17441	29.1	8720	20.6
		Barrel_diag	74086	63.9	37043	45.2
		Barrel_long	223165	76.3	111583	53.9
		Barrel_transv	12611	24.7	6306	17.5
Complete longitudinal barrel vault			21413	63.2	10707	44.7

Table 2: Comparisons between natural frequencies and modal masses in Z (transverse) direction.

Model	f_z (Hz)	M_z (%tot)	% Actual f_z	% Actual M_z
Actual	1.90	42.36	0.00	0.00
Simplified	1.82	38.86	-4.33	-8.27
Simplified + add. masses	1.56	37.63	-17.85	-11.16
Horizontal slabs 6 cm	1.76	42.90	-7.50	1.26
Horizontal slabs 60 cm	2.53	45.80	32.90	8.11
No vaults	0.56	18.53	-70.71	-56.27

Table 3: Comparisons between Eigen frequencies and modal masses in X (longitudinal) direction.

Model	f_{x1} (Hz)	M_{x1} (%tot)	% Actual f_{x1}	f_{x2} (Hz)	M_{x2} (%tot)	% Actual f_{x2}	Sum M_x (%tot)
Actual	4.12	36.79	0.00	4.36	11.99	0	48.78
Simplified	3.87	56.06	-6.00	-	-	-	56.06
Simplif. + add.masses	3.75	26.25	-8.89	3.82	21.96	-12.30	48.21
Horizontal slabs 6 cm	4.88	11.25	18.54	4.96	8.12	13.80	19.36
Horiz. slabs 60 cm	6.09	26.82	48.09	7.17	34.77	64.51	61.59
No vaults	3.50	47.82	-15.08	3.51	10.60	-19.35	58.42

2.2. TIME-HISTORY ANALYSIS

Time-history analysis based on time step integration is often the most reliable approach to evaluate the dynamic response, provided that the damping value and the stress-deformation relationship are accurately considered as material properties. Hereby spectrum-compatible accelerograms (spectra with return period 75 years and behavior factor 1.5) were considered and comparisons were performed between significant points of lateral naves, namely longitudinal, transverse, diagonal relative displacements along the transverse direction of the church (Fig. 12). That points are located at the ribbing arches imposts of lateral naves. They have been chosen to compare the response both in a central zone, subjected to larger transverse displacements, and in a zone near to the main façade. Modal analysis also allows computing relative displacements, but the choice of the mode to consider would be arbitrary. Time histories outcomes are showed just in case of application along the transverse direction (Z), since along that relative displacements are larger and vaults play a more significant role in the overall seismic response.

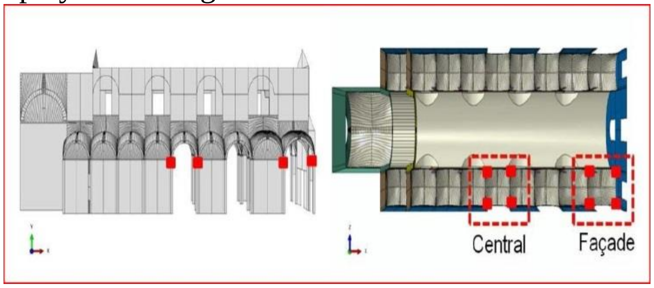

Fig. 11: Nodes for monitoring relative displacements.

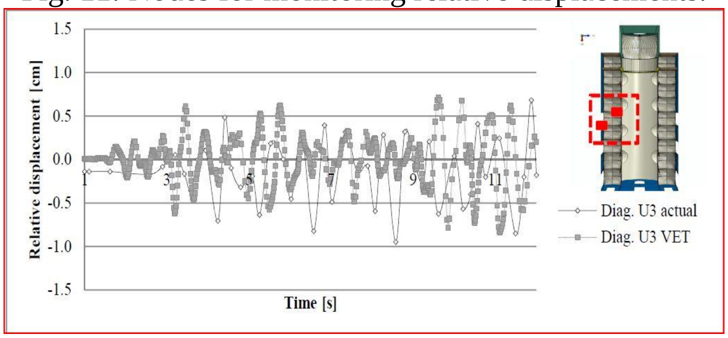

Fig. 12: Time-History Z: relative transverse displacements in diagonal direction (central part of lateral nave).

2.3. SUMMARY OF VET TECHNIQUE

Equivalent trusses, in place of full-modeled vaulted systems, do not alter the global dynamic behavior of a historic church strongly reducing the computational time without invalidating its whole response. Modal and time-history analyses have been used to demonstrate, in an example, the equivalence between simplified and full models, resulting in a good correspondence in terms of global stiffness. Moreover, it was investigated the sensitivity of the dynamic response of the building depending on vaults modeling techniques. Further applications of the illustrated procedure will be performed in nonlinear static and dynamic analyses and kinematic analyses. Parametric analysis may provide diagrams of stiffness depending on vaults thickness, plan dimensions, boundary conditions. Such diagrams can be easily and quickly used for the definition of trusses to be implemented in simplified models. Moreover, by means of the equivalence in terms of steel diameter, the method can become a tool normally used in the designing of safety features such to improve the seismic response of the building.

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