COMPDYN 2013
4th ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, V. Papadopoulos, V. Plevris (eds.)
Kos Island, Greece, 12–14 June 2013

SEISMIC ASSESSMENT OF A NON – SYMMETRIC STONE MASONRY BUILDING WITH FLEXIBLE FLOORS

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Keywords: masonry buildings, strengthening, seismic assessment, relative drift, flexible floor

Abstract. *For the rehabilitation of a 150 years old water mill, seismic assessment was necessary. According to provisions of Eurocode 8, Part 3, it is not applicable to stone masonry buildings. However, due to the lack of a code in force for this type of structures, it was applied to a structure consisted of two buildings which share the adjacent wall. Three alternatives for the simulation of the cooperation of the common wall is considered and also the two alternative analysis methods compatible with the limited knowledge level which was obtained, the lateral force method and the response spectrum modal method is used. The inconsistencies of the code with relevant structures are pointed out and results for each case and analysis method in terms of demanded relative drifts are compared to capacity of the masonry. The main conclusion is that the large gap of codes in force must be filled otherwise the existing non-monumental stone masonry structures will stay unprotected if their strengthening will become a process which demands explicit calculations and inspection works of high cost.*

1 INTRODUCTION

During the redesign process of an existing building a variety of problems arises which are not encountered during the design process of a new building. In stone masonry buildings uncertainties related to the unknown properties of structural masonry, the bonding of the masonry units at the corners or through the width of the wall, the number of wythes, the way floors and beams are supported by the walls etc, are risen. Non destructive tests (NDT) and high technology devices are available nowadays for both the inspection of the masonry and the determination of its properties and not only the choice of each NDT, but also the extent of application depends on the importance of the structure and the budget of the project, as well. Another subject is the way all the collected data about the construction details of a masonry building should be used when modeling the structure for seismic assessment. It is reasonable that treating with a unique monumental structure, the cost for the analysis is of minor importance. The later is the cost to supply a computer program of high capabilities as well as the cost of person hours to model in detail the structure and to evaluate the huge amount of results. But the cost of analysis for the assessment of a more or less non-monumental structure is critical and engineers are often obligated to simplify the modeling by adopting reasonable assumptions which are accepted or proposed by the codes in force.

Worldwide, industrial buildings from the early industrial era, left functionless for decades, were restored and rehabilitated. Being hundreds, their survival is essential for the preservation of both the architectural heritage and the historical memory; they often stand for milestone of a location. Nevertheless, this is not a strong reason to treat them as unique monuments in which renovation cost and especially computational cost is of minor importance. The permissible simplifications are a matter of code and engineer's experience as well.

Eurocode EN 1998-3:2005 [1] is pertained to existing structures but it is mainly oriented to frame structures and especially to reinforced concrete ones. According to EN 1998-3:2005 the capacity of a structure, in terms of deformation, is calculated in control node and it is compared to the demanded deformation which depends on the performance level adopted. This procedure is simple in structures with rigid floors and roof where the control node is the center of the mass at the top of the building. The following methods may be used for the analysis of new structures, depending on the restrictions for the application of each one: (a) the simple equivalent single degree of freedom representation where demand is obtained directly from the spectrum (see appendix B in EN 1998-1), [2], (b) the linear lateral force analysis procedure (static), (c) the multi modal response spectrum analysis (linear, with CQC or SRSS type modal response combination), (d) the non-linear static (pushover) analysis and (e) the non-linear time history (dynamic) analysis. In EN 1998-3 the same methods are adopted for existed structures but instead of design spectra, the elastic spectra must be used. Additionally in informative Annex C of EN 1998-3, it is mentioned that it is not applicable to stone masonry buildings leaving a gap relating to the vast majority of traditional buildings.

It is well known and it is well documented in [3] that the use of non-linear static analysis in case of structures with deformable floors and roof presents difficulties and codes have not yet taken these issues into consideration. In the absence of diaphragms the choice of the control node cannot be the center of mass because each wall is deformed almost independently and this is a matter of study in this paper. Because of the wide variety of masonry types, the non-linear behavior of each type, especially for the out of plane response is not yet well documented. In order to overcome these time consumer and doubtful for these particular structures methods, in [4] and [5] a simple procedure for the seismic assessment of ordinary traditional masonry structures is proposed. Herein, emphasis is given to aspects related to the control node and the analysis method used. A structure consisted of two buildings which are constructed at different time periods, not bonded together is used as benchmark; it is analyzed: a) considering three alternatives for the simulation of the sheared adjacent wall and b) by both a lateral force analysis and a response spectrum analysis considering both the fundamental modes only and mode superposition of the twenty first modes. Displacements of each wall and the maximum relative drift in plane and in elevation is calculated for each one of the analysis methods and of simulation methods.

2 DESCRIPTION OF THE STRUCTURE

The structure is a water mill constructed in two phases from uncoursed stones of local schist in island Andros, Greece, during the years 1876-1900. The older building (Building A in Fig. 1) of dimensions 18,72x12,07m in plan and 19,16m in elevation consists of only external load bearing walls of width ranging from 1,09m to 0.55m as seen in Fig. 2. The newer building B of maxima dimensions in plan 11,34mx8.53m has three external load bearing walls sharing the adjacent wall W2 of building A. The area of ground floor because of the slope of the ground is smaller than of the upper floors (see Fig. 2(d)). As seen in Fig.1 the complex is L-shaped in plan. It is noteworthy to point out that the wall W2 bears loads from both buildings A and B. Another point of interest is that there is not bonding between the masonry units of walls W5 and W7 and the ones of the wall W2 as it is clearly seen in the picture of Fig. 1(c). Although the structure is located in an earthquake prone area, there is no evidence or signs of structural damage except environmental one. This is actualy surprising because of: a) the height of the structure, b) the "open plan" of building B due to lack of connection of one end of the walls W5 and W7, c) the existence of wall W5 which could pound the wall W2 in the middle of its span, and d) the different seismic response of the two buildings.

For the seismic assessment both the properties and the loads of the structure must be known, so for the study of the structure under consideration, in order to calculate the live loads it was proposed a rehabilitation concept fully compatible with the nature of the old water mill and the needs and history of island Andros which exhibits a remarkable cultural life with museum of modern art, libraries, naval museum, municipal theater etc. The proposal does not alter the configuration of load bearing walls and gives the possibility of any use provided the live load does not exceed 2kN/m^2.

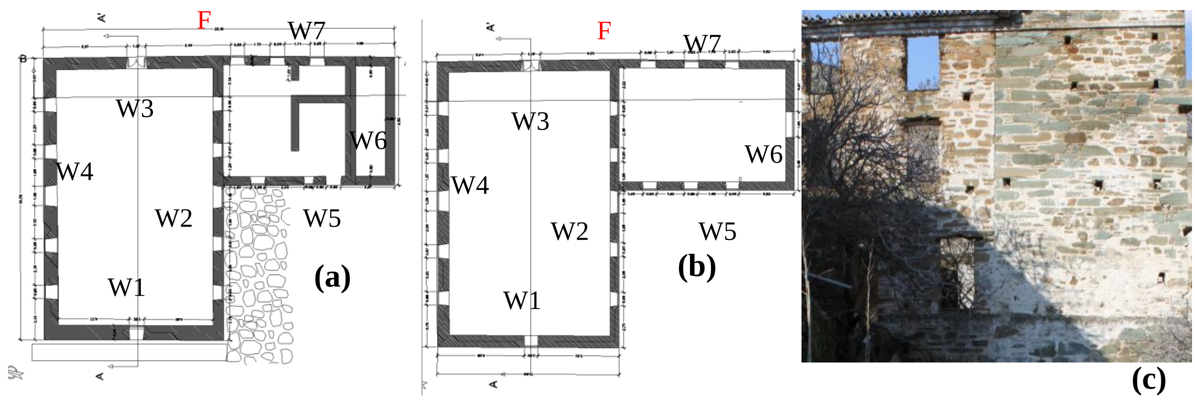

Figure 1: Plans at levels z= 11.22m a), z=13,96m b) and a picture taken from point F which shows the lack of connection among the buildings (c)

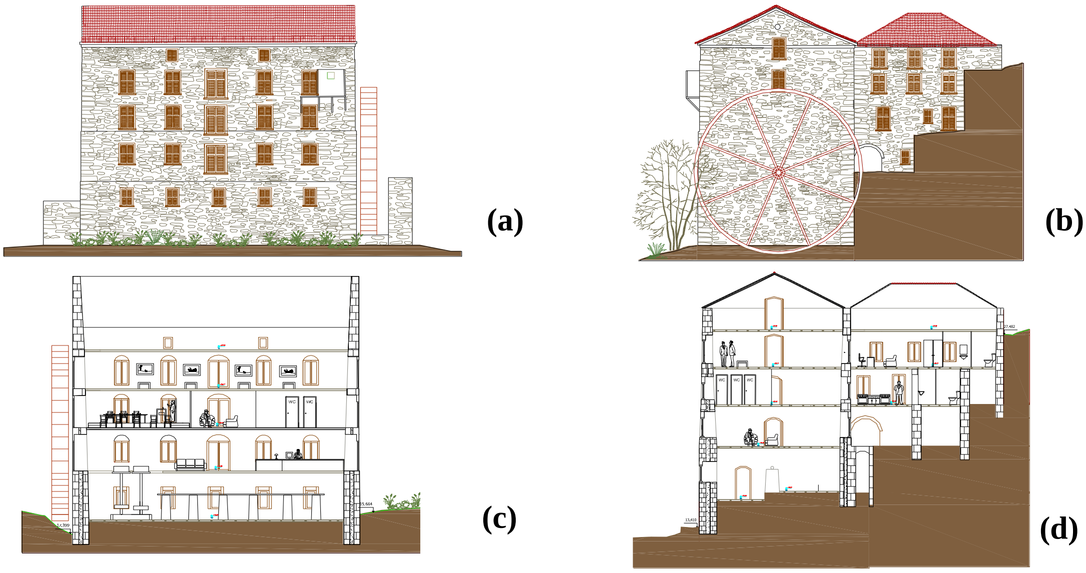

Figure 2: The north, (a) and west facade, (b) and elevations A-A, (c) and B-B, (d)

3 ASSUMPTIONS FOR THE ANALYSIS

3.1 Mechanical properties of masonry

All load bearing walls are of uncoarced stones of local schist. For the determination of compressive strength of stone the Schmidt rebound hammer test was used, (see Fig. 3). The test was performed in twenty seven different locations of lower parts of the walls because due to destroyed floors, there was not access to upper parts the walls. In Table 1 the corrected, according to hammer instructions, values of the compressive strength of masonry units are presented and one can notice the variation of mechanical properties of masonry over the structure. The mean compressive strength of masonry units derived as f_b=36,79 MPa with a standard deviation σ=13,82 MPa. The compressive strength of mortar f_m was estimated equal to 1.0MPa. The compressive strength of masonry f_w =3.03MPa was calculated from equation 1 which was proposed in [6] in which α=1.5 for the uncoursed stones of the structure under consideration.

$$f_w = \frac{2}{3}\sqrt{f_b} - a + 0{,}5 f_m \qquad (1)$$

In the analysis the Young modulus E* = *0,5E* was taken so that cracking is considered, where E=1200f_w from [6]. The specific weight of masonry was estimated 22kN/m^3. The equations of En 1996-1-1 [7] are not applicable to uncoursed stone units.

12,0	45,0	45,0	60,0	26,0	12,0	27,0	46,0	42,0	12,0
25,0	37,0	26,0	50,0	30,0	34,0	26,0	34,0	45,0	39,5
39,0	39,0	62,0	34,0	34,0	59,0	54,0			

Table 1: Compressive strength in (MPa) of masonry units after Schmidt rebound hammer test

3.2 Loads

For the calculation of dead loads the weight of masonry, the timber roof and the timber floors as well was taken into consideration. The floors consist of steel beams parallel the small side of each building which transfer the loads to the longitudinal walls. The roof trusses of building A are supported by the walls W4 and W2, part of the later and the walls W5, W6, W7 transfer the loads of the roof of building B.

Type I spectrum of EN 1998-1:2005 with a_g= 0,16g from the Greek National Annex and S=1.0 because of the rocky soil was used for the seismic analysis. More details about the seismic analysis are presented in a separate following section.

Figure 3: Hammer Schmidt test

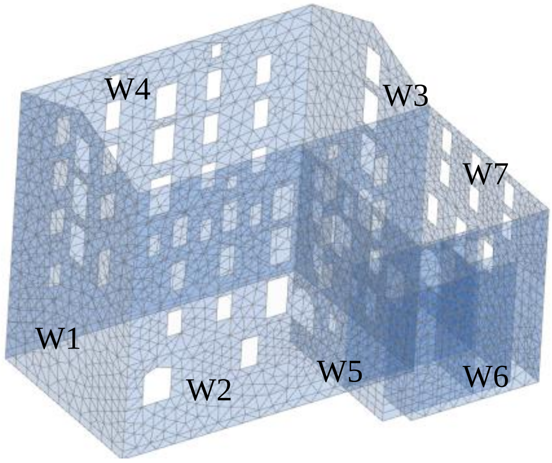

Figure 4: Finite element discritization

4 ALTERNATIVE SIMULATIONS

In Section 2 it is pointed out the lack of bonding of walls W5 and W7 of building B with the adjacent wall W2 of building A, which however bears part of the loads of building B. This fact raises the question for the proper simulation of the entire structure and its response under earthquakes. For a very eligible simulation the use of special contact elements provided by program codes, not so familiar to everyday practice of engineers, is recommended. The scope of this study is to compare the results of the three possible alternative simulations which an engineer may proceed using wide spread program codes. In all three cases, the spatial model of the structures is discritized with shell finite elements which take into account the in-plane and the out -of -plane response of the walls in the absence of rigid diaphragms. The finite elements are of dimensions about 0,5 x 0,5 m in their plane and of width equal the actual one of each wall. In Figure 4 the finite element discritization is shown. The three alternative cases considered in the present study are:

Case I: in this case the two buildings are considered as a unique non-symmetrical in plan structure, namely as if the masonry units of wall W2 are bonded to the ones of walls W5 and W7. In case steel connectors will be placed to tie the walls together, the assumption of the considered full cooperation of the walls is valid.

Case II: in this case there is no connection between the two buildings and each one is more or less symmetrical building independently analyzed, provided a wall transverse the free ends of the walls W5 and W7 and parallel the wall W2 is constructed to outcome a cellular formation of load bearing walls in the plan of building B.

Case III: in this case, the addition of the wall of case II is omitted and the building B is of an open U-shaped plan. For comparison reasons, no redistribution of loads was considered in this case; taking into account the low values of dead and live loads of wooden floors, this assumption does not affect the response of structure.

5 SEISMIC ANALYSIS

According to modern codes for structural performance to near linear behavior, the linear analysis is adequate and economical because behavior will be essentially elastic in regular structures with short periods and linear static procedures are adequate. For regular structures with long periods and all irregular structures, linear dynamic procedures are better and response spectra is accurate enough. In EN 1998-3, Annex C it is clarified that the code is not in force for stone masonry buildings as well as linear analyses may be performed in case of floors and roof stiff enough that service like horizontal diaphragms, conditions which is not fulfilled in the majority of existing masonry structures and hence nonlinear procedures are requested. However also according to EN 1998-3, nonlinear analysis is not applicable in case of limited knowledge level, KLI. Herein, knowledge level higher than KL1 was not possible at a reasonable cost and the two permitted methods of analyses for this level of knowledge are performed for a stone masonry structure with flexible floors despite the fact the conditions for their application are not fulfilled. The linear methods performed are:

- A modal response spectrum analysis using the Type I elastic spectrum of EN 1998-1:2005 with ground acceleration a_g = 0,16g ground of type A with S=1.0 and characteristic periods T_B and T_C 0,15 and 0,40 sec respectively.

  Seismic codes demand a combination of modal maxima in order to estimate “design” values, but contrary to what is seen in lumped systems, where the fundamental mode is usually the translational mode, engaging very large fractions of mass participation (over 75%), several tens to hundreds of modes need to be considered when applying the same procedures in structures with distributed mass before a tolerable amount of mass may be excited (less than 65%), whereas it is very difficult to identify the fundamental translational mode from among the multitude of modes estimated, which can be related to the vibration of a subordinate component (such as a spandrel or an intermediate wall, or even a beam). Herein, results are presented for the superposition of the twenty first eigen modes as well for the first fundamental eigenmode for each direction (herein an eigenmode identified as fundamental when the effective mass is over 10% of the total mass). In Table 2 the effective mass as percentage of total mass participated in each egeinmode (denoted as "mpf") is presented for the first twenty modes for each one of the cases examined as well as the corresponding periods, in sec. In the relevant Figures 5 and 6 the deformation in plan and in perspective for both the fundamental modes and the superposition of the twenty modal shapes, are depicted. In case I, the mass participated in each mode is very low and a small percentage of mass (about 41%) is activated even twenty modes are considered. When 50 eigenmodes are considered, (66% in x and 70,1% in y of the mass is participated) both the time cost and the required free computer space increased disproportionally the accuracy of the results as it will be discussed in the next. Practically, it is difficult to point out the fundamental eigenmode in case I. In case II, where each building is considered separate from the other, the mass contributed after the twenty eigen modes considered, is about 70%; the same states for case III. Difficult, but easier than in case I, one can distinguish first modes for the building A and higher modes for the building B, as fundamental ones.

- A linear later force analysis (LF) procedure (static) loading the structure with an horizontal force F uniformly distributed along the height of the building from Equation 2, along horizontal axes x and y combined with dead and live vertical loads.

  $$F = mS(e) \tag{2}$$

  where m is the mass of the structure and $S(e)$=0,40g (m/sec^2) a uniform acceleration from the equation 3.

  $$T_B \leq T \leq T_C : \quad S(e) = a_g \cdot S \cdot \eta \cdot 2,5 \tag{3}$$

where for ground of type A, S=1,00, T_B = 0,15sec, T_C=0,40sec, damping ratio ξ=5% ($\eta = \sqrt{\frac{10}{5+100\xi}}$=1 for ξ=5%)

mode	Case I			Case II-Building A			Case II-Building B			Case III-Building B		
	T(sec)	mpf x %	mpf y %	T(sec)	mpf x %	mpf y %	T(sec)	mpf x %	mpf y %	T(sec)	mpf x %	mpf y %
1	0,35	**10,8**	0	0,52	0	**27,8**	0,26	0	0	0,51	11,6	0
2	0,32	4,5	0,6	0,35	**15,2**	0	0,25	0	0	0,28	0	0
3	0,29	0,3	**18,4**	0,32	6,5	0,1	0,25	0	0	0,26	0	0
4	0,26	0	0	0,29	0,3	0	0,23	**24,1**	0	0,25	0	0
5	0,25	0	0	0,23	0	0	0,21	0	0	0,25	0	0
6	0,25	0	0	0,23	0	0	0,2	0	**33,9**	0,24	2,2	0
7	0,23	0	0	0,23	0	19,5	0,19	0,1	0	0,2	0	27,6
8	0,23	0	0	0,21	0	0	0,18	0	4,9	0,19	0	0
9	0,22	0	0	0,21	13,1	0,9	0,15	0	0	0,18	**18,7**	0
10	0,22	7,2	0,2	0,21	1	5	0,14	14	0,1	0,18	0,1	0
11	0,22	1,5	6,1	0,2	0	0	0,13	8,5	0,6	0,16	0,8	0,1
12	0,21	0	0	0,2	0	11,9	0,12	0,1	30,5	0,13	0,4	**33,1**
13	0,21	8	0,6	0,19	0	0	0,11	0	0,4	0,13	2,2	1,9
14	0,2	0	0	0,19	0	0	0,1	6,2	0,4	0,12	0	0
15	0,19	0,2	12,9	0,19	15,1	0	0,1	8,3	0,2	0,11	0	0,7
16	0,19	0	0	0,17	0	0	0,1	5	0	0,11	9,3	0,1
17	0,18	1,5	3,4	0,16	0	0	0,09	0	0	0,11	11,8	0,9
18	0,18	7,1	0,4	0,15	0,7	0,6	0,08	2,3	0	0,1	1,2	1,8
19	0,17	0	0	0,14	11	0	0,08	0,2	0	0,1	0,2	0,2
20	0,16	0	0	0,14	0	0	0,08	0,3	0,2	0,09	12,1	0,1
total		***41,0***	***42,6***		***63,0***	***66,0***		***69,0***	***71,3***		***70,5***	***66,7***

Table 2: modal mass participated (mpf) in each case

		Mode supp. x		Mode supp. y		Fundamental Mode x		Fundamental Mode y		LF-x		LF-y	
	wall	u	v	u	v	u	v	u	v	u	v	u	v
Case I	**1**	36,2	2,4	16,5	7,7	12,4	-0,1	-7,1	5,40	25,3	-0,2	1,3	7,6
	2	2,8	6,4	2,8	**25,3**	1,3	0,5	1,2	**23,6**	5,3	-0,4	1,9	17,0
	3	**54,4**	1,1	9,9	1,9	**46,3**	0,5	-0,4	0,1	**26,5**	-0,2	-0,6	2,4
	4	3,4	7,7	3,9	24,9	2,5	-3,2	2,3	23,0	6,1	1,6	-1,3	**17,4**
	5	**16,1**	2,3	6,3	4,6	1,8	-0,3	-1,8	**3,5**	9,7	-0,7	1,1	5,4
	6	1,7	12,4	1,5	**21,7**	-0,1	-0,2	0,80	2,9	2,6	-1,2	0,4	**10,2**
	7	16,1	0,8	5,8	1,7	**-2,5**	0,2	-1,0	-0,3	**10,4**	-0,5	1,20	2,8
Case II	**1**	37,4	1,0	5,3	5,4	12,1	0,2	2,9	2,8	25,4	-0,2	2,0	8,4
	2	4,2	5,2	2,80	54	2,8	3,9	0,40	52,2	5,9	2,1	0,2	54,2
	3	**57,6**	2,8	9,5	6,0	**49,7**	0,1	-4,3	3,6	**28,4**	0,2	-2,4	9,8
	4	4,0	5,0	2,8	**54,5**	2,5	-3,3	-0,3	**53,0**	6,1	2,1	-0,9	**55,7**
	5	**15,7**	1,8	2,5	2,7	**15,2**	-0,9	-1,40	2,5	11,0	-0,9	0,8	3,1
	6	2,2	2,9	0,5	**18,1**	0,4	1,7	0,3	**16,3**	2,5	0,7	0,2	**9,2**
	7	15,6	1,8	2,3	2,6	**15,2**	0,9	1,8	2,2	**11,6**	-0,3	0,8	3,0
Case III	**5**	52,0	1,1	3,0	2,9	**-11,1**	0,6	-1,5	1,80	55,1	-0,4	-0,8	2,9
	6	1,8	1,2	0,9	**16,0**	1,1	0,6	-0,6	**-4,5**	2,8	-1,2	-0,2	**9,1**
	7	**52,2**	1,1	2,8	2,8	-8,3	-0,6	-1,7	1,7	**55,9**	-1,4	0,8	2,8

Table 3: Maximum displacements u, along x and v, along y, in mm, of each wall for each case and analysis method

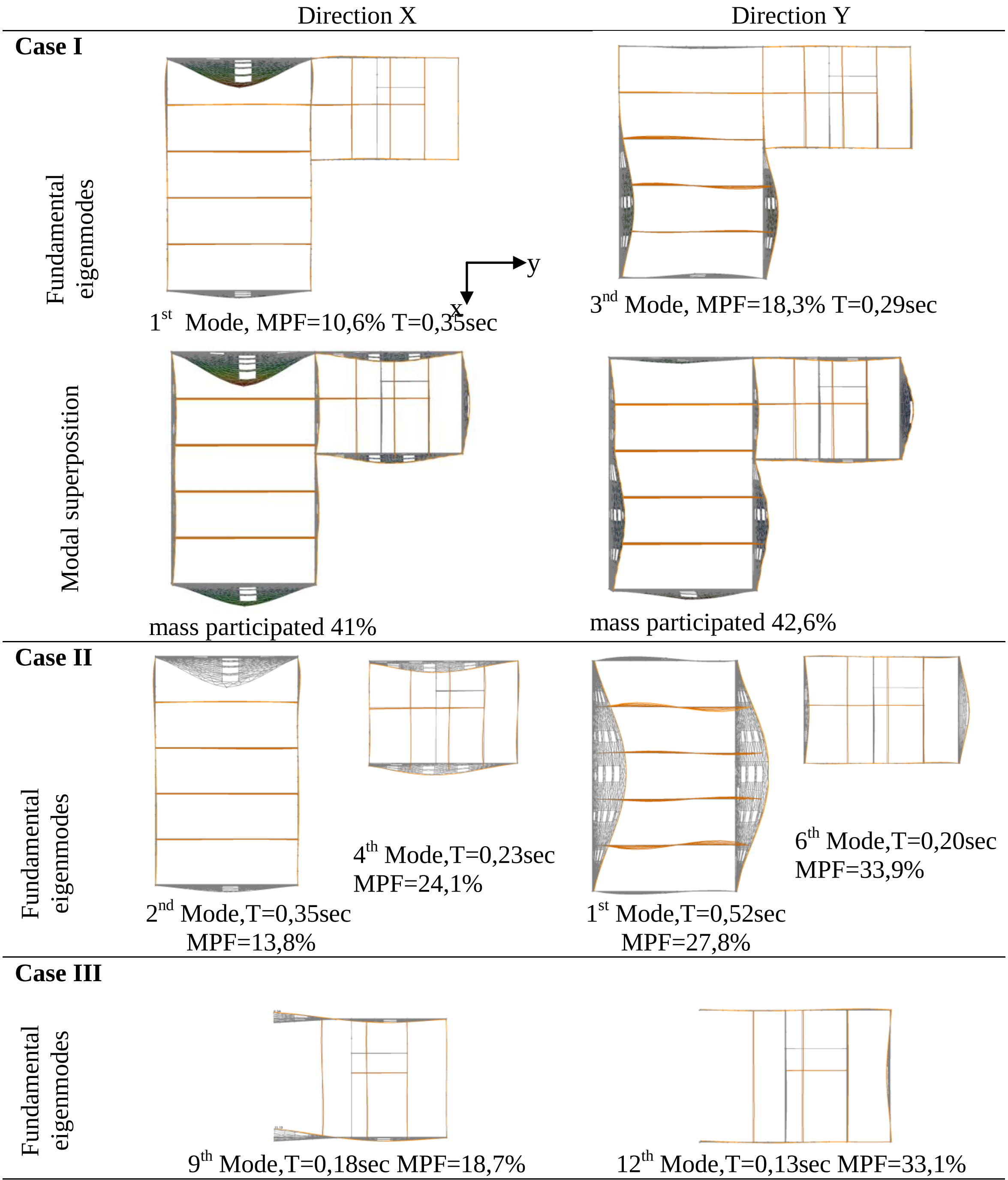

Figure 5: Modal shapes along X (left) and Y (right) axes

For the analyses, the program code Acord-expert [8] was used. The maxima displacements u and v along x and y axis, respectively, of each external load bearing wall of the building, are depicted in Table 4 for the three simulation and the two analysis methods. In bold letters the maximum displacement of the whole structure (case I), for both the buildings A and B (case II) and only the building B in case III is denoted. From the table and the Figures 5 and 6, it is obvious that the out of plane bending is the dominate response of the walls. In case I, the displacements of building A are smaller than in case II when each building considered separate from the other, especially those ones of the wall W4, which is at the opposite of building B. On the contrary, displacements of building B are greater in case I than in case II when

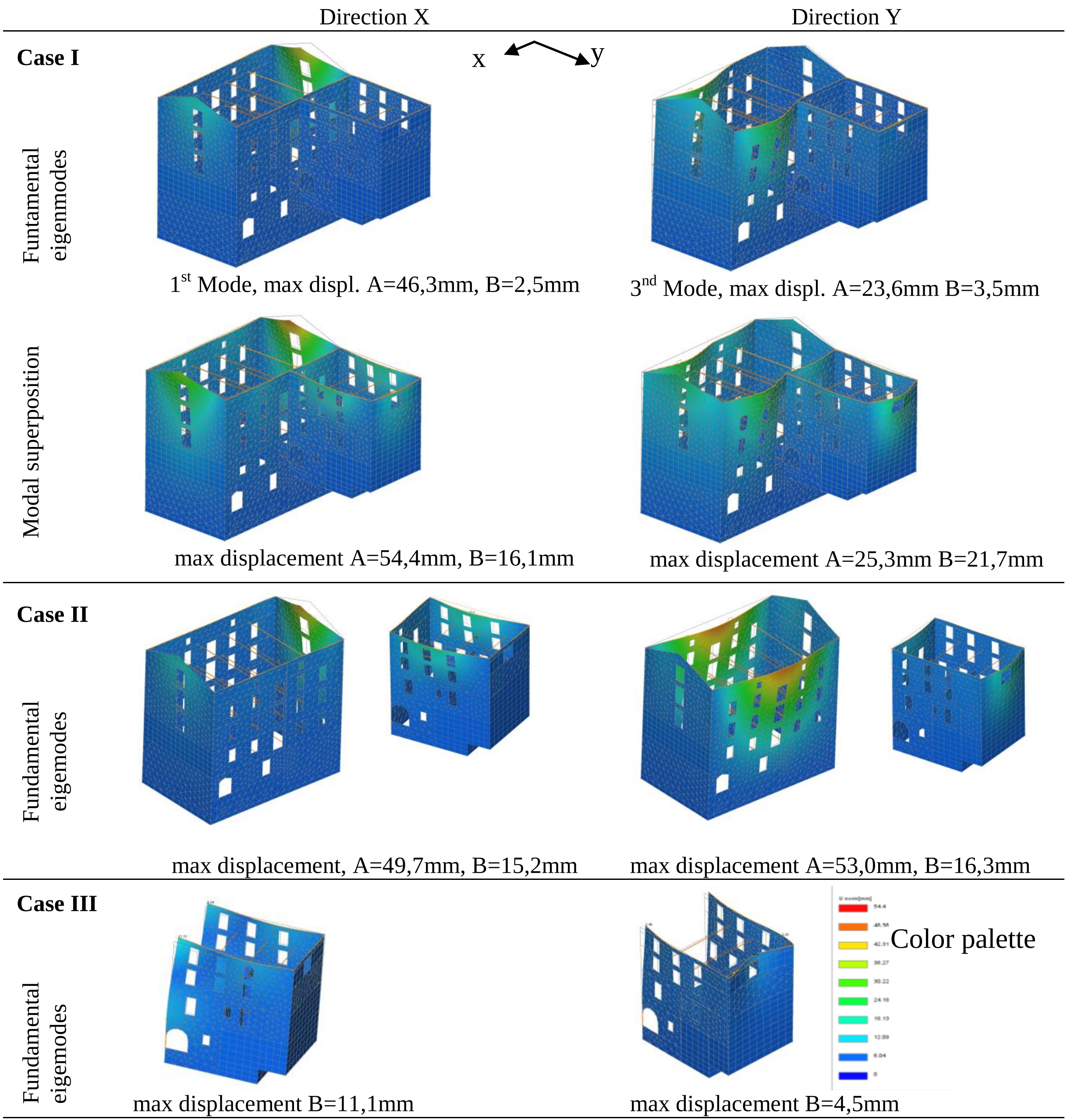

Figure 6: maxima displacements of buildings A and B, for excitation along x (left) and along y (right)

superposition of modes was considered, but times smaller if only the fundamental mode shape is regarded. In case III, as it was expected, when walls W5 and W7 are free at one edge, the deformations of building B along x axis are three to four times larger than in cases I and II. On the other hand, not only the value of maximum displacement but also the associated wall differs according to the analysis method. At this point it must be added that when fifty modes are superimposed, the displacement u of the wall W3 is 55,0 mm instead of 54,4m or 1% greater in case of twenty modes and of the wall W7 16,6mm instead of 16,1mm, or 3% greater. According to EN 1998-3 for the seismic assessment the seismic demand for the performance level considered, e.g. the drift at the top of the structure, is compared with the corresponding capacity of the structure. In case of flexible roof as it is seen in Table 4 the displacement of each external wall is different irrespectively the modeling considered. In [4] and [5] the introduction of two relative drifts with the greater one to be a measure of the seismic demand, which will be compared to the capacity of the structure, in terms of relative drifts, is discussed in detail.

Relative drift (in height) measures the rotation of the structure at the point of peak lateral response from the vertical axis: θ_v is defined by the ratio of relative displacement occurring between two reference points located at different heights (z_1 and z_2) on the same vertical line, divided by their distance, (z_1-z_2). θ_v is owing primarily to the shear distortion of walls oriented parallel to the ground motion, (θ_{sh}), as well as to the out of plane flexural action of walls oriented in the orthogonal direction (θ_{fl}). Figure 7 clarifies the concept of the relative drifts. For the out of plane response, *relative drift in plan*, θ_{plan}, refers to the relative displacement of the point with peak outwards deflection as compared to the wall corner. In Table 4, where the maximum relative drift for in-plane and out-of-plane deformation is presented for the three alternatives of modeling and the two methods of analysis, the maximum relative drift irrespective of the analysis method is in bold. In Figure 8 the relative drifts are presented in diagrams for better comparison. In all the three cases, the relative drift θ_{sh} is very small compared to θ_{plan}. Relative drift θ_{plan} is almost three times greater than θ_{fl} in cases I and II of "close" plans and almost equal in case III of "open" plan. Shear relative drift is very limited compared to flexural drift. The results obtained from the lateral force method are closer to those from multimodal response spectrum than the ones obtained from modal response only in case III.

			Modal sup.		Fundamental Mode		LF	
	Building		‰	wall	‰	wall	‰	wall
Case I	**A and B**	θ_{plan}	***9,161***	3	7,857	3	3,661	3
		θ_{sh}	0,402	1	**0,647**	1	0,397	1
		θ_{fl}	**2,839**	3	2,416	3	1,383	3
Case II	**A**	θ_{plan}	***9,571***	3	*8,964*	4	5,271	4
		θ_{sh}	**0,282**	1	0,271	3	0,618	3
		θ_{fl}	3,006	3	3,340	4	**3,510**	4
	B	θ_{plan}	**3,919**	6	3,565	6	1,587	7
		θ_{sh}	0,254	5	0,235	5	**0,256**	5
		θ_{fl}	**1,704**	6	1,535	6	0,957	7
Case III	**B**	θ_{plan}	4,394	7	2,585	5	**4,629**	7
		θ_{hs}	**0,248**	5	0,149	5	0,239	5
		θ_{fl}	4,307	7	0,916	5	**4,612**	7

Table 4: Maximum relative drifts

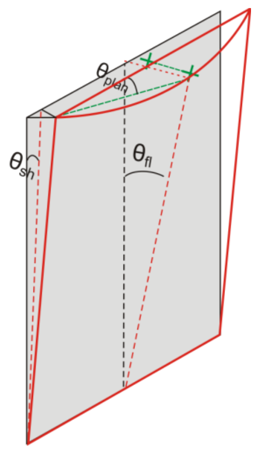

Figure 7: Relative drifts

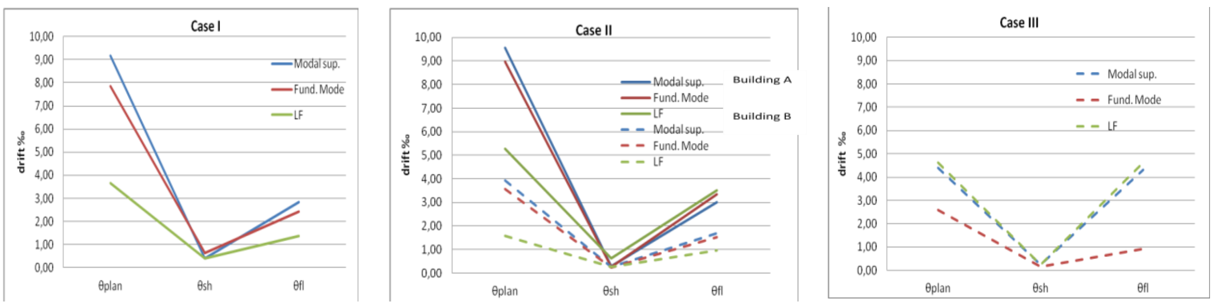

Figure 8: Relative drifts for the three simulation cases and the analysis methods

The relative drifts θ_v and θ_{plan} is the seismic demand and must be compared to the capacity of masonry. In EN1998-3 for the limit state "severe damage" the capacity of a wall of plain masonry (from brick or concrete units) for primary members in terms of drift is 0,004 if governed by shear and 0,008 H_o/D if governed by the in plane bend, where H_o is the distance between the section to be verified and the contraflexure point and D is the depth of the wall. As pointed out the structure of the current study is governed by the out of plane bending so EN 1998-3 does not suggest the capacity of masonry. It is clear that 4‰ is higher than the maximum demanded 0,6‰ and shear failure is not expected under the design earthquake. If we make the assumption that the relation suggested for the in-plane bending is also valid for the out-of-plane capacity, we must accept a rather high capacity of unreinforced stone masonry walls. As an example, for the wall W3 with depth, D=12,07m and H_o =19,16m the capacity in terms of maximum drift for in plane bending equals 10‰ which is greater than the maximum demanded θ_{plan} =9,57‰ (see Table 4). According to this, no strengthening of the structure is required. It is necessary to draw attention that regardless EN 1998-3 is not applicable to stone masonry structures, this conclusion derived from two assumptions which is not allowed to apply in this particular structure, in the lack of any other codified choice : a) nevertheless the absence of horizontal diaphragms linear analyses were performed because the knowledge level achieved did not allow non-linear ones, and b) capacity, in terms of relative drifts, suggested for in plane response is adopted for the out of plane one as well.

6 CONCLUSIONS

Although the existing stone masonry buildings with flexible floors and roof represent a large part of the building stock of Europe, part 3 of Eurocode 8 does not cover them for seismic assessment. As suggested, in order the analysis methods of EN 1998-1 to be performed, detail documentation is available so that a knowledge level higher than limited (KL1) to be obtained; for ordinary traditional masonry buildings this is a matter of cost.

The outcome of the application for both the lateral force method and the modal response spectrum analysis (in the later considered either the fundamental mode or the modes superposition), is different drift ratios without any systematic deviation to the three cases examined.

In case of adjacent buildings, the impact of sharing the adjacent wall may be either positive or negative to the building considered. A trend to conservative results in case the adjacent structures were not considered is coming into view.

The need for accounting the out-of-plane response by the codes is of great importance. The large gap of codes must be filled otherwise the existing non-monumental stone masonry structures will stay unprotected if their strengthening will become a process which demands explicit calculation and inspection works of high cost.

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