Second Conference on Smart Monitoring,
Assessment and Rehabilitation of Civil Structures

SMAR 2013

Criteria Guiding Seismic Upgraded Strategies of Traditional Masonry Buildings in Greece

F.V.Karantoni[1] , M. Papadopoulos[2] and S. Pantazopoulou[3]

[1]University of Patras, Patras, Greece
[2]Demokritus University of Thrace, Greece
[3]Univ. of Cyprus (on leave of absence from Democritus Univ., Greece)

ABSTRACT: Objective of the present work is to establish a framework for seismic assessment and retrofit of traditional unreinforced masonry structures. Both demand indices and acceptance criteria are geometric variables related through derived expressions with the fundamental response of the building. These include drift ratios that quantify the intensity of out of plane differential translation and in plane shear distortion of masonry walls oriented transversally to and along the seismic action, respectively, for in plane and out of plane deformation. This framework is particularly useful for setting retrofit priorities and for management of the collective seismic risk of historical settlement entities. A characteristic Balkan type of traditional building is used in the study as a model structure for illustration of concepts. The structure represents the construction methods and building characteristics of the historical town of Xanthi (Greece).

1 INTRODUCTION

Stone masonry construction has been used throughout the Balkans for building structures from time immemorial. A traditional unreinforced masonry (TURM) building type comprising timber-laced construction (TL), was the preferred structural system in several cities of Northern Greece up to about 60 years ago when it was displaced by reinf. concrete. This structural system, known by various local trade names, actually draws its origin from the ancient Minoan times; timber lacing is used as a metaphor in the Bible, "*where the strength of soul secured by faith at times of trial, is compared to the strength of houses imparted by timber lacing in the event of an earthquake*". In Herculaneum, near Pompeii, samples of timber laced buildings that survived the volcanic eruption of 78 A.D. are still standing, considered by historians a low-cost type of construction dating from that ancient period, referred to as *opus craticium* in roman times [1], [4] (Fig. 1a). This later became known as *fachwerk*, *chatmas*, or *half-timbered system* in the various parts of Europe and Asia where it was found (Fig. 1b, 1c) [2], [3].

With regards to the old-town of Xanthi the TURM-TL buildings comprise a vital portion of its historical

Figure 1: (a) Timber-Laced Masonry House from Herculaneum dating 78 A.D.), (b) Timber Laced Wall, (c) Stone masonry timber laced wall

fabric, identifying the city (Fig. 2). Primary construction materials are stone (natural blocks, usually in the foundation and in the lower floors, and man-made solid clay-bricks in the upper levels) and timber (such as timber structural elements, floor ties, timber lacing elements, etc.), often tied in strategic locations with iron clamps and ties to improve member connectivity. The structural system combines a stiff load-bearing timber-laced stone-masonry wall system for the lower floor, with the upper floor made of an infilled timber frame, particularly in the southern or south-eastern sides of the building. The load bearing structure comprises stone masonry foundation with connecting mortar; in some cases, to improve the redundancy of the foundation particularly in compliant soils, a supporting substrate layer made of treated timber is provided under the foundation.

Load bearing walls in the first floor including the major interior divisions are made of stone masonry with lime-type connecting mortar and carefully tied timber-laces. Frequently, the connections between timber laces reveal many techniques borrowed from the local ship-making industry. Secondary interior dividing walls were made of light timber-woven gages coated with a lime-based mortar (mud-based mortar was used in poorer dwellings), usually reinforced with straw or animal hair; this is also evident in ancient monuments, but its use is found throughout southern Europe and Asia. In construction of a traditional house these three structural forms were used selectively, combined in an overall structural system and expanded in space following well-defined rules depending on their weight, load-carrying capacity, and stiffness so as to optimize distribution of mass, stiffness and deformation compliance. Energy dissipation through internal friction is a characteristic mechanism for all three structural forms described (laced masonry, infilled timber frames and timber-woven walls), extending over a large range of deformation capacity prior to failure. This type of behavior to seismic loads is enhanced by the partial diaphragm action of the floor system, to a degree that depends on the robustness of its structure and the manner of its connection or attachment to the load bearing walls. In many of these buildings the roof timber truss is elastic and does not contribute by diaphragm action to the structure.

Figure 2: Typical Samples of traditional houses in the historical center of the city of Xanthi.

2 A SAMPLE STRUCTURE REPRESENTATIVE OF THE BUILDING POPULATION

After assembling an extensive database that record the local characteristics of the TURM-TL building population of the city of Xanthi [9], a representative sample with geometric and construction characteristics that correspond qualitatively to the median of the values and details of the database was developed to be used as an object of study of the available assessment procedures in this class of buildings (Fig 3). The building consists of timber-laced stone masonry with lacing at regular intervals throughout the exterior walls of the lower floor and the northern side of the second floor (Fig. 3(d)). It is a typical middle-class traditional house with a morphology that combines the typical masonry-trade characteristics and the neoclassical elements that were often included due to the european influences imported by the local tobacco merchants. The south side of the buildings usually consists of a timber-laced frame that is set out (protrudes) in the corner relative to the supporting masonry walls of the first floor in a so-called “bay-window” or “erker” or “sahneshi” formation (for example, Fig. 2b). The way the protruding part of the structure is supported is as follows: a horizontal beam running parallel to the first-floor exterior masonry wall is supported on diagonal timber braces defines the lower end of the bay. The diagonal braces are fixed in one of the horizontal laces of the lower floor masonry wall; first storey floor beams supported on the exterior masonry wall extend outwards up to the end of the bay and are supported on the perimeter beam described above. The timber-laced infilled frame walls of the protrusion are supported on the perimeter beam and floor. Interior divisions both in the first and second floors comprise timber infilled frames which are integrally functioning with the overall structure to secure its characteristic resilient earthquake behavior.

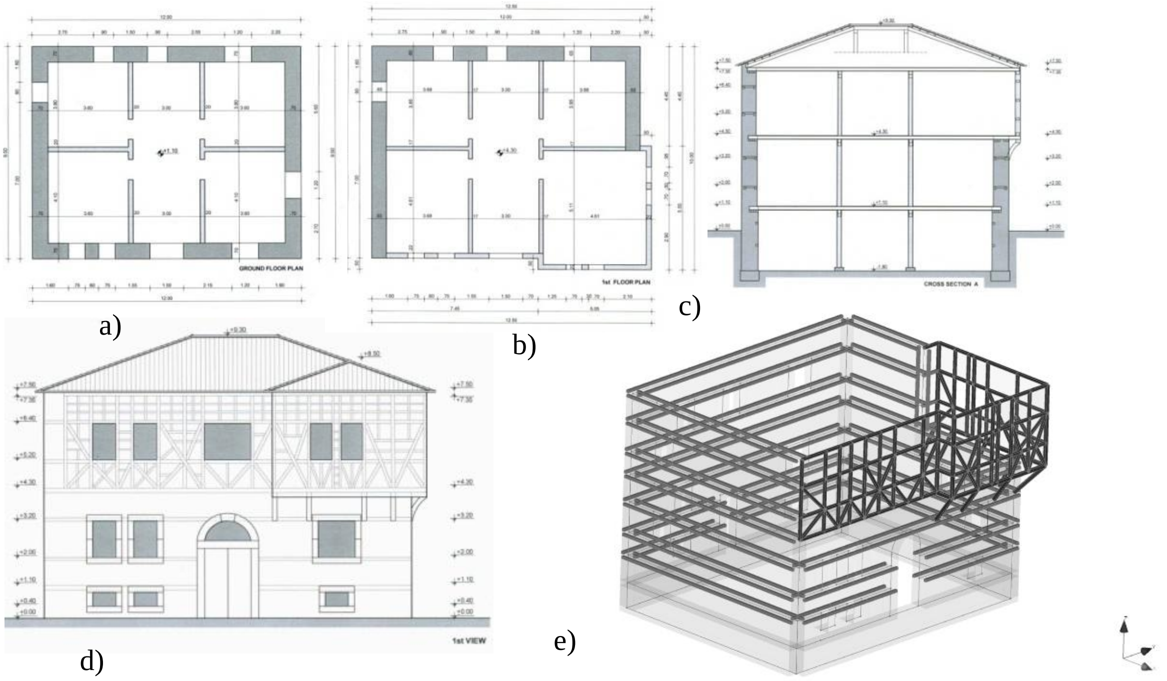


Figure 3: Representative sample building from the historical city core of Xanthi. (a) Plan view of first floor, (b) Plan view of upper floor, (c) Building cross section, (d) Front view, (e) Arrangement of timber laces

To evaluate the anticipated seismic response of the structure a 3-d finite element model was assembled for linear elastic analysis (Fig. 4); shell elements were used to model the perimeter walls including the upper-floor south-east facing bay. Interior walls were not considered for simplicity (as recommended by EC8-III, secondary elements may be neglected in the model). Beam elements were used to model the timber-laced frame, connecting nodes of the underlying shell mesh in a manner that mimicked the geometry of the frame (Fig. 3(d)). Type-I earthquake spectrum as prescribed in EC8-I was used to represent the seismic hazard, with a peak ground acceleration (PGA) of ag = 0.16g (coord. on the left side of the acceleration spectrum) which is the design acceleration level for Xanthi (Seismic Zone I as per the Hellenic Nat. Annex to EC8-I).

Properties of the Model Structure

The building has an orthogonal plan arrangement, 8.8m × 11.3 m in the x, y-directions. The foundation comprises 0.85 m thick walls extending 0.5m below grade in the basement which is a -1.8m from street level; perimeter walls are 0.75m thick, timber laced with embedded timber laces about 0.1m thick, extending over the entire perimeter and spaced at 0.8m height-wise. Interior divisions are infilled timber frames of about 0.2m thickness. Basement height is 2.9 m (clear height is 2.72m). The first floor is raised at +1.1m from street level, having a total height of 3.2m (clear height of 3.02m), whereas the floor diaphragm thickness (comprising beams and floor planks) is 0.18m. The upper floor at +4.3m from ground level has a total height of 3.0m (clear height of 2.82m), and a floor thickness of 0.18m. Perimeter masonry walls are 0.65m thick, timber laced over the entire perimeter spaced at 0.8m in height. Interior divisions are infilled timber frames -0.17 m thick, whereas the southern and eastern exterior timber-frames including the range over the bay are 0.22m thick. The timber roof is at +9.30m, comprising timber trusses, roof cover and byzantine-type roof tiles. The compressive strength of the stone masonry walls is taken equal to f_w=3.0 MPa, and the assumed effective modulus (50% reduced from the nominal elastic value to account for cracking according to § 9.4 of EN 1988-1:2005) is, E_{eff} = 1500MPa. Effective cracking strength of the masonry, f_{tw}, is taken about a tenth of f_w (=0.3MPa). Specific weight of the masonry walls is taken 22 kN/m^3, whereas timber used throughout the structure is classified as C14 ($E_{0,mean}$ =7GPa, specific weight of 5.5 kN/m^3).

3 SEISMIC ASSESSMENT OF TURM-TL BUILDINGS

According with EC8-III which sets the framework for assessment and retrofit of all type of existing structures, the various alternative methods which have been proposed in order to estimate demand for conventional reinforced concrete buildings, theoretically are also applicable to also apply to URM buildings including the traditional types. Thus, (a) the simple equivalent single degree of freedom representation where demand is obtained directly from the spectrum (see appendix B in EC8-I), (b) the linear lateral force analysis procedure (static), (c) the 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 are all considered applicable in this domain. It is relevant to underline at this point the significant differences between frame structures for which the above methods have been extensively proof-tested and URM structures: the former are marked by lumped masses at the floor levels (discrete system), whereas the latter have distributed mass throughout the structure (distributed or continuous system). By default this difference raises the level of difficulty in required modeling of URM as compared with RC structures even while in the elastic range of response (shell elements in a spatial mesh are needed), whereas all points of contact between different materials should be represented with nonlinear gap/spring elements in order to reflect the localized compliance. The result is that the level of confidence in the results is disproportionately lower than the effort required in conducting the analysis particularly with (d) and (e). Furthermore, with regards to option (c) which leads to combination of modal maxima in order to estimate "design" values, it is also relevant to note that 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 be considered when applying the same procedures in distributed systems such as URM structures 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 relating to the vibration of a subordinate component (such as a spandrel or an intermediate wall, see Fig. 5). In this light both the CQC and SRSS approaches yield excessively overestimated values the irrelevance of which can lead to excessive interventions if used as benchmark for acceptance criteria, since no building would actually be able to sustain the levels of the iconic demands thus estimated (Fig. 5d, Table 1); most remarkable is that the estimated values for the displacements exceed by a factor of 2 the value associated with the estimated translational period of the structure, corresponding to the displacement value that would be developed by a structure with a period of at least 1 sec.

Assessment Response Indices

Systematic seismic assessment and upgrading of traditional masonry buildings to levels comparable with modern requirements for seismic resistance of residential structures requires analytical methods that will identify possible damage localization. Here, damage is identified by the amount of deformation occurring in the various components of the structure; in structural components deformation is measured by the relative displacement, or preferably, by the relative drift ratio between successive points of reference. Relative drift ratio (θ) is the displacement difference that occurs between successive points, normalized by their distance; being a non-dimensional parameter it may be used in direct comparison to material deformation capacity at milestone points of response (cracking, rupture, collapse) both in plan and height-wise.

TURM buildings typically have flexible diaphragms and as such, are particularly vulnerable to out of plane bending of walls oriented orthogonal to the earthquake action. For this problem, *relative drift in plan* - θ_{plan}, refers to the relative displacement of the point with peak outwards deflection as compared to the wall corner. Similarly, *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}). All these assessment parameters are geometric quantities. The magnitude and localization of these during seismic response, at least the peak values which are of interest in practical design, are implicitly contained in the normalized deflected shape that the structure assumes at peak displacement under the design earthquake.

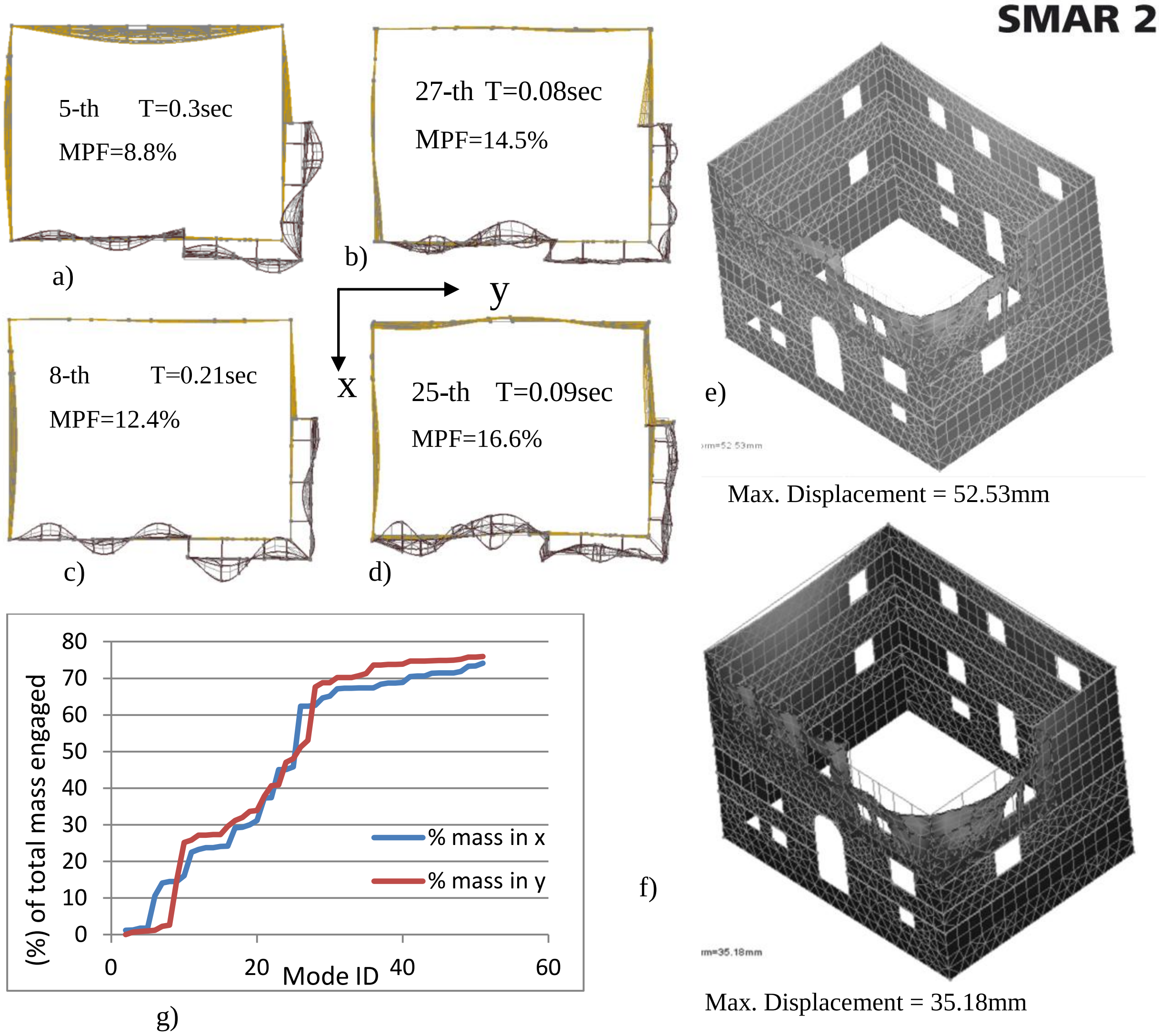


Figure 4: (a)-(d) Plan view of various modes and corresponding mass participation factors (MPF). (e), (f) Displacement values estimated from modal CQC for combined gravity loads and Earthquake action along x and y. (g) Percent of total mass engaged in vibration as a function of the total number of modes considered.

<u>Essential Elements of a Seismic Assessment Method for TURM-TL Buildings</u>

In the context of an equivalent single degree of freedom formulation (ESDOF) used routinely according to design and assessment codes (EC8-I, EC8-II, 2005) for estimating dynamic response, the generalized properties of the structure are obtained with reference to the shape of the fundamental mode of lateral translation; thus, if $\Phi(x,z)$ is the normalized deflected shape of a structure undergoing a ground motion in the y-direction, then the peak displacements at any point in the structure may be calculated from the product of the spectral displacement of the ESDOF, times the coordinate of the fundamental mode shape at the point considered, as $u(x,z)=\Phi(x,z)S_d(T)$, where T is the associated period of the structure when it vibrates free in the mode $\Phi(x,z)$ (obtained from generalized properties based on standard procedures, Clough and Penzien 1993).

Several analytical alternatives have been proposed for estimating the translational mode shape, $\Phi(x,z)$, the complexity of which is beyond the scope of the structures considered in the present paper (Vamvatsikos and Pantazopoulou 2010). A universal procedure opted for by most engineers is to conduct a computer analysis using a finite element idealization of the structure and classical numerical calculation of eigenvalues / eigenvectors. This approach is fraught with the difficulties detailed earlier in this section. Furthermore, because of the great uncertainty regarding the material properties and the degree of restraint provided at connections between different materials, results obtained ought to be considered primarily in a qualitative light. As a rule of thumb, assessment of TURM structures should be conducted on seismic response demand indices that are relatively insensitive to the accuracy of the estimation of period and damping, and should secure some conservatism in the process. Considerations adopted in the present study are as follows:

(a) Because TURM buildings rarely exceed 7-8 m in height, typically having flexible diaphragms and roof beams, they usually belong in the plateau range of the acceleration spectrum (i.e. their fundamental translational period does not exceed 0.4-0.5 sec) – a finding that is confirmed by detailed finite element studies. Thus, for the needs of practical assessment, demand for the entire class of these buildings is linked to the values at the end of the plateau region of the design spectrum (i.e., assuming $T=T_C$, where, Type I spectrum of EC8 is considered for representation of the seismic hazard with $a_g=0.16g$ and S=1.0 for the location). Thus, assessment is performed from Equations 1 for spectral acceleration S_e and spectral displacement $S_d(T)$ assumed to occur at the top of the building when it is considered to vibrate in lateral translation in the event of the design earthquake:

$$T_B \le T \le T_C : S_e = a_g \cdot S \cdot \eta \cdot 2{,}5 / q \qquad (1.a)$$

$$S_d(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \frac{T_C^2}{4\pi^2} \cdot \left(\frac{\mu}{q}\right) \approx 0.025 a_g \cdot 2{,}5 \cdot S \cdot \eta \cdot T_C^2 \cdot \left(\frac{q^2+1}{2q}\right) \qquad (1.b)$$

For T_C=0.5sec, and damping ratio ξ=5% ($\eta=\sqrt{\frac{10}{5+100\xi}}$ =1 for ξ=5%) the above set of expressions yield $S_e=0.4g=3.9m/s^2$, $S_d=0.025m$.

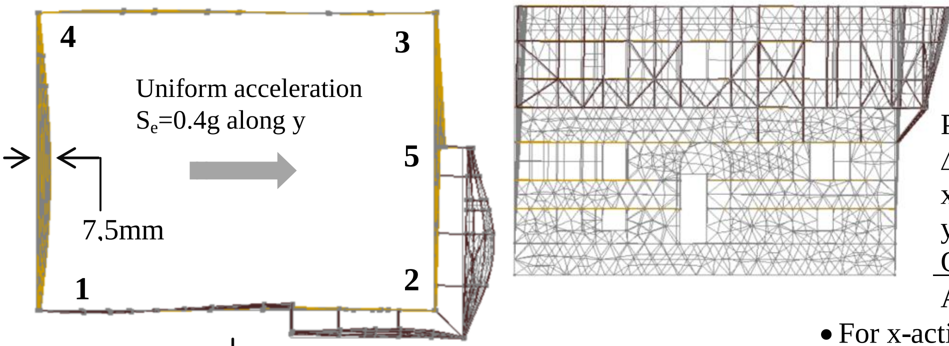
(b) In conventional modal analysis, S_e and S_d calculated in the preceding need be multiplied by the coefficient of excitation of the structural system considered (term Γ, see EC8-I 2005, Appendix B), the value of which depends on the shape of lateral translation assumed for the generalized ESDOF system; whereas for lumped systems this may be in the range of 1.2-1.3, in distributed systems this may well exceed that value of 2 depending on the assumed mode shape.

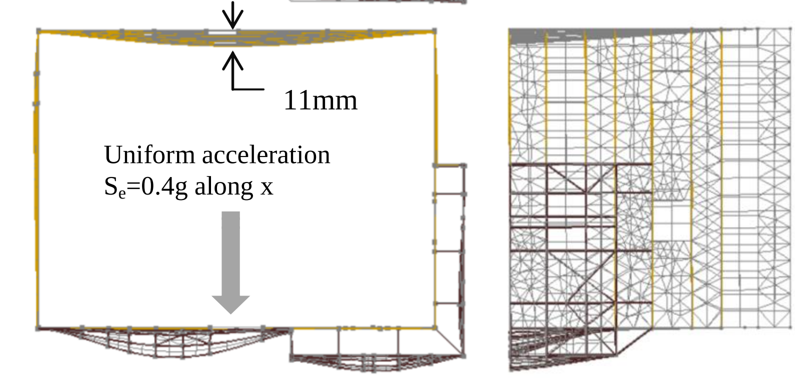
(c) For a distributed mass / distributed stiffness system, it was stated that most computer analysis programs produce a multitude of eigenvectors each engaging a small or even insignificant fraction of the total mass in dynamic excitation. To overcome this difficulty, the shape of the fundamental translational vibration, Φ(x,z) or Φ(y,z) may be best estimated from a pushover analysis rather than from the solution of the eigen-analysis; according with Clough and Penzien (1993), in estimating the fundamental translational mode of free vibration it is noted that the displacements result from the application of the inertia forces acting on mass, which in turn are proportional to the distributed mass and the associated accelerations, which are obtained as the second time derivative of displacement.

Thus, the fundamental translational shape Φ(x,z) (or Φ(y,z)) is the displacement profile resulting from the application of a static load p(x,z) (or p(y,z)), proportional to m(x,z)*Φ(x,z) (or m(y,z)*Φ(y,z), respectively). If the perimeter wall thickness is approximately constant, the distributed mass is proportional to the weight of the walls per unit area of wall surface. Results of sufficient accuracy, according with the iterative procedure of Rayleigh, will be obtained even if the applied distributed load is not exactly proportional to the exact mode. A common approach is to calculate the lateral deflection u(x,z) (or v(y,z)) for the structural system when it is loaded by its self-weight acting in the horizontal direction (as a first approximation); if a more refined result is sought, then the applied load could be the self-weight multiplied by the height coordinate z/H (where H the height of the structure).

The fundamental mode of translational vibration is then estimated by normalizing the displacement profile with respect the maximum displacement coordinate in the direction of interest (i.e., $\Phi(x,z)=u(x,z)/u_{max}$). It is interesting to note that when using this particular shape for calculating the response, mass participation is very high (above 90%) whereas the coefficient Γ converges to 1 in this case.

The results of the procedure outlined in the preceding are illustrated in Figure 5, which depicts the deformed shape of the structure when loaded with mass proportional loads in the x and y directions, respectively, and corresponding lateral displacement profile, for definition of the θ_{plan} and θ_v distributions for the estimated S_d value (22.03 mm at the most displaced point of the crest).





Peak roof displacements,
$\Delta_{max:}$ For action along:
x-dir: 22,07mm
y-dir: 17,65mm
Optional Safety factor:
Amplify results by S_d/Δ_{max}

- For x-action: 0,025/0,022 = 1,13
- For y-action: 0,025/0,0176 = 1,4

Peak drift values: Action in x:
θ_{plan}=(11-1,28)x10^{-3}/(11.3/2)=**0.17%**
θ_{sh}=1,61x10^{-3}/7,5=0,02%
θ_{fl}=11x10^{-3}/7,5=**0,14%**

Peak drift values: Action in y:
θ_{plan}=(7,5-1,46)x10^{-3}/(8,8/2)=0.13%
θ_{sh}=2x10^{-3}/7,5=**0,026%**
θ_{fl}=7,5x10^{-3}/7,5=0,1%

Figure 5: Deformed structure under uniform acceler. according to the proposed method. Critical is action in x

Evidently the procedure described provides a much more transparent picture of the state of the structure and identifies easily the tendency for localization of deformation (points of anticipated damage) for the purposes of assessment, whereas avoiding the implications of overestimation of demands caused by superposition of modal maxima. To illustrate this point, results obtained for displacements and moments from the two approaches, i.e. (complete quadratic combination of modal maxima, and the static procedure proposed) are listed in the following table for the vertical lines at the corners of the building's plan, 1, 2, 3, 4, 5, identified in Figure 5(a). Cells in bold identify points where the values of the moments are large and at the same time, critically different between the two methods. Note that the values of cracking moments (per unit width of wall strip) for the masonry walls range between the values of 7 kN-m/m in the upper floor to 16 kN-m/m at the base. Thus, cracking will evidently occur, and if q the ratio of peak moment to the above cracking strength values, estimated here in the range of q=3 for motion in the x-direction, it follows that the peak elastic drift values estimated in the note of Fig. 5 should be multiplied by a factor of $(3^2+1)/(2\cdot 3)=1{,}67$ (see Eq. 1(b)), thus, the corresponding peak drift values (for motion in the x-direction) are, θ_{plan}=0.17%·1,67 = 0,29%, θ_{sh}=0,02%·1,67=0,035% and θ_{fl}=0,14%·1,67=0,24%.

ACCEPTANCE CRITERIA IN TERMS OF DEFORMATION - CONCLUDING REMARKS

Deformation measures calculated above can be used to determine the performance level (characterization of damage level) attained by the structure in response to the design earthquake. Considering that cracking rotations (drift ratios) in masonry elements are in the order of 0.15%, the above values for drift demands correspond to ductility levels of two in the masonry walls (2≈0.29%/0.15%) – this demand level in within the ductility capacity of the timber laced masonry wall, illustrating the resilience and favourable earthquake response prescribed by the timber-laced mode of construction of these traditional buildings. The level of demand is much higher locally in the bay region, however, this part of the structure is particularly ductile and resilient due to the timber connections.

The procedure described for assessment of TURM-TL structures is a displacement-based method that eliminates the over-estimation of combination of modal maxima, enabling identification and quantification of the locations of potential damage in this class of historical structures.

Table 1: Summary of calculated results from the two methods: G+E_x represents values from static analysis for gravity loads and uniform lateral acceleration equal to the value at Spectral Plateau. Modal superposition represents the result of CQC on modal maxima.

			G+E_x / Modal Superposition				G+E_y / Modal Superposition			
edge	z (m)	node	u (mm)	v (mm)	M_x (kNm/m)	M_y (kNm/m)	u (mm)	v (mm)	M_x (kNm/m)	M_y (kNm/m)
1	7.5	157	1,19/0,7	-0,33/1.4	0,13/0,82	0,94/4,26	0,03/0,53	2,0/2,44	-0,07/1,3	-0,13/6,72
	5.8	108	1,03/0,5	-0,19/0,76	0,08/0,15	0,35/0.64	0,04/0,4	1,42/1,38	-0,03/0,13	-0,13/0,54
	4.3	107	0,87/0,44	-0,12/0,39	22,6/22,7	2,33/9,67	0,01/0,31	1,0/0,75	-37,7/44,4	-12,94/18,4
	2.6	99	0,62/0,3	-0,04/0,2	11,77/10,3	2,6/2,3	0/0,2	0,66/0,41	-16,4/16,5	-1,97/3,78
	1.1	59	0,36/0,16	-0,01/0,09	3,9/3,63	1,74/1,5	0/0,11	0,33/0,18	-5,85/5,25	-0,66/1,62
	0	385	0/0,06	0/0,03	-2,26/1,76	0,02/1,55	0/0,04	0,11/0,05	-1,18/1,87	-4,33/3,14
2	4.3	166	0,86/0,36	0,21/0,23	-0,03/0,67	0,02/0,4	0/0,3	0,82/0,49	0,06/0,68	0,04/0,34
	2.6	539	0,60/0,23	0,11/0,16	-1,45/0,97	-0,3/3,86	0/0,18	0,59/0,33	-0,13/1,18	1,12/4,1
	1.1	50	0,34/0,12	0,05/0,08	0,55/2,33	0,56/0,57	0,01/0,09	0,32/0,16	1,94/2,34	-0,63/0,84
	0	387	0/0,05	0/0,03	-2,85/1,5	0,21/0,5	0/0,04	0,12/0,06	-1,11/1,75	1,09/0,96
3	7.5	154	1,61/0,89	-0,21/0,62	33,5/38,87	1,7/4,66	-0,28/0,63	1,45/0,81	-20,8/26,23	-0,9/3,28
	5.8	886	1,31/0,64	-0,18/0,48	32,7/27,26	6,17/5,3	-0,19/0,45	1,27/0,66	-15,95/16,5	-2,17/3,44
	4.3	116	1,0/0,44	-0,13/0,38	**33,43/24,2**	7,32/5,4	-0,11/0,32	1,08/0,54	-14,80/13,5	-4,0/3,32
	2.6	858	0,65/0,26	-0,08/0,25	-20,7/14,7	-4,06/2,2	-0,05/0,19	0,78/0,37	9,08/9	0,40/1,95
	1.1	47	0,34/0,12	-0,04/0,13	7,4/5,67	2,39/1,94	-0,02/01,0	0,42/0,2	-3,33/4,02	-2,13/1,44
	0	388	0/0,05	0/0,05	-1,38/2,6	1,57/1,46	-0,01/0,04	0,15/0,07	0,17/1,97	-0,15/0,96
4	7.5	147	1,28/0,74	0,02/0,52	31,65/33,13	3,29/4,8	0,13/0,51	1,46/0,87	**25,95/52,65**	1,55/5,25
	5.8	29	1,12/0,6	0,02/0,44	25,87/20,7	5,16/4,2	0,12/0,43	1,27/0,71	**21,21/34,38**	3,41/7,06
	4.3	109	0,93/0,46	0,01/0,36	-27,15/19	-5,83/4,24	0,07/0,33	0,89/0,57	**-15,21/31,0**	-2,37/4,93
	2.6	876	0,64/0,3	0,01/0,25	-14,81/10	-2,72/1,73	0,06/0,21	0,76/0,37	-11,72/14,2	-0,78/2,88
	1.1	40	0,32/0,14	0,0/0,13	-5,76/4	-2,22/1,59	0,03/0,1	0,42/0,19	-5,69/5,91	-1,36/1,58
	0	389	0/0,05	0/0,05	-1,68/1,97	-2,25/1,1	0,01/0,04	0,16/0,07	-1,16/2,23	1,64/1,03
5	7.5	158	1,57/0,84	1,19/8,26	0/0	0/0	-0,27/0,6	6,09/6,7	0/0	0/0
	5.8	326	1,26/0,61	0,53/5,65	0,44/1,0	0,17/6,83	-0,17/0,44	4,64/4,5	-0,52/0,61	4,84/5,2
	4.3	121	0,86/0,36	0,03/3,34	**-33,4/76,3**	**-34,5/52,9**	0/0,3	3,11/2,6	**40,27/46,68**	**-24,4/47,26**
	2.6	1262	0,60/0,24	-0,02/1,70	3,98/12,71	1,7/11,87	-0,02/0,18	1,87/1,4	-8,34/8,4	-2,46/10,84
	1.1	1190	0,32/0,12	-0,02/0,7	1,38/3,14	-0,34/8,56	0/0,09	0,88/0,6	-1,76/2,24	3,32/6,7
	0	1165	0/0,05	0/0,19	0.46/1,51	-0.25/13,8	0/0,04	0,26/0,17	2,17/1,47	15,28/11,6

REFERENCES

1. Vitruvius, On Architecture, pp.147-175 Vol. I, Books I-V, Athens, Plethron, 1996.
2. Langenbach R (2002), "Survivors in the midst of devastation, Traditional Timber and Masonry Construction in Seismic Areas", *Conference Proceedings, 7th US National Conference on Earthquake Engineering*, Boston, Massachusetts, July 21-25.
3. Wolfgang Müller-Wiener (1995), *Architecture in Ancient Greece*, *pp.*70-101, University Studio Press, Thessaloniki.
4. CEN. EN 1998-1 Eurocode 8: Design of Structures for Earthquake Resistance-Part 1: General Rules, Seismic Actions and Rules for Buildings, *Europ. Comm. for Standardization*, Brussels, 2004
5. CEN. EN 1996-1-1 Eurocode 6: Design of Masonry Structures - Part 1-1: General Rules for Reinforced and Unreinf. Masonry Structures, *Europ. Comm. for Standardization*, Brussels, 2005.
6. CEN. EN 1998-3 Eurocode 8: Design of Structures for Earthquake Resistance - Part 3: Assessment and retrofitting of Buildings, *European Committee for Standardization*, Brussels, 2005
7. ACORD-expert http://www.itech-bois.com/fr/Telechargement/log/ACORD-Expert/ACORD-Expert.htm
8. Vamvatsikos, D. and S. J. Pantazopoulou, "Simplified Vulnerability Assessment of Historical City Cores – the Example of the Old City of Xanthi", *CD-ROM Proceedings*, 9th U.S. and 10th Canadian Conference on Earthquake Engineering, Toronto, Canada, July 2010.
9. Papadopoulos, M., "Seismic Assessment of Traditional Houses in the Balkans – Case Studies in Xanthi", *J. of Civil Engineering and Science*, (in print), http://www.ij-ces.org/, 2013.