CRITERIA GUIDING SEISMIC UPGRADING OF TRADITIONAL MASONRY BUILDINGS

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ABSTRACT
The paper establishes a framework for seismic assessment and retrofit of traditional masonry structures with emphasis on simple tools that may be used in preliminary identification of those buildings that are at greater risk for earthquake damage due to inherent pathologies. Readily accessible indices such as the eccentricity of gravity load required to resist the overturning moment are used to evaluate the maximum tolerable ground acceleration prior to excessive damage. Local criteria are used to assess the intensity of out of plane differential translation and in plane shear distortion of masonry walls oriented orthogonal to and parallel with the seismic action, respectively. Retrofit through pertinent interventions aims at the reduction of the deformation components as they both quantify the likelihood, extent and localization of damage. The effectiveness of the intervention scenarios is evaluated through the improvement of resulting performance, using the derived relations between intervention morphology and anticipated damage. This framework is particularly useful for setting retrofit priorities and for management of the collective seismic risk of historical settlement entities. A typical Balkan-type 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 Siatista, Greece.

KEYWORDS: plain masonry, seismic behavior, strengthening, traditional buildings

INTRODUCTION
Traditional unreinforced masonry (TURM) buildings have common structural characteristics in the greater geographical region of the Balkans, but within the narrow confines of a township they also have common architectural features that emanate from the special needs of the local community, the local masonry trades practices and local material supplies (stone quarries, limestone availability). As a result it is often seen in Greek towns that traditional houses which comprise unreinforced load bearing masonry are practically identical in terms of structural system with small variations in dimensions as required by the terrain and the land plot.

In light of the fact that TURM buildings are generally considered highly vulnerable to earthquake effects, there is an urgent need for a quick assessment method by which to identify those examples that are more likely to collapse in a future earthquake having the spectral characteristics and intensity of the ground motion used in practical seismic design. The method should be low cost, immediate, and palatable to structural engineering practitioners.

This is the objective of the present paper. For this purpose a sample structure is used as reference, in order to define the average details of a traditional town building population. The township used as a model for this study is Siatista, a historical town in the prefecture of Kozani in Northern Greece, defining the characteristics of the typical TURM of this region. The sample structure is used to assess the relevance of the proposed simple-assessment procedure through comparison with results from detailed finite element simulations, obtained both for the initial condition of the structure (insignificant diaphragm action both at the floor levels and at the roof), but also after implementation of the most commonly used rehabilitation schemes, namely (a) addition of a ring-beam at the roof level and (b) enhanced diaphragm action at the floor levels and at the roof.

PREREQUISITES FOR APPLICATION OF THE METHODOLOGY

The proposed approximate method is applied to buildings of unreinforced load bearing masonry that satisfy a number of necessary conditions, imposed by the need to increase the methods' reliability, and to avoid its being used in special cases where a more detailed analysis ought to be pursued. These conditions are as follows:

a)The building should have a fundamental natural translational period T_1, in both principal directions of the floor plan, that falls below the range: $\{4T_c,\ 2.0\}$ *sec*, where T_c is the characteristic "transitional" period that marks the end of the plateau of the design acceleration spectrum of EN 1998-1 [1].

b) Additionally, it is required that:

- The building is approximately rectangular (so that torsional effects may be considered negligible).
- The walls are continuous heightwise
- Floors on opposite sides of a single wall are at the same height.

The fundamental translational period of the building, T_1, is approximated in each of the principal directions of the plan as follows:

$$T_1 = C_t \cdot H^{3/4} \ \text{(sec)} \tag{1}$$

In Eq. (1), H is the total building height, in m, measured from the level of foundation or the level of rigid basement and C_t a constant given by the relationship:

$$C_t = 0,075 / \sqrt{A_c} \leq 0,05 \tag{2}$$

The upper level in Eq. (2) refers to buildings with flexible diaphragms.

A_c depends on the total area of load-bearing walls in the ground floor of the building (in m^2) and is calculated from:

$$A_c = \Sigma\left[A_i \cdot \left(0,2 + \left(l_{wi} / H\right)\right)^2\right] \tag{3}$$

A_i is the active cross section of the i-th wall in the direction of seismic action considered in m^2

l_{wi} is the length of the i-th wall in the direction of seismic action, in m.

DETERMINING THE BASE SHEAR FORCE

Type I earthquake spectra as prescribed by EN1998-1[1] is used to determine the acceleration at the top of the structure in the fundamental mode of vibration, $S_e(T_1)$ (using the value of peak ground acceleration, a_g, for the site in consideration given from seismicity maps, and the soil type coefficient, S); in light of the fact that for the usual period range of the typical TURM structure (seldom higher than two storey, relatively stocky structures) the fundamental period falls in the constant acceleration range of the spectrum, it is conservative to estimate $S_e(T_1)$ from:

$$S_e(T_1) = 2.5 \cdot a_g \cdot S \cdot \eta \text{ m/sec}^2 \tag{4}$$

The soil coefficient S is taken as 1.0 for stiff or rocky soil and is increased to 1.2 for more compliant soil conditions, whereas multiplier η depends of the viscous damping ratio, ξ: $\eta=[10/(5+100\xi)]^{1/2}\geq 0.55$. Therefore, the peak inertia force acting on the structure during the earthquake, and thus the base shear, V_o, may be obtained from the product:

$$V_o = C_1 C_m S_e(T_1) \cdot W / g \quad \text{(KN)} \tag{5}$$

In Eq. (5), C_1 is the inelastic amplification ($\geq$1) that relates the anticipated peak inelastic displacement of the structure to the elastic value of the design acceleration obtained from the spectrum and C_m is the mass participation coefficient which is taken equal to 1 for one and two storey houses and 0.8 for higher structures.

The lateral forces amounting to the base shear value given by Eq. (5) are distributed height-wise in the structure assuming a linear distribution of lateral forces since mass is no longer lumped at the floor levels particularly in the case of buildings with flexible diaphragms. Thus, at a distance z from the base of the structure, the value of the external force is obtained from: $v(z) = v(z|_H)\cdot(z/H)$, with $v(z|_H)=v(H)=2V_o/H$.

OVERTURNING MOMENTS

In this simplified analysis for quick assessment the building is treated as a cantilever structure, having a cross section defined by the plan of the structure – this, neglecting the occasional position of openings in the overall scheme, the building resembles a hollow tube having a wall thickness t equal to the actual thickness of the perimeter walls of the building; the lateral forces acting on the structure cause a flexural moment and shear force on the cross-section at a distance z from the base of the structure, which is calculated from the equilibrium requirements as:

$$V(z) = (H - z)[v(z) + v(H)] / 2 \tag{6}$$

$$M(z) = v(z)(H - z)^2 / 6 + v(H)(H - z)^2 / 3 \tag{7}$$

The most critical level is the ground level $(z=0)$; Equations (6) and (7) yield the base shear V_o and overturning moment M_o of the building:

$$V_o = H \cdot v(H) / 2 \tag{8}$$

$$M_o = v(H)H^2 / 3 = V_o \times \frac{2}{3}H \tag{9}$$

To quickly evaluate the building against the implications of the stress resultants defined above the plane sections assumption is invoked for the idealized cantilever in order to determine normal and shear stresses through the wall thickness of the building. Considering the two ends of the plan in the dimension parallel to the ground motion, P_1 and P_2, A_w the cross sectional area of the walls in the plan, and Ω_w the flexural modulus of the buildings' cross section (i.e., plan), normal stresses are calculated at the two extremes of the building's plan, as follows:

$$\sigma_{P1} = -\frac{W}{A_w} + \frac{M_o}{\Omega_w} \quad \& \quad \sigma_{P2} = -\frac{W}{A_w} - \frac{M_o}{\Omega_w} \tag{10}$$

In the above, $W(z=0)$ is the overbearing self-weight of the structure, at the level considered (ground level). From the normal stress distribution it is possible to determine the position of the neutral axis in the building's plan. If the axial compression load due to overbearing pressure is very large, the neutral axis will be located outside of the building's plan, and thus the entire cross section of the building will be active in carrying load, including the shear stresses required to equilibrate V_o. If on the other hand, the neutral axis is located within the boundaries of the structure's plan at ground level, then the part of the wall that is estimated to be subjected in direct tension will be considered inactive in resisting shear; actually in that situation a criterion limiting the magnitude of the earthquake that may be tolerated by the building without local failure will be related to the magnitude of the nominal tensile strength of masonry as follows:

$$\sigma_{P1} = f_{tm} = W\left(-\frac{1}{A_w} + C_1 C_m \frac{S_e(T_1)}{\Omega_w g} \times \frac{2H}{3}\right) = W\left(-\frac{1}{A_w} + C_1 C_m \, a_g \frac{2.5\eta S}{\Omega_w g} \times \frac{2H}{3}\right) \tag{11}$$

Thus, the ground earthquake acceleration level beyond which there will be inactive regions due to direct tension at the base of the structure is defined by,

$$a_g \leq 0.6 \frac{\Omega_w g}{C_1 C_m \eta S H}\left[\frac{f_{tm}}{W} + \frac{1}{A_w}\right] \tag{12}$$

A more austere check can be accomplished by examining the eccentricity of the load combination defined by (M_o, W), where $e=M_o/W$ and setting limits so as to eliminate tension $(\sigma_{P1}=0)$. Thus, for a structure with a rectangular plan of dimensions ℓ_x and ℓ_y and a wall thickness t, to eliminate tension at the wall base for earthquake parallel to the x direction requires that $e \leq e_{lim}$:

$$-\frac{W}{A_w} + \frac{M_o}{\Omega_w} = 0 \Rightarrow \frac{W}{A_w} = \frac{M_o}{\Omega_w} \Rightarrow e_{\lim} = \frac{M_o}{W} = \frac{\Omega_w}{A_w} \approx \frac{1+3\left(\frac{l_y}{l_x} - 2\frac{t}{l_x}\right)\left(1-\frac{t}{l_x}\right)^2}{6\left(1-\frac{t}{l_x}\right)\left(1+\frac{l_y}{l_x} - 2\frac{t}{l_x}\right)} l_x \tag{13}$$

For usual wall thickness values and plan aspect ratio values (ℓ_y/ℓ_x) Eq. (13) yields the values of Table 1.

Table 1: Calculated limits for e_{lim}/ℓ_x

ℓ_y/ℓ_x \ t/ℓ_x	0.05	0.1	0.15	0.2
0.5	0.261	0.246	0.234	0.226
1	0.317	0.303	0.290	0.280
2	0.372	0.355	0.340	0.326

EVALUATION OF BASE SHEAR STRENGTH

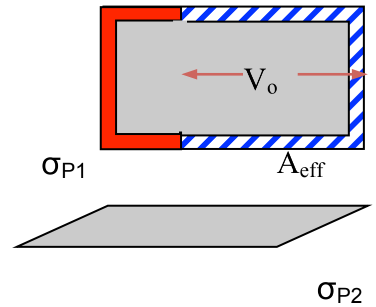


Figure 1: Definition of A_{eff} in plan

Only the active area of the wall cross section, $A_{w,eff}$, is assumed to support the applied shear force, V_o (Fig. 1). Thus, if no diaphragm action is present, only walls parallel to the seismic action are assumed to participate in shear resistance according with their area. In this calculation only the part of the wall cross section is considered where normal stresses resulting from the combination of self-weight and overturning moments are compressive. Walls orthogonal to the direction of action also participate only if (a) they are within the active area of the plan cross section and (b) the diaphragm at the level above the one considered is rigid, so as to ensure equal lateral displacements for all points in its perimeter. Acceptance criteria is that shear demand should not exceed the shear strength of unreinforced masonry, which is estimated according with a Mohr-Coulomb type frictional law:

$$f_v = f_{v0} + 0.4\sigma_d \leq 0.065 f_b \tag{14}$$

where, f_{v0} is the mean value of cohesion (here it was taken $f_{vo}=0.20$ MPa), σ_d is the normal compressive stress owing to overbearing loads only; f_b is the mean compressive strength of the masonry units. These values should be obtained from field tests if such are available; otherwise, code-prescribed characteristic values should be used instead.

Therefore, $$\tau_o = \varepsilon \frac{V_o}{A_{w,eff}} \leq f_v \tag{15}$$

Coefficient ε is an amplification factor which accounts for the presence of openings in the critical section within the ground floor; thus ε is the ratio of $A_{w,eff}$ to the minimum effective wall area, $A_{w,eff}^{min}$, in the critical storey. Similar checks can be made with reference to the storey shear at higher floors of the structure. This check is expected to be critical in tall, slender TURM structures with several openings or in structures with stiff diaphragms.

OUT OF PLANE ACTION

TURM structures with flexible diaphragms are much more vulnerable to failure due to out of plane action, i.e., when responding to ground motions that occur normal to their orientation. For such structures, the walls bend out of plane under the influence of normal pressure the magnitude of which is, $p(z) = (z/H)\cdot S_e(T_1)\cdot t\cdot w/g$ (kN/m^2) acting throughout the wall surface from the base to the roof level where w the self-weight per unit volume of the walls (in kN/m^3).

An approximate solution of the state of stress of the walls under the $p(z)$ pressure may be obtained if they are treated as vertical plates in the structural model, using simple analysis procedures such as the method of strips. This approach is more consistent with the observed field performance of such structures, even in the presence of openings, as compared to procedures that subdivide the plate to smaller homogenous sub-plates [2], because in this manner the global boundary conditions of the wall are allowed to play a determining role in the manner by which the system responds; openings may be accounted for by proper amplification of the calculated moments in the strips that contain them.

To simplify the analysis, the walls may be considered fixed or partially restrained to rotation along the vertical boundaries at the corners of the building and at the base (the degree of fixity between orthogonal walls may be considered a parameter of epistemic uncertainty for this problem, in the event of fragility type analysis of this class of structures). The boundary at the roof level is unrestrained in the absence of a continuous confining ring beam tying the perimeter of the roof, or the wall may be considered simply supported at that level (unrestrained against rotation) if a ring beam exists or is added during retrofit. As in the case of plates the direction of load transfer is determined from tributary areas, depending on the boundary conditions on the perimeter of the wall. Examples of this type of distribution and the ensuing loading pattern of the associated strips are depicted in Fig.2 for several alternative examples. Note that the static model for strips oriented along z for an unrestrained top boundary lies between the cantilever and the fixed-simply supported case; it is fair to assume that z-strips are closer to the cantilever example if $L_y>L_z$, the reverse if $L_y<L_z$. (In case of densely spaced openings as seen in neoclassical buildings of the European urban centers of the 19th century, the walls in out of plane action could be alternatively solved when assuming the yield line formation which would lead the wall-plate to failure). The degree of fixity at the ends of strips 1 and 3 is reflected through λ: a value of $\lambda=0$ implies no restraint to rotation, $\lambda=0.7$ corresponds to noncompliant fixed supports, $\lambda<0.5$ refers to partial restraint against rotation. $\bar{g}3$ is an equivalent uniform load for the load case of strip 3.

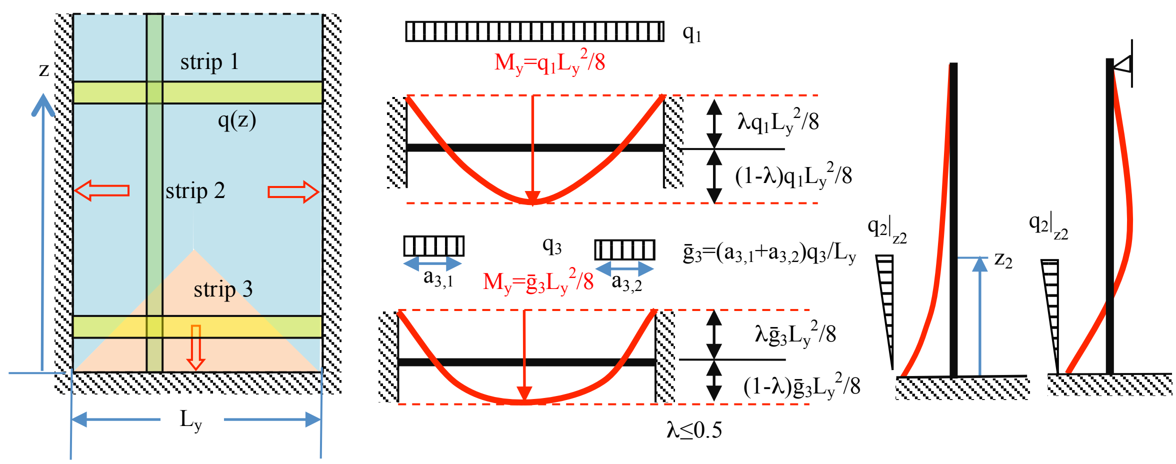


Figure 2: Influence Areas and Application of the Strip Method for Estimation of the State of Stress in walls transverse to the Earthquake Action

Acceptance criteria for this problem are related to the moments M_y and M_z (the subscript corresponds to the orientation of the strip considered) which are compared with the cracking moments of the wall; the peak ground acceleration tolerated prior to cracking may be estimated by setting the peak moment equal to the cracking strength. Using the same concept, higher values of peak ground acceleration that may lead to cracking over prescribed fractions of the total wall area may be determined in the process of assessment. By the same token, modifications to the boundary conditions of the walls, as possible rehabilitation measures, may be gauged by the amount of peak tolerable ground acceleration increase they may be able to secure for the building, enabling the structure to sustain a higher magnitude design earthquake through these measures. The cracking moment is estimated from:

$$M_{z,cr} = \left(f_{x1} + w\cdot(H-z)\right)\cdot\frac{t^2}{6} \quad (16)$$

and $$M_{y,cr} = f_{x2}\cdot\frac{t^2}{6} \quad (17)$$

where f_{x1} and f_{x2} is the flexural strength for failure plane parallel and normal to joints, respectively.

At any level, the moments in the z-strips should be amplified locally by the ratio L_y/ℓ_y, where ℓ_y is the dimension of the plan minus the breadth of the openings. If the strength of the orthogonal walls suffices to support a lower pressure, $p'(z)<p(z)$ than what would be estimated for the applied design value of a_g, then it follows that the assessed base shear strength of the building should be scaled down to the reduced ground acceleration thus estimated.

NUMERICAL APPLICATION

The simplified procedure described in the preceding is meant to be used in rapid seismic assessment of TURM structures, so as to determine whether the structure considered can withstand the design earthquake, or alternatively, to determine the magnitude of the ground acceleration that may be sustained without failure. To demonstrate the relevance of the method with more detailed calculations, a typical building was analysed using a 3-D finite element model, comprising shell elements for the wall elements of the structure and truss members for the connecting components, the diaphragm and roof elements using the program code ACCORD-CP [3]. Results were correlated with those obtained from the practical procedures described. Comparisons are done along the sections 1-1 and 2-2 in wall T1, and along the sections 3-3 and 4-4 in wall T2. The necessary input data for application of the simplified assessment methods are listed in Table 2.

The ground motion was prescribed using the EC8-I design spectrum with peak ground acceleration of 0.16g (in the constant acceleration range the design value of the acceleration at the top of the structure for $S=1$ and $\eta=1$ is, $S_e(T_1)=0.40g$) along each of the principal directions of the building in combination with the self weight of walls, diaphragms and roof.

A separate additional comparison is made between the values for the out of plane action estimated according with the proposed simplified procedure and those obtained from finite element analysis of the transverse wall. The wall was solved assuming fixed boundaries along the vertical edges and the base, and free edge at the top, under normal pressure acting over its surface.

The normal pressure was varied linearly along the height of the building from zero value at the base, to a peak pressure at roof level (at $z=H$) obtained from:

$$p\left(z=H\right) = 2.5\times 0.16\times S\left(=1\right)\times\eta(=1\ for\ \xi=5\%)\times t\left(0.6m\right)\times 22kN/m^3 = 5.28\ kN/m^2 \quad (18)$$

Masonry was modeled using $E=0.50E_{el}$ where E_{el} the modulus of elasticity of the uncracked material state which was estimated from the relationship $E_{el}=1000f_w$, where $f_w=3.5MPa$ the mean compressive strength of masonry.

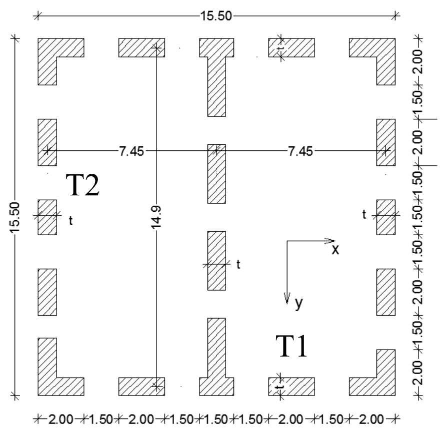


Figure 3: Plan View of the Structure

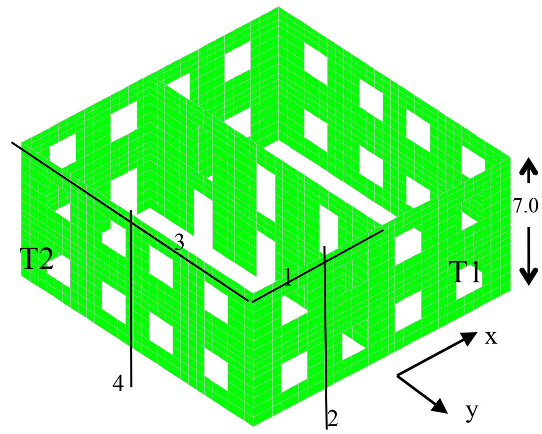


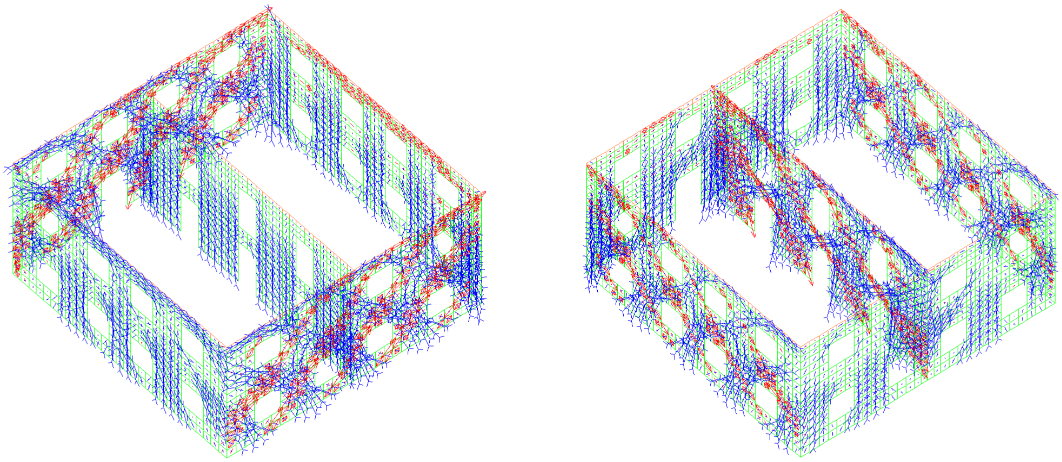
Figure 4: FE model and reference sections used for comparison with Simple Analysis

Table 2: Parameter Values Used in Numerical Example

	symbol	value	unit	source
plan dimensions	L1, L2	15.5, 15.5	m	input data
Height	H	7.0	m	input data
Wall surface area in the plan	Ax, Ay	18.6, 27,9	m^2	calculated
Flexural Resistance	Ωx, Ωy	216.29, 192.3	m^3	calculated
Effective cross section	A_{cx}, A_{cy}	2.83, 3.61	m^2	Eq. 3
constant	C_{tx}, C_{ty}	0.044, 0.039		Eq. 2
Fundamental period	T_{1x}, T_{1y}	0.191, 0.169	sec	Eq. 1
weight	W	6916.54	KN	calculated
Ground acceleration	a_g	0.16g	m/sec^2	input data
Design spectral acceleration	$Se(T_1)$	0.40g	m/sec^2	Input data
Weight per unit volume	w	22	KN/m^3	input data
Base shear	V_o	2856.27	KN	Eq. 5
Overturning moment	M_o	13329.26	KN	Eq. 9
Stresses along x	σ_{P1}, σ_{P2}	-84.25, -222.87	KN/m^2	Eq. 10
Stresses along y	σ_{P1}, σ_{P2}	-91.94, -215.19	KN/m^2	Eq. 10
Incremental factor for shear	ε_x, ε_y	1.55, 1.43		calculated
Mean shear stress	τ_{ox}, τ_{oy}	0.24, 0.14	MPa	Eq. 15
Uniform load	q_1, q_2,	5.28, 2.81	KN/m	Fig. 2
Uniform load	q_3, $\bar{q}_3$, q_4	5.28,4.96,5.28	KN/m	Fig. 2

IN PLANE ACTION

Figure 5 depicts the distribution of axial forces obtained from the seismic combinations $G+E_x$ and $G+E_y$ calculated at the wall thickness midpoint, where G denotes the gravity loads and E_x, E_y the seismic action along x and y axes, respectively. In the example considered it is evident that normal compression is acting in all the perimeter walls at the base of the structure, therefore the entire wall cross section parallel to the earthquake action is actively engaged in shear resistance. This is consistent with the results of the approximate solution where it was found that shear stress demand in all cases was less than the value obtained from Eq. (14), not exceeding the value of 0.26 MPa.



a) b)

Figure 5: Axial Forces N for Load Combination: a) G+Ex and b) G+Ey

OUT OF PLANE RESPONSE

For the out of plane action moments were compared for the locations in the structure corresponding to strips 1-4 in Fig. 4. Table 3 presents peak values for flexural stress resultants (moments M_y, M_z per unit width of strip) obtained from: (a) detailed F.E. analysis of the entire structure (spectral analysis), (b) F.E. analysis of the transverse wall under normal pressure that varies linearly heightwise simulating earthquake effects, (c) the proposed simplified assessment method (for earthquake action along y, it was assumed that λ=0.40 and λ=0.60 for the edge and intermediate supports of the transverse wall, to account for the different rotational restraint). All analysis models considered identify consistently as being most critical the case of earthquake action along the x axis (longer unsupported wall in out-of-plane action). The peak flexural moment occurring at the base of the longer wall bending out of plane is M_z =86.24 KNm; this is compared with the cracking moment $M_{z,cr}$ =12.24KNm, from Eq. (16). Similarly, the maximum midspan moment at roof level, M_y=68.84 is compared with flexural strength $M_{y,cr}$ = 6KNm from Eq. (17).

Table 3. Bending Moments

	Ground motion along Y					Ground motion along X				
	Line 1			Line 2		Line 3			Line 4	
model	left	middle	right	Bottom z=0.0	Middle z=3.5	left	middle	right	Bottom z=0.0	Middle z=3.5
FE, acc	13.65	-18.56	38.99	16.32	-9.64	56.24	-39.4	56.39	62.11	-13.83
FE, pres	12.07	-7.22	13.58	2.85	-3.72	42.59	-19.14	42.57	23.24	-6.96
Simplified	5.95	-22.35	8.92	12.99	0	68.84	-68.84	68.84	86.24	-26.95

Evidently out of plane action is the controlling mode leading to failure at a much lower level of earthquake action as compared with the design ground acceleration of 0.16g. The peak spectral acceleration, $S_{ult}(T_1)$ that may be supported by the building without any form of cracking may be estimated from the proposed methodology by setting the critical moment value $M_{y,cr}$ equal with that produced by the limiting acceleration value. From solution of strips no. 3 and 4 the limiting value of acceleration was estimated at 0.037g. This acceleration value, $S_{ult}(T_1)$, is a limit for the so called performance level 1 in the established framework of EN 1998-3 [4], "Continued Operation". The extent of tolerable damage in the transverse walls, which could be associated to more damage-tolerant performance levels, such as "Reparable Damage" and "Collapse Prevention" ought to be defined with reference to the fact that some degree of ductility is imparted by the membrane forces developed in the masonry walls due to their thickness, which give rise to residual flexural strength in the cracked walls.

STRENGTHENING

The procedure described was also used to assess the effectiveness of two of the usual rehabilitation schemes used frequently with TURM structures, namely: (a) construction of a ring or tie perimeter beam at roof level, (b) addition of a reinforced concrete plate at roof level to secure diaphragm action. Both techniques are easy to implement, almost concealable in the final project and low cost. Note that the addition of diaphragm plates in intermediate floors is less advisable as it is generally more costly, presents technical difficulties and has little effect in mitigating the out of plane action of the top floor. Implementation procedures are as follows:

Addition of tie beam

Construction of tie beams contributes greatly to the seismic strength of a TURM building. Previous studies have shown than construction of tie beams at the roof level of such structures may reduce by as much as 50% the intensity at the critical upper level against out of plane action, even after removal of internal bearing walls. In the proposed method addition of a tie beam is reflected by the addition of a support at the top end of the vertical strips, which is important for the longer walls that did not satisfy this criterion based on aspect ratio alone (see Section 6). Furthermore, the influence areas are modified to the more favorable distribution shown in Fig. 6 with the critical horizontal stip being located now further down at *z=4.45m*. Peak flexural moments are reduced to $M_{y3}=43.70\ KNm$ both at mid-span and at the edges whereas, $M_{z4}=2.15KMm$ at $z=0$ and $maxM_{z4}=10.96\ KNm$ at $z=4.70m$. Again critical is the horizontal strip (#3) from which it is shown that peak tolerable spectral acceleration without cracking (Performance level 1) is *0.055g* corresponding to 48% increase in the strength of the building, consistent with the results from F.E. analysis of traditional unreinforced masonry buildings after rehabilitation with this procedure [5].

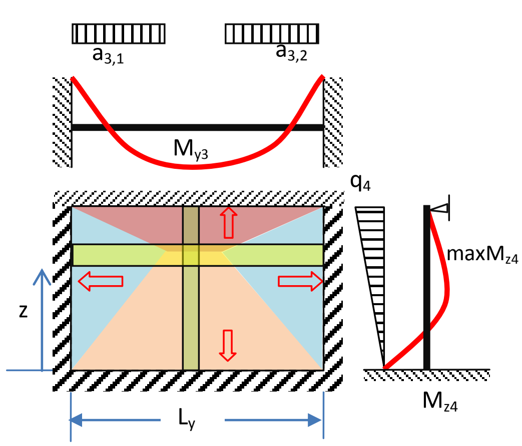


Figure 6: Tributary areas and Strip Boundary Cond.: Case with tie-beams

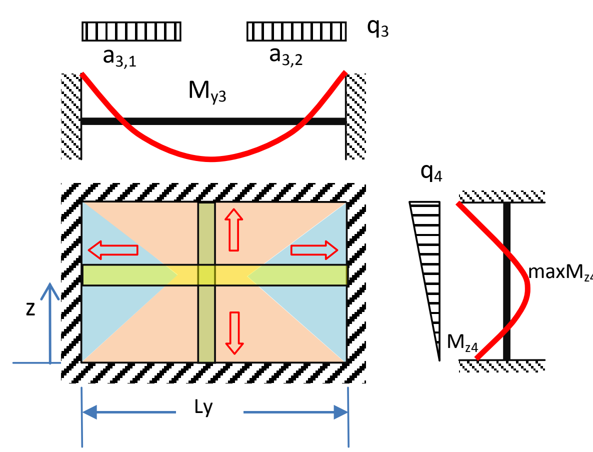


Figure 7: Tributary areas and Strip Boundary Cond.: Case with Rigid Roof

Addition of Inextensible Diaphragm at Roof Level

Prerequisite for this intervention is the favorable layout of the load bearing walls at the top floor level. In the example structure it is assumed that two continuous plates each having a theoretical span of 7.45 m (Fig. 3) will be constructed. The boundary conditions of the standing walls examined in the preceding will now be re-established to fixed supports on the entire perimeter. The layout of the influence zones and corresponding location of the critical strips are illustrated in the Figure 7. Flexural moments in strip 3 at $z=3.5m$ are $M_{y3}=34.42\ KNm$ both at mid-span and at the ends, whereas the corresponding values in strip 4 are $4\ M_{z4}=2.15KMm$ at $z=0$ and $maxM_{z4}=12.93\ KNm$ at $z=7.0m$. From the critical strip #3 it follows that the peak spectral acceleration associated with Performance level 1 (onset of first cracking) is $0.07g$, therefore, the building strength is increased by twofold over its initial condition (i.e., 100% strength increase).

CONCLUSIONS

Objective of this paper was to develop and proof test, through comparison with detailed finite element analysis, a simple, easy to implement, rapid assessment procedure for traditional, unreinforced masonry buildings, which populate the historical regions of old towns and cities in the Mediterranean region. The typical TURM structure considered is built of stone or clay-brick masonry, and despite the architectural details that vary throughout the major region in consideration, it typically has flexible diaphragms and inadequately-tied roof perimeter beams. These structures are particularly vulnerable to seismic effects particularly in light of the large mass of the vertical walls, and the controlling mode of failure is out of plane bending. The proposed procedure includes criteria derived from basic principles to limit the risk of damage due to tension in the walls parallel to the earthquake, as well as the moments supported in out of plane bending of the vulnerable walls. Performance of the method is correlated against Finite Element results from the analysis of the structure as a whole, but also from separate F.E. study of the critical walls in out of plane loading. It is shown that the proposed methodology consistently estimates the mode of failure in the original structure, but also after rehabilitation, underscoring the salient characteristics of TURM buildings that may cause their demise in an earthquake, and how by changing these characteristics an improved performance may be anticipated, as quantified herein by the increased level of ground acceleration that the structure may support with no cracking.

REFFERENCES

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