DISPLACEMENT-BASED EARTHQUAKE LOSS ASSESSMENT OF TURKISH MASONRY STRUCTURES

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ABSTRACT :

Masonry structures constitute a high percentage of the European building stock. However, the inherent variability of the material properties and other epistemic uncertainties of existing masonry buildings render the assessment of both single buildings and classes of buildings rather more complicated, as compared to their reinforced concrete counterparts. Nonlinear techniques for the assessment of masonry structures are still under development. A simplified nonlinear method (DBELA) is described herein that defines the vulnerability of a masonry building class by relating its deformation potential at different limit states and comparing this with the displacement demand from an over-damped displacement response spectrum at the period of vibration of the structure. In order to calibrate the methodology, nonlinear static and dynamic analyses of masonry structures have been run to obtain the displacement limits and fundamental periods of different types of masonry structures of varying heights. Experimental test results on Turkish masonry walls have also been collected in order to further calibrate the methodology.

KEYWORDS: Turkish masonry buildings, loss assessment, displacement based

1. INTRODUCTION

The seismic assessment of the existing building stock is of increasing importance nowadays due to the fact that there are many large cities in regions of high seismic hazard with hundreds of thousands of buildings with unknown structural properties. The percentage of these buildings constructed in masonry will depend on the traditions of the city or country. It is well known that masonry elements and structures attain specified performance levels, in terms of sustained damage, at lower levels of interstorey drifts as compared to reinforced concrete (RC) structures; on the other hand, it is evident that the fundamental periods of masonry structures are naturally lower than those of RC buildings and consequently, the displacement demand will also be comparatively smaller (Priestley *et al.*, 2007). Hence, the seismic risk of these structures may not necessarily be higher than that of their reinforced concrete counterparts. The risk assessment of masonry buildings, individually or on an urban-scale, requires an accurate definition of the material properties and other structural features. The inherent variety of the material used, the dimensions, the loading and boundary conditions render the assessment of both single buildings and classes of buildings rather complicated. Furthermore, different materials may be used in different parts of the structure (e.g. stone and much thicker outer walls may be used externally for insulation concerns, and clay brickwork may be employed for internal walls). Additionally, masonry buildings are often not treated as engineered structures in many parts of the world as code provisions may not include rules for the seismic design of such buildings. In Turkey, only the most recent earthquake code (of 2007) provides a set of engineering rules for the design of unreinforced masonry (URM) buildings. Hence, in general, one cannot really examine design codes in order to better understand the structural properties of existing URM buildings.

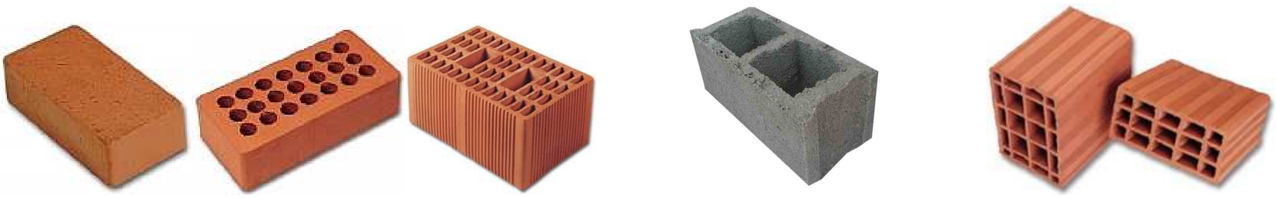
2. URM BUILDINGS IN THE NORTHERN MARMARA REGION, TURKEY

The use of masonry bearing walls has traditionally formed a large part of the residential construction in rural areas of Turkey. The material used, in order of frequency, includes clay bricks, concrete blocks, timber, earth blocks and stone. Clay brick masonry is thus the most common material for bearing wall construction, whilst reinforced masonry constructions are very rarely constructed. According to the 2000 Building Census, 23.4% of the whole building stock in the northern Marmara Region is constructed in masonry.

The typical damage to masonry buildings can be caused by inadequate ties to parapets, out-of-plane failure of walls, damage due to thrust from the roof, and pounding of adjacent buildings. Nevertheless, documentation of damage to Turkish masonry buildings is not found extensively in the literature, though this may be because reconnaissance teams generally focus their efforts within urban areas where reinforced concrete construction dominates. It is claimed by Sucuoğlu (1996) that most earthquake damage to URM buildings in Turkey is due to improper use of the brick materials used in the construction. Masonry buildings used to be constructed with clay blocks with a low (or even zero) percentage of voids, but the production of this type of brick has been terminated over the last few decades. URM buildings have recently been constructed using the same bricks that are produced to serve as infill material for RC buildings. This type of brick is more fragile and has a large percentage of voids (more than 50%) compared to the load-bearing block material with a low or zero void ratio. This bad construction practice has also been underlined by Akman (1996) and Kuran (2006).

2.1. *Masonry Building Classes in Turkish Building Stock*

There are several masonry building types in the Turkish building stock but only the major groups are considered herein; their classification is based on brick material, slab type and number of floors. There are three types of bricks considered: solid clay brick, briquette and hollow clay brick. Solid clay bricks are the bearing type brick materials with a low or zero void ratio (see Figure 1a). The behaviour of briquette (see Figure 1b), which is a fragile material, is not well known since there is a lack of technical information about this brick type. Nevertheless, its behaviour is close to that of the clay brick material with a high void ratio, but further research is still needed to verify this. The third brick type, hollow clay brick (see Figure 1c), is not a proper material for construction of masonry structures and is intended to serve as infill of reinforced concrete structures; however, it is widely used for masonry buildings in Turkey since it is easy to access and cheaper than other materials.



(a) (b) (c)

Figure 1 Main brick types used in Turkish masonry

There are two main types of slabs in the Turkish URM building stock: timber and RC. There are two main differences between these slabs: firstly, the weight and secondly, the boundary conditions created for the walls. RC slab types lead to a rigid diaphragm behaviour which should keep the walls together during strong shaking; however, the larger mass leads to higher inertia forces induced in the building. Hence, it is not straightforward to ascertain which floor type behaves better during strong ground shaking.

More than two thirds of the masonry buildings in the northern Marmara Region have a single storey. The percentage of single storey URM buildings increases as one moves to more rural areas or regions of Turkey which are less developed than the Marmara Region. There is also a correlation between the material and slab types with the number of floors. It is found that in the existing building stock, timber-floor structures with clay bricks do not exceed two floors, or briquette structures very rarely exceed a single floor. URM buildings with RC slabs and with clay brick material generally go up to 4 storeys. In total, 14 types of masonry buildings have been identified for the northern Marmara Region: 1 to 4 storey RC slab solid clay brick buildings (4 classes), 1 to 4 storey RC slab hollow clay brick buildings (4 classes), 1 to 2 storey timber slab solid clay brick buildings (2 classes), 1 to 2 storey timber slab hollow clay brick buildings (2 classes), single storey RC slab briquette buildings (1 class) and finally single storey timber slab briquette buildings (1 class). There are other types of masonry found in the building stock, namely, adobe, stone, and bağdadi. Bağdadi is an old type of construction where a timber frame is filled in with masonry units. However, these building classes have not yet been considered as part of this research work.

3. NONLINEAR ANALYSES OF TURKISH MASONRY BUILDINGS

Nonlinear dynamic analyses have been conducted on 28 different case studies (hollow and solid clay bricks with timber and RC slabs with varying number of storeys) in order to understand the structural properties of Turkish masonry buildings. Analyses have been carried out using the structural analysis software Ruaumoko (Carr, 2007). In each model, the masonry walls and spandrels are modelled as simplified frame elements; a representation of a 3-storey RC slab masonry wall with frame elements and ring beams is shown in Figure 2.

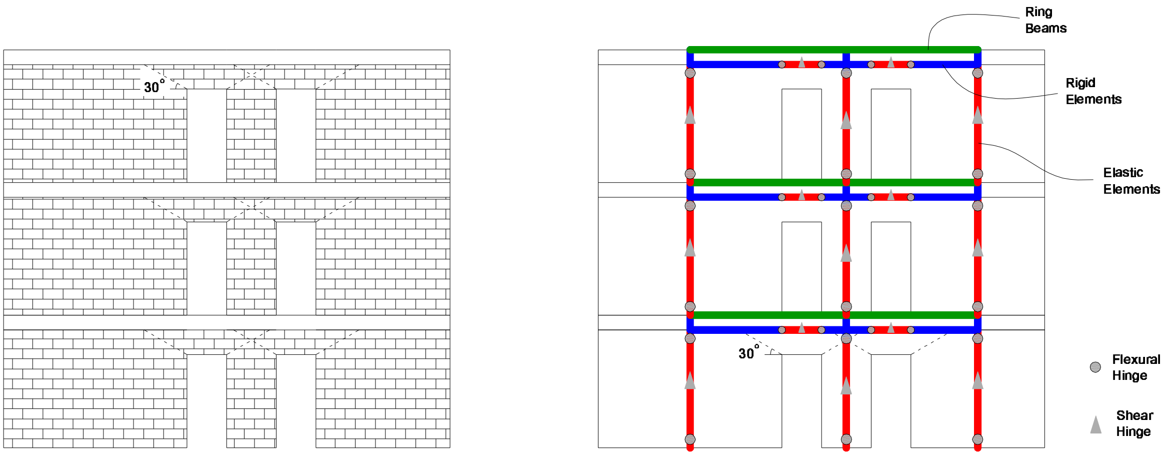


Figure 2 Representation of a 3-storey masonry wall with frame elements

The pier and spandrel elements are represented with simple elastic 1D elements whilst the flexural and shear hinges are introduced at the member ends and at the member centres, respectively (see Restrepo-Velez and Magenes, 2004 and Pasticier *et al*., 2008 for similar modelling details). Three different types of failure modes have been considered for the piers: flexural (rocking) (see Eqn. 3.1), diagonal shear (see Eqn. 3.2) and sliding shear (see Eqn. 3.3).

$$M_y = \frac{pD^2 t}{2}\left(1 - \frac{p}{kf_u}\right) \tag{3.1}$$

$$V_y^d = \frac{f_{tu} D t}{b}\sqrt{1 + \frac{p}{f_{tu}}} \tag{3.2}$$

$$V_y^s = \frac{f_{tu} + \mu \frac{p}{\gamma_m}}{1 + \frac{2H_0}{Dp} f_{tu}} \tag{3.3}$$

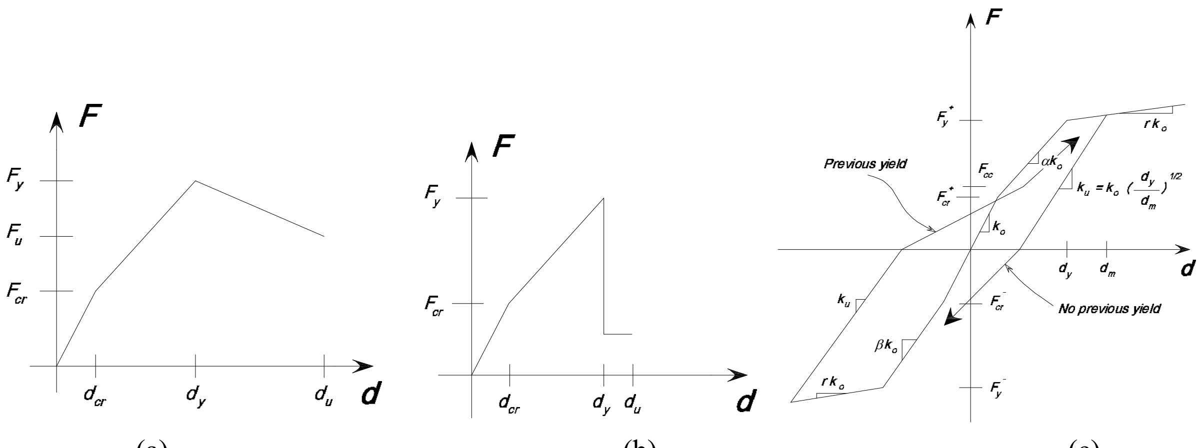
where f_{tu} represents the conventional tensile strength of masonry (not the tensile strength of the bed joints), b is a parameter which is assumed to be dependent on the H/D aspect ratio of the pier (b=1 for $H/D \leq 1.0$, b=H/D for $1<H/D<1.5$ and b=1.5 for $H/D \geq 1.5$: see Benedetti and Tomazevic, 1984, for further details), D is the pier section depth, p is the mean vertical stress on the pier, f_u is the compressive strength of masonry, t is the pier thickness, and k is a coefficient which takes into account the vertical stress distribution at the compressed toe (it can be assumed as a block, thus leading to k=0.85). Further details of these formulae can be found in Calvi and Magenes (1997) and Pasticier *et al*.(2008).

Two failure modes have been assumed for the spandrels: flexural and shear. The capacity for the flexural springs is described in Eqn. 3.4, whilst the ultimate shear capacity is given as $V_u = f_{v0d}ht$ where h is the section depth of the spandrel and f_{v0d} is the design shear strength with no axial force.

$$M_u = \frac{H_p h}{2}\left(1 - \frac{H_p}{k.f_{hd}.h.t}\right) \tag{3.4}$$

where H_p is the minimum of the horizontal shear resistance of the element or the value $0.4f_{hd}ht$, where f_{hd} is the compressive strength of the masonry in the horizontal direction in plan of the wall; when carrying out nonlinear static analysis, the design codes suggest that f_{hd} can be assumed to be equal to the mean value f_{hm}, which is determined by the following: $f_{hm}=f_{hk}/\alpha_{mc}$ where α_{mc} is the coefficient (assumed equal to 0.7) applied to the characteristic strength f_{hk} (ANDILWall, 2007).

The force – deformation relationship for flexural behaviour has been represented with a tri-linear curve (see Figure 3a) whilst the shear behaviour is defined again with a tri-linear curve with brittle behaviour following yield (Figure 3b). In the case of RC slabs, there are also ring beams circling around the building and sometimes above every wall, including the interior ones. The nonlinear behaviour of the ring beams has been modelled with the Modified Takeda rule (Carr, 2007). The cyclic behaviour of hinges has been defined with a tri-linear SINA hysteretic rule (Carr, 2007) with stiffness degradation (see Figure 3c). This hysteretic behaviour is also suitable for the definition of three different limit states which are used in the displacement-based loss assessment calculations, described in Section 4. The shear deformation limits considered in this study for the masonry spandrels are 0.06, 0.1, and 0.21%, shear deformation limits for piers are 0.06, 0.1 and 0.3%, whilst the flexural deformation limit states are 0.12, 0.2, and 0.5% for the 1st, 2nd and 3rd limit states, respectively. These values are based on the limit state drift capacities suggested by Restrepo-Velez and Magenes (2004) and Pasticier *et al*. (2008).



(a) (b) (c)

Figure 3 Monotonic (a and b) and cyclic (c) force-deformation rules used for the definition of nonlinear hinges

Twenty strong motion records have been extracted from the PEER NGA database and applied to the case study buildings for nonlinear dynamic analyses. The records have been selected using the following criteria: earthquakes on strike-slip faults, with magnitudes between 6.8 and 7.5 and at a distance (considering all distance definitions in the database) of 20 to 160km. The records have peak ground accelerations which vary steadily between 0.02 and 0.51g. The analysis results for a sample case study wall can be seen in Figure 4. Static nonlinear analysis results with two different loading profiles are shown (inverted triangle and storey-mechanism shapes), together with the capacity curve obtained from multiple nonlinear time history analyses (NLTHA). The main reason for using the more complicated NLTHA is due to the uncertainty in the deformed shape of the case study buildings, which is seen from the nonlinear static analyses to have a large influence on the initial stiffness, and thus the period of vibration, of the building.

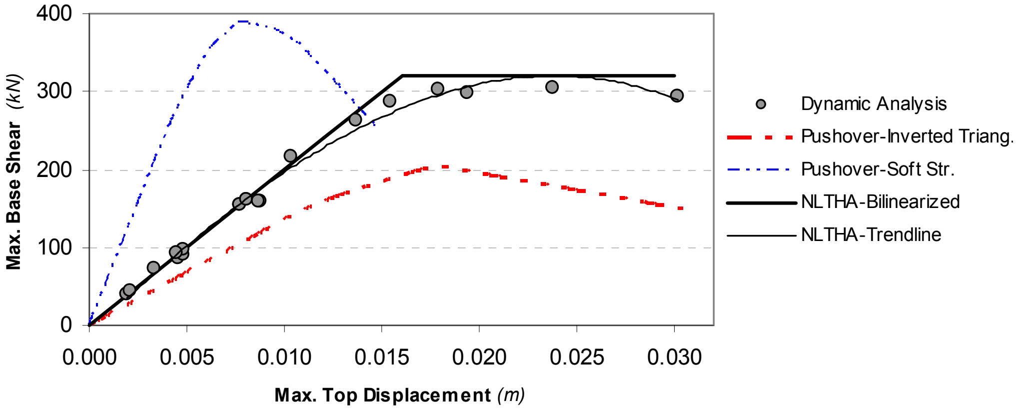


Figure 4 Comparison of the results of dynamic and static time history analyses for a single case study building

4. DISPLACEMENT BASED LOSS ASESSMENT METHOD FOR TURKISH URM BUILDINGS

The analyses described in the previous section have been used to calibrate the equations used in a displacement-based method for the assessment of Turkish masonry buildings at an urban scale. A brief summary of this method is provided herein, whilst readers are referred to Crowley *et al*. (2004; 2006) for further information about the basics of the Displacement-Based Loss Assessment Method (DBELA) for RC buildings.

The first step of the method is the generation of a random population of buildings which should represent the urban building stock. Monte Carlo simulation is used to generate thousands of buildings, each of which featuring properties (e.g. storey height, pier height, limit drift values) with a variability that is defined *a priori* using a mean, standard deviation and probabilistic distribution. Once the population has been generated, the period of vibration of each building is estimated using an empirical relationship between the yield period of vibration (T_y) and the height of the building.

In order to obtain a realistic estimate of the yield periods of URM buildings as a function of their height, the nonlinear dynamic analyses conducted on 2D walls extracted from real buildings in the existing stock described in Section 3 have been used. The calculation of T_y is based on the yield stiffness found by a bi-linearization of the capacity curve obtained based on a set of nonlinear time-history analyses (see Figure 4). The effective mass has been taken as 90% of the total mass following the suggestions by Priestley *et al*. (2007). The periods of vibration of the case study masonry buildings with different brick types (solid and hollow clay) and slab types (RC and timber) are presented in Figure 5.

The results show that the slab type has a higher influence on the period of vibration than the type of bricks and thus period-height relationships for timber and RC slab buildings have been derived, as presented below:

$$T_y = 0.039H \qquad \text{(with timber slabs)} \tag{4.1}$$

$$T_y = 0.062H^{0.87} \qquad \text{(with RC slabs)} \tag{4.2}$$

where H is the height of the building in meters. The results are compared with the formula provided in 2003 version of Eurocode 8, which has been increased by 20% in order to reach an estimate of the period at yield (Goel and Chopra, 1997). Briquette masonry buildings were not modelled as part of this study, but considering that the type of bricks was seen to have a reduced influence on the period of vibration, and also considering that this type of masonry should be similar to the hollow clay brick buildings, the same period-height relationships described above may be adopted for this building type.

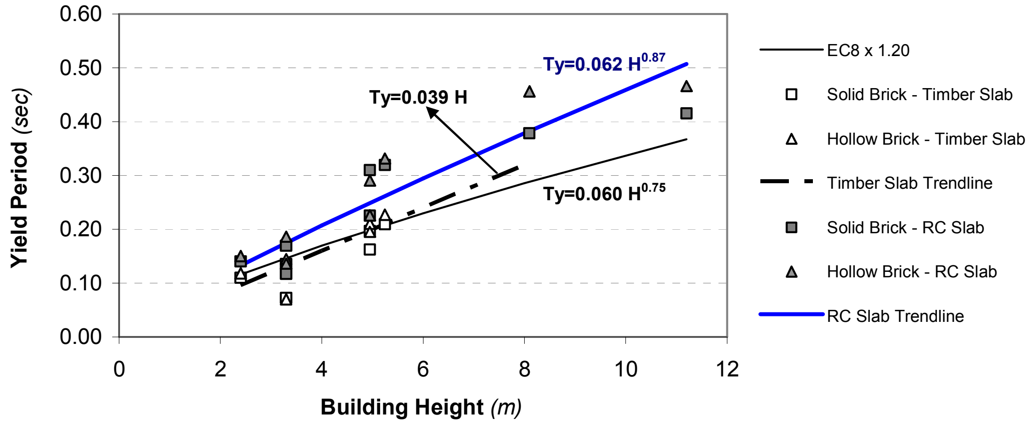


Figure 5 Period-height relationship obtained for Turkish URM buildings

In the DBELA method, once the period of vibration is calculated, the displacement capacity of each building in the random building population is then estimated at different limit states to damage (moderate, extensive and complete damage). Formulae for the displacement capacity of a single-degree-of-freedom (SDOF) representation of the building class have been derived from simple structural mechanics principles. Masonry buildings are currently assumed in DBELA to have a storey-sway response mechanism at the ground floor. The formula for the limit state displacement capacity at the centre of seismic force of the storey-sway mechanism is given in Eqn. 4.3 (Restrepo-Velez and Magenes, 2004).

$$\Delta_{LS} = \theta_y \kappa_1 H + \kappa_2 (\theta_{LS} - \theta_y) h_s \tag{4.3}$$

where θ_y is the yield rotation capacity, κ_1 is the effective height coefficient (to obtain the equivalent height of the deformed SDOF system), H is the height of the building, θ_{LS} is the second or third limit state rotation capacity, h_s is the pier height, and κ_2 is the effective height coefficient of the masonry piers. The κ_1 and κ_2 coefficients are based on how the structure deforms. These coefficients can be calculated for a certain building if the mass distribution and mechanism shape is known (details of these coefficients can be found in Restrepo-Velez and Magenes, 2004). It should be noted that the κ_1 coefficient would be 2/3 if the masonry building had whole mass lumped at the storey level. This coefficient has to be calculated based on the ratio of the floor mass (m_f) to the mass of the masonry walls in the same floor (m_m) and the expected ductility, whilst the variation of the κ_2 coefficient with ductility is negligible. The variation of these two coefficients with m_f/m_m ratio and with the number of floors is given in Figure 6. The m_f/m_m ratio is found to be 0.87 with a 52% coefficient of variation for timber-slab structures and 0.84 with a 46% coefficient of variation for RC slab structures.

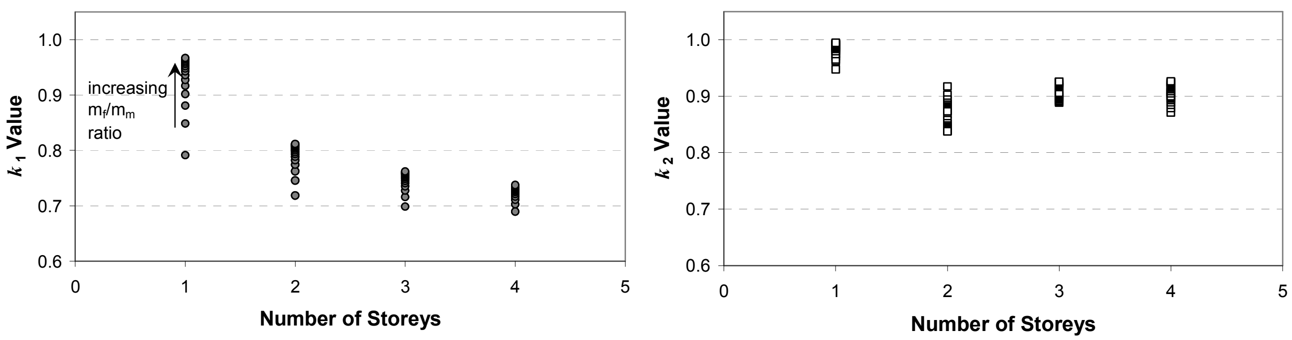


Figure 6 Variation of κ_1 and κ_2 coefficients with number of storeys

In the masonry building stock located in north-western Turkey, the average storey height has been found to be 2.62m with a coefficient of variation of 8% and a lognormal distribution. The pier height has also been examined and has been found to be 2.40m on average with a 15% coefficient of variation and a normal distribution (Bal *et al.*, 2008).

There is a lack of laboratory tests on the materials used in masonry walls in Turkey. The properties of the recently constructed masonry buildings can be extracted from the intensive studies which have been carried out on infill wall materials. Most of the knowledge about the URM buildings in Turkey is based on the findings by Bayülke (1992) following several shaking-table tests on single-storey URM buildings constructed with different materials used in Turkey. A continuation of these tests for URM buildings constructed using bricks with a high void ratio has been conducted by Kuran (2006). The main outcome of all of these tests has been the drift limits for URM buildings. Bayülke (1992) found that 1/200, 1/130 and 1/100 interstorey drift values were suitable limits for slight, moderate and extensive damage for URM buildings with low or zero void ratios. Whilst the test results extracted from Kuran (2006) revealed drift limits of 1/370, 1/200 and 1/100 for URM buildings constructed with bricks with a high percentage void ratio.

The displacement capacity of the case study buildings from the nonlinear analyses has also been checked in order to compare the limit state drifts with the aforementioned experimental values. Floor displacement values were recorded when the limit state deformation of the piers (as described in Section 3) was exceeded in a given floor. Limit state drifts of 1/625, 1/490 and 1/400 for timber slab structures for LS1, 2 and 3, respectively, have been found whilst limit states of 1/580, 1/440 and 1/260 for RC slab structures for LS1, 2 and 3, respectively, were obtained. These limit states are lower than those proposed by Bayülke (1992); however, it should be noted that the limit states defined following the nonlinear dynamic analyses conducted within this research work are based on the pier limit state deformations (the hinge element deformations) proposed by Restrepo-Velez and Magenes (2004) and Pasticier *et al*. (2008). These element deformation limits need to be calibrated for Turkish type masonry either by experimental tests or at least by detailed finite element analyses, and thus further research is needed on this aspect. The drift limits of briquette masonry structures can be considered to be the same as the hollow clay brick buildings, but further research is needed for this type of building which comprises 5% of the whole building stock.

With a database of structural characteristics for the given building stock, such as those which have been described above for the Turkish masonry buildings, random populations of buildings can be generated for each building class (e.g. hollow clay brick masonry structures with a storey-sway mechanism and with a given number of storeys) and the yield period of vibration and displacement capacity at the three different damage limit states can be calculated for each randomly generated building. An equivalent linearization approach is applied in DBELA and hence for post-yield limit states, if an elastic-perfectly plastic behaviour is assumed, the buildings can be modelled using the secant period of vibration, based on the following formula:

$$T_{LS} = T_y \sqrt{\mu_{LS}} \tag{4.4}$$

where μ_{LS} is the ductility at the limit state in question calculated from the limit state displacement divided by the yield displacement. The next step in defining the vulnerability of the building stock involves the comparison of the structural capacity of the buildings with a prediction of the ground motion from a given scenario earthquake. In DBELA, ground-motion prediction equations are used to define the demand in terms of over-damped displacement response spectra. The damping correction equation presented in the 1994 version of EC8 (CEN, 1994) has been assumed herein following the recent recommendations given in Priestley *et al.* (2007):

$$\eta = \sqrt{\frac{7}{2 + \zeta_{eq}}} \tag{4.5}$$

where η is the correction factor and ζ_{eq} is the equivalent viscous damping; for masonry buildings the damping values suggested for each limit state (5%, 10% and 15%, respectively) in Restrepo-Velez and Magenes (2004) have been adopted. For a given displacement response spectrum, the displacement demand at the limit state period of vibration of a given building in the random population can be compared with its limit state displacement capacity; the sum of all buildings whose displacement capacity is lower than the displacement demand divided by the total number of buildings gives an estimation of the probability of exceeding a given limit state. The sample size is gradually reduced from one limit state to the next by removing the buildings which do not exceed the limit state. Three limit states are currently considered for masonry buildings such that the proportion of buildings falling within four damage bands (slight, moderate, extensive and complete) can be calculated. An application of the DBELA methodology to masonry buildings can be found in Bal (2008).

5. CONCLUSIONS

A procedure for the displacement-based earthquake loss assessment of masonry buildings in the building stock of the northern Marmara Region has been presented herein. A classification system for the masonry buildings within this region along with the geometrical properties (i.e. storey height and pier height values) of these building types is provided. Nonlinear time history analyses have been conducted on 28 different case study buildings from the region with 20 randomly chosen acceleration records with PGA values varying from 0.02g to 0.51g. Period-height relationships and drift limit states for timber and RC slab structures have been extracted from the results of these analyses. There is a need for further research to define the failure mechanism and deformed shapes of masonry buildings in more detail. The results of the dynamic analyses from the present study can be utilized for this purpose. Furthermore, detailed finite element analyses are needed for the definition of the member deformation limit states since there is a lack of analytical and experimental studies considering this issue for Turkish buildings.

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