DEVELOPMENT OF A DISPLACEMENT-BASED EARTHQUAKE LOSS ASSESSMENT METHOD FOR TURKISH BUILDINGS

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ABSTRACT :

A Displacement-Based Earthquake Loss Assessment (DBELA) methodology for urban areas, which compares the displacement capacity of the building stock with the displacement demand from earthquake scenarios, has been under development throughout the recent years. The building stock is modelled as a random population of building classes with varying geometrical and material properties through Monte Carlo simulation. The period of vibration of each building in the random population is calculated using a simplified equation based on the height of the building and building type, whilst the displacement capacity at different limit states is predicted using simple equations which are a function of the randomly simulated geometrical and material properties. The displacement capacity of each building is then compared to the displacement demand obtained from an over-damped displacement spectrum, using its period of vibration; the proportion of buildings which exceed each damage state can thus be estimated. The methodology has recently been further verified and calibrated by carrying out a number of structural analyses on case study buildings and frames from the existing European building stock, and in particular from Turkish buildings. Dual (frame-wall) buildings and reinforced concrete structures with infill walls have also been included in the methodology. The method has also been extended to include different types of Turkish masonry structures constructed with concrete and timber slabs. DBELA has been further calibrated to the Turkish building stock following the collection of a large database of structural characteristics of buildings from the northern Marmara Region. The probabilistic distributions for each of the structural characteristics (e.g. storey height, steel properties etc.) have been defined using the aforementioned database. Damage ratios relating cost of repair to cost of replacement for slight, moderate, extensive and complete damage for reinforced concrete buildings have also been obtained for the Turkish building stock. Preliminary results in terms of social losses for a single earthquake scenario close to the city of Istanbul are presented.

KEYWORDS: Loss assessment, displacement-based, reinforced concrete, masonry, Turkey

1. INTRODUCTION

Assessing the seismic vulnerability of buildings in earthquake-prone regions of the world is of growing importance since such information is needed for a reliable estimation of the losses that future earthquakes are likely to induce. The outcome of such loss assessment exercises is useful in the planning of urban/regional-scale earthquake protection strategies which has become a priority in many countries, and in particular in Turkey following the destructive earthquakes of 1999. Considering the size of the building inventory, it is not a simple task to define the properties and characteristics of the buildings in Istanbul and the surrounding area. A recent study focused on the characteristics of the RC building stock in the northern Marmara Region of Turkey (Bal *et al.*, 2007; 2008a). This region, north of the Marmara Sea, consists of three provinces: Tekirdağ, İstanbul and Kocaeli (Edirne and Kırklareli are also in this region, but they are relatively far away from the Northern Anatolian Fault which is the main source of seismic activity).

In this paper, the latest developments of a displacement-based earthquake loss assessment methodology, initially developed by Crowley *et al.* (2004 and 2006) are presented. The existing method has been extended to dual systems, masonry infilled RC frames and masonry structures. A preliminary study to define the losses expected in İstanbul, based on a single scenario earthquake, is also presented.

2. DISPLACEMENT-BASED EARTHQUAKE LOSS ASSESSMENT (DBELA) METHODOLOGY

A brief summary of the DBELA method is provided herein, though readers are referred to Crowley *et al.* (2004; 2006) for further information. 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 with different material and geometrical properties (e.g. storey height, beam length, section dimensions, steel yield strain or pier height for masonry structures); the variability of each property 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. For reinforced concrete buildings, the following formula, derived by Crowley and Pinho (2006) for infilled RC European buildings, is adopted:

$$T_y=0.055H \tag{2.1}$$

where H is the height in meters. The coefficient of variation of this formula has been estimated as 25%. The period-height relationship of existing dual (frame-wall) systems has also been examined and the equation presented below has been obtained (see Vuran *et al.*, 2008):

$$T_y=0.075H \tag{2.2}$$

Dual systems in the existing Turkish building stock exhibit a weak contribution from the structural RC walls since the wall length, which is a key parameter affecting the overall behaviour of the structure, is relatively short in existing buildings. The average wall length has been found to be 2.25m for embedded beam systems and 1.62m for emergent beam systems (Bal *et al.*, 2007; 2008a). Details of the calculation for the yield period of vibration for masonry structures, based on nonlinear time-history analyses, can be found in Bal *et al.* (2008b). For masonry buildings, the yield period can be estimated as a function of the height and slab typology:

$$T_y=0.039H \quad \text{(with timber slabs)} \tag{2.3}$$
$$T_y=0.062H^{0.87} \quad \text{(with RC slabs)} \tag{2.4}$$

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. For reinforced concrete buildings, different building classes are defined as a function of the assumed response mechanism; buildings may respond with a global response mechanism (beam-sway), or a soft storey response mechanism (column-sway). Masonry buildings are assumed to have a column-sway response mechanism at the ground floor. The formula for the displacement capacity at the centre of seismic force of the beam-sway mechanism is presented in Eq. (2.5) whilst that for column-sway mechanism (for either RC or masonry) is shown in Eqn. (2.6):

$$\Delta_{LS}=\theta_{by}\kappa_1 H+(\theta_{bLS}-\theta_{by})\kappa_1 H \tag{2.5}$$
$$\Delta_{LS}=\theta_{cy}\kappa_1 H+\kappa_2(\theta_{cLS}-\theta_{cy})h_s \tag{2.6}$$

where θ_{by} and θ_{cy} are the yield rotation capacities of the beams and columns, respectively; 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_{bLS} and θ_{cLS} are the rotation capacities at a given post-yield limit state of the beams and columns, respectively; h_s is the ground floor story height for RC frames and the pier height for masonry buildings; κ_2 is the effective height coefficient of the masonry piers (for RC buildings this is taken as 1). For reinforced concrete buildings, the calculation of the rotation capacity of the beams and columns and the effective height coefficients for the two mechanisms is further described in Crowley *et al*. (2004). The rotation capacities are functions of the limit state strains of the steel and concrete and the dimensions of the section. For masonry buildings, the effective height coefficients, which are originally suggested by Restrepo-Velez and Magenes (2004), for buildings of 1 to 4 storeys have been calibrated for the Turkish building stock by Bal *et al*. (2008b).

The definition of the collapse mechanism for RC buildings (beam- or column-sway) may be determined, for instance, by a reliable displacement-based adaptive pushover (DAP, Antoniou and Pinho, 2004); however, the nature of loss assessment studies at an urban scale does not allow pushover analyses to be conducted for every single building in the stock. One possible solution can be to calculate a sway potential index which gives an estimate of the expected mechanism of the structure based on the structural properties. Priestley *et al*. (2007) suggest the use of a strength-based sway potential index; however, including the strength of the buildings in the DBELA methodology would require the collection of reinforcement ratios of different structural elements and would thus add significant time and effort to the procedure. A new stiffness- (or deformation-) based approach which compares the rotational stiffness (or rotational capacity) of columns and beams in a given floor has been proposed by Abo el Ezz (2008) and further developed by Bal (2008). The application of both the strength- and displacement-based sway potential indices to a number of case study buildings has shown a fairly good agreement with the DAP results, as presented in Bal (2008). The displacement-based sway index considers that a higher sway index will represent a column-sway mechanism, where the columns will yield before the beams. In order for this to occur, it means that the beams are stiffer than the columns and thus the ratio of the beam depth to beam length will be higher than the ratio of column depth to column length (i.e. storey height):

$$(h_b/L_b) / (h_c/L_c) \tag{2.7}$$

where h_b and h_c are the beam and column section depths, respectively, while L_b and L_c are the beam and column lengths, respectively. The value of the index for the i^{th} joint of a certain floor is given by:

$$R_i = \frac{h_{b,L} / L_{b,L} + h_{b,R} / L_{b,R}}{2(h_{c,B} / L_{c,B})} \tag{2.8}$$

where the sub-indices “L” and “R” refer to “Left” and “Right” and “B” refers to “Below” the joint “i”. The index per floor, $S_{def,j}$ can then be obtained by averaging the results of Eqn. (2.8):

$$S_{def,j} = \left[\sum_{i=1}^{n} R_{i,j}\right] / n \tag{2.9}$$

where “n” is the total number of joints at floor “j”.

With a database of structural characteristics for the given building stock, random populations of buildings can be generated for each building class (e.g. reinforced concrete column-sway frames of a given number of storeys), the mechanism can be identified for each building (in the case of RC buildings) and then 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:

$$T_{LS} = T_y \sqrt{\mu_{LS}} \tag{2.10}$$

where μ_{LS} is the ductility at the limit state in question. 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 overdamped 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):

$$\eta=[7 / (2+\xi_{eq})]^{0.5} \tag{2.11}$$

where η is the correction factor and ξ_{eq} is the equivalent viscous damping, which for reinforced concrete frames has been obtained as a function of ductility using the equations presented in Priestley *et al*. (2007), whilst for masonry buildings the damping values suggested for each limit state in Restrepo-Velez and Magenes (2004) have been adopted.

For a given displacement response spectrum, the displacement demand for 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 have been considered in the present study such that the proportion of buildings falling within four damage bands (slight, moderate, extensive and complete) can be calculated.

3. CALIBRATION OF DBELA TO THE TURKISH BUILDING STOCK

The DBELA methodology has been calibrated to the Turkish building stock following the collection of an extensive database of buildings from the northern Marmara Region. Bal *et al*. (2007; 2008a) present a review of the structural characteristics of reinforced concrete buildings which have been obtained from this database. A further set of masonry buildings have been collected from the same region for the present study in order to define the characteristics of this building typology. A summary of the geometrical and material properties which have been obtained from the aforementioned studies is presented in what follows.

3.1. Building Classes

After a detailed assessment of the properties of the existing building stock, reinforced concrete building in the Turkish building stock have been divided into 8 main groups. These groups are based on “good” and “poor” structural behaviour in terms of the “expected” (not necessarily the resultant) failure mechanism, “emergent” and “embedded” beam types and “frame” or “dual” vertical bearing system. As far as masonry buildings are concerned, the main groups are based on whether the slab is RC or timber and whether the walls are constructed with solid clay brick, hollow clay brick or briquette.

3.2. Geometrical Properties

Tables 3.1 and 3.2 show the probabilistic distributions of the geometrical properties which have been obtained from the Turkish database of masonry and reinforced concrete buildings, respectively, and which have been used to define the vulnerability of these buildings, as described in Section 2. Further information is available in Bal *et al*. (2007).

Table 3.1. Geometrical characteristics of masonry buildings

Parameter	Mean Value	CoV	Probabilistic Distribution
Regular storey height	2.62m	8%	Lognormal
Ground floor pier height	2.40m	15%	Normal

Table 3.2. Geometrical characteristics of reinforced concrete buildings

Parameter	Mean Value	CoV	Probabilistic Distribution
Regular storey height	2.84m	8%	Lognormal
Ground floor storey height	3.23m	15%	Lognormal
Beam length	3.37m	38%	Lognormal
Poor - Beam depth	0.60m	15%	Normal
Good - Beam depth	0.48m	14%	Normal
Structural Wall Length *(Poor – Dual – Emergent)*	2.25m	48%	Lognormal
Structural Wall Length *(Poor – Dual – Embedded)*	1.62m	32%	Exponential ($e^{-0.8x}$)
Column depth at ground floor *(Poor-Frame-Emergent Beam)*(1)			
1-3 storeys	0.45m	12%	
4 storeys	0.49m	30%	Lognormal
5 storeys	0.65m	30%	
≥ 6 storeys	0.70m	28%	

(1) Due to a lack of space, the column properties for all types of structures could not be listed here. Readers are referred to Bal *et al*. (2007) for further information.

3.3. Material and Limit State Properties

Two different steel types have been used extensively in Turkey over the past 40 years: S220 and S420; the characteristic strength of these steel types is 220MPa and 420MPa, respectively. For the vulnerability assessment of the Turkish building stock, it is important to have a reliable estimate of the mean properties and coefficient of variation of the yield strain, as this is used in the calculation of the yield rotation described in Section 2. Bal *et al*. (2007 and 2008a) have obtained rebar testing data from the Turkish Chamber of Civil Engineers and studies by Akyüz and Uyan (1992) to define the yield strength distributions of the two steel types. The S220 steel has been found to have a mean yield strength of 371MPa and a coefficient of variation (CoV) of 24%. The variability of this steel type is much higher than that found using US steel (e.g. Mirza and MacGregor, 1979) and recent sensitivity studies with the DBELA methodology (Crowley *et al*., 2005) have shown that this variability has a large influence on the loss model results. Hence, it is particularly important to correctly define this parameter for the Turkish building stock. The production of S420 steel has been found to have improved since it was introduced in the ‘70s and thus two different distributions have been identified for the study herein; one for the steel produced from 1978-1988 and another for the steel between 1988-1998. The ’78-’88 S420 steel has been found to have a mean strength of 430MPa and a CoV of 40% whilst the ’88-’98 steel has a mean strength of 470MPa and a CoV of 16%. In order to calculate the limit state rotation described in Section 2, the limit state strains in the steel and concrete at the second and third (post-yield) limit states are required. For this preliminary study, a set of strains taken from moment-curvature analyses of some typical sections used in Turkish construction have been used, as presented in Table 3.3.

The interstory drift capacity of different types of masonry buildings in Turkey has been taken from experimental tests and data reported in Bayülke (1992) and Kuran (2006), as presented in Table 3.4. However, further research on these limit states is needed since Bal *et al*. (2008b) found much lower drift values from nonlinear time-history analysis based on pier interstorey drift limit states suggested by Restrepo-Velez and Magenes (2004), though these values were not calibrated for the Turkish building stock.

Table 3.3. Steel and concrete post-yield strain capacities for poorly confined members

Limit State	Mean Concrete Strain	CoV	Probabilistic Distribution	Mean Steel Strain	CoV	Probabilistic Distribution
2	0.0035	51%	Lognormal	0.015	25%	Normal
3	0.0075	51%	Lognormal	0.035	25%	Normal

Table 3.4. Interstory drift capacities of masonry walls

Limit State	Mean Value (Clay high % voids or Briquette)	Mean Value (Clay low % voids)	CoV	Probabilistic Distribution
1	0.27%	0.5%	25%	Normal
2	0.5%	0.77%	25%	Normal
3	1.0%	1.5%	25%	Normal

3.4. Damage Ratios

Data which can help to improve the calculation of the mean damage ratio for Turkish building construction has also been studied in this research work. A damage ratio represents the ratio between the cost of repair/retrofitting to the cost of replacement of the building stock for a given level of damage; this is an important ratio for loss estimation studies as when this ratio, for example, is multiplied by the proportion of buildings with moderate damage and the value of the building stock, the direct loss due to moderate damage can be directly obtained. The same procedure can be applied for slightly or heavily damaged and collapsed buildings. The cost of retrofitting applied to 231 buildings has been obtained from a variety of sources, as discussed in Bal *et al*. (2007). The real retrofitting costs have been utilised rather than the predicted costs (*kesif bedeli*) reported within the appendix of the retrofitting design calculations. In order to calculate the cost of replacing a building, the area of the building has been multiplied by the "Approximate Unit Construction Costs for New Buildings" published by the Ministry of Public Works and Settlement annually. Using the ratio obtained from each source, a mean ratio of 33% has been obtained for retrofitting the moderately damaged buildings. The standard deviation of this value has been found to be 8.2 %.

According to the code and law requirements in Turkey, after an earthquake, only moderately damaged buildings are retrofitted: extensively and completely damaged buildings are demolished and slightly damaged buildings are repaired. Theoretically, a ratio of cost of repair/retrofitting to cost of replacement of 100% would thus be obtained for the extensive and complete damage bands, as the building needs to be rebuilt. In reality, additional costs arising from the need to demolish the existing building and transport the rubble away from the site are incurred. An exercise to calculate the order of such additional costs has been carried out via the announced official unit costs of the Ministry of Public Works and Settlement. This calculation has been carried out using a sample of 37 buildings, the details of which can be found in Bal *et al*. (2007). The ratio of the cost of carriage and demolition to the cost of replacement of 5% for extensive damage and 4% for complete damage has been obtained. Thus, the final ratios of cost of repair/retrofitting to cost of replacement would be 105 % for extensive damage and 104% for complete damage. It is perhaps counter-intuitive that the cost incurred by extensive damage is higher than that caused by complete damage, however, as mentioned previously, this is because the whole building needs to be demolished when extensive damage occurs whilst only a proportion of the building needs to be demolished after partial or complete collapse.

The ratio of repair cost of slight damage to the replacement cost is the most difficult damage ratio to determine, and has been studied by Bal *et al*. (2007) by examining the governmental loans that have been given after recent earthquakes such as Adana (1998), Kocaeli (1999) and Bingol (2003). The loan which was given to the owners for repairing their properties for slight damage was found to be almost half of that which was given to the owners of the moderately damaged buildings. The slight damage ratio can thus be assumed to be approximately 16 %. To summarise, the cost ratios for the reinforced concrete Turkish construction are proposed as follows; 16% for slight damage, 32% for moderate damage, 105% for extensive damage and 104% for complete damage.

4. PRELIMINARY RESULTS OF A TEST-BED APPLICATION

The developments to the method presented in the previous sections are still in the process of being coded into the DBELA software and thus only a preliminary set of results, based a test-bed study, will be summarized herein; these results will be updated in the future based on the developments described herein. The test-bed study has been carried out as part of the European NERIES project where the aim was to compare the results of a number of European loss estimation tools. As part of this exercise, all necessary information on the exposure of the buildings and the ground motions for a M_w 7.5 earthquake, beneath the Sea of Marmara, within 8131 geocells have been provided, whilst the vulnerability has been predicted using 5 different methodologies The 8131 geocells, which are approximately 600x400m rectangular areas have been used as the smallest geographical area at which soil conditions, building inventory information and ground-motion input are modelled; it is noted that the influence of the resolution at which data is collected within a loss model is currently being studied by the authors to ascertain whether it is worthwhile to collect inventory and soils data at higher levels of resolution. Readers are referred to Strasser *et al.* (2008) for further information on the exposure and ground-motion data and the other methods applied in this test-bed study. The building classes considered in the DBELA method and the properties assumed for each of the 54 building classes adopted are presented in Bal *et al.* (2008c).

The results of this scenario earthquake exercise indicated that about 47,000 buildings (6.4% of the total building stock) would collapse, 81,000 buildings would be extensively damaged (i.e. beyond repair), 200,000 buildings would be moderately damaged and about 400,000 buildings would experience none to slight damage. Figure 4 shows the spatial distribution of the collapsed buildings in the 8131 geocells used in this study. In terms of casualties, based on the model by Spence (2007), these damage results would translate to 150,000 deaths and 364,000 serious injuries and around 4.5 million people rendered homeless, were the earthquake to occur at night. The spatial distribution of the collapsed buildings can be seen in Figure 1. It is observed that most of the damage will be concentrated in the European side of the city, especially south and south-west of the Golden Horn. The districts which are expected to be most affected are Eminönü, Kağıthane, Zeytinburnu, Bakırköy, Bahçelievler, Bağcılar in the European side, and the coast of Kadıköy, Maltepe and Kartal in the Asian side.

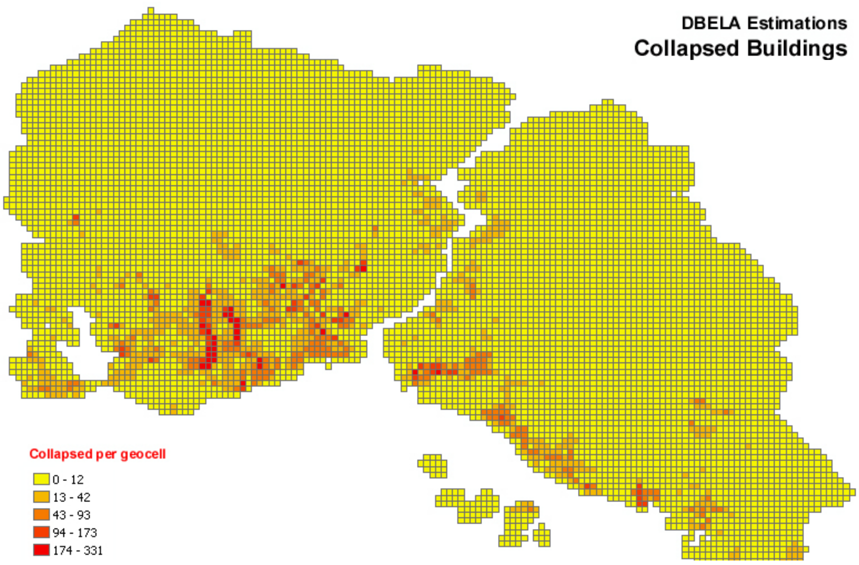

Figure 1 Spatial distribution of the estimated collapsed buildings in İstanbul for a single scenario earthquake

5. CONCLUSIONS AND FUTURE DEVELOPMENTS

A method for the displacement-based estimation of direct losses from scenario earthquakes has been introduced herein. The main concept of the methodology is that the displacement capacity of a representative SDOF system is compared with the displacement demand from an overdamped spectrum at the effective period of vibration of the structure. Modifications and additions to the methodology have been presented herein, in particular additional building classes covering dual systems, frames with infill walls, and masonry buildings have been added to the methodology. The accuracy of the method depends on the quality of the input data, which should be obtained from the properties of the building stock under examination. A summary of the collected input data for the Turkish building stock in the northern Marmara Region has thus been presented.

There is still a need for further research on the period-height relationship of infilled dual and infilled embedded frame structures as well as the equations to calculate the displacement capacities of infilled RC frames, dual and embedded frame systems. Damage ratios for masonry buildings also need to be calculated based on the cost of repairing and retrofitting this construction type; such data has already been collected from past Turkish earthquakes and will soon be processed. The influence of the resolution of the input data on the loss results will also be investigated to ascertain whether detailed data collection is worthwhile. Finally, a repetition of the scenario earthquake for Istanbul will also be carried out which will include all of the developments which have been presented herein, as well as those which will be undertaken in future research work.

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