Using sensitivity analyses to evaluate behaviour factor for mixed masonry-RC buildings in Lisbon

Fator de comportamento de edifícios mistos alvenaria-BA em Lisboa – recurso a análises de sensibilidade

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Abstract

The reliable estimation of the behaviour factor is still nowadays a relevant issue, since linear analysis is certainly one of the most used procedure for the seismic design or assessment of structures in the engineering offices. Even that a range of values for structural behaviour factor for different construction system is provided in the document of the code, detailed values for each typology should be defined regarding the characteristic of masonry structures. Therefore, in this work, the behaviour factor is evaluated for mixed masonry-reinforced concrete ("Placa") buildings in Lisbon. The behaviour factor is estimated by nonlinear static sensitivity analyses. Within sensitivity analyses, aleatory (in terms of mechanical parameters) and epistemic (in terms of structural and construction characteristics) uncertainties are considered. Considered uncertainties are important, due to the wide variety of material and construction details for masonry and mixed masonry-reinforced concrete buildings. The main results of the performed study are presented in this paper.

Resumo

Atualmente, a estimativa cuidada do fator de comportamento é ainda um assunto relevante, uma vez que as análises lineares são um dos procedimentos mais utilizados nos gabinetes de engenharia civil para o dimensionamento e a avaliação sísmica de estruturas. Apesar de uma gama de valores dos coeficientes de comportamento estrutural para diferentes sistemas de construção ser fornecida nos Eurocódigos, estes também deveriam ser definidos de forma detalhada para cada tipologia atendendo às características das estruturas de alvenaria. Deste modo, neste estudo, o fator de comportamento é avaliado para estruturas mistas de alvenaria-betão armado existentes na cidade de Lisboa. Este fator é definido com base nos resultados obtidos de análises estáticas não lineares sob a forma de análises de sensibilidade. Dentro das análises de sensibilidade consideraram--se incertezas aleatórias (relativamente aos parâmetros mecânicos) e epistémicas (relativamente às características estruturais e construtivas). Estas incertezas são importantes, devido à grande variedade do material e dos detalhes construtivos utilizados nos edifícios de alvenaria e nos edifícios mistos de alvenaria-betão armado. Os resultados mais relevantes deste estudo encontram-se apresentados neste artigo.

Keywords: Mixed masonry-RC buildings / Sensitivity nonlinear static analyses / / Behaviour factor

Palavras-chave: Edifícios mistos alvenaria-BA / Análises de sensibilidade estáticas não lineares / Fator de comportamento

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1 Introduction

Old masonry and mixed masonry-reinforced concrete (RC) buildings in Lisbon are one of the most vulnerable building types, since they were built considering only simple rules of construction, without reference to any particular seismic code. Since these buildings represent an important part of the building stock and still serve for housing and services, lately higher effort has been placed on engineering developments to understand better their seismic behaviour and to provide strengthening solutions to preserve these buildings, but also to protect the people.

To assess the behaviour of masonry buildings, different analysis methods have been progressively developed. The seismic assessment of the masonry or mixed masonry-RC structures can be obtained by performing static or dynamic analysis with linear or nonlinear behaviour. The adoption of linear methods for seismic analysis is considered mainly as a conventional approach and the most used and familiar among practicing engineers; thus, particular care should be taken on the definition of the behaviour factor (q-factor approach) and overstrength ratio (OSR) [1]. Due to the absence of specific q-factors for different masonry typologies, it is recommended that the assessment of the behaviour factor of a specific class should always be careful evaluated.

In the European context, static and dynamic experimental tests for the evaluation of the behaviour factors for different types of masonry buildings have been carried out by different authors (e.g. [2, 3, 4, 5, 6, 7]. Da Porto et al. [8] define the value of behaviour factor by using the cyclic lateral resistance test of walls. Moreover, the probabilistic approach [9] have been derived the behaviour factor for reinforced concrete structures. However, any of these structures correspond completely to the structures under investigation.

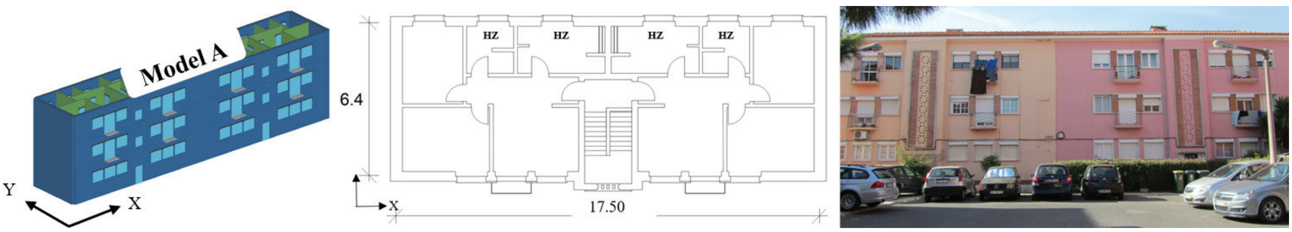
In this study, in order to enhance the existing information regarding the possible ranges of values of structural behaviour factor (q) and overstrength ratio, the seismic behaviour of mixed masonry-RC structures located in Lisbon with different structural configurations and masonry materials has been investigated. Then, behaviour factor and overstrength ratio are defined based on the nonlinear static sensitivity analysis, following the different methods proposed in EC8 [10] and in the related research (e.g. [11], [12]). For the sensitivity analysis, both types of uncertainties are considered, epistemic and aleatory in order to account, as much as possible, to different material and structural elements. On this way, more correct value of behaviour factor is assessed. After obtaining the values of the q-factor for each direction (X and Y), for two load patterns (uniform and triangular) and for all considered models, values are compiled and the final ones for the q-factor for the typology under study are proposed. The main results of performed study are presented and discussed in this paper. It should be mentioned that for the seismic evaluation of these buildings, all the steps of an interdisciplinary approach have been followed [13, 14].

2 Case studies and definition of uncertainties

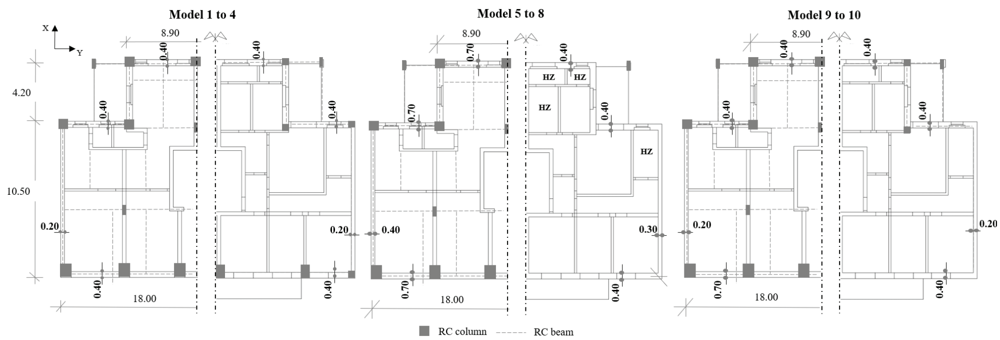
Mixed structure of masonry and RC exist all over the city of Lisbon, although they are predominant in streets or areas urbanized during the 30s and 40s. The detailed structural characterization as well as the data need for the study of the structural seismic behaviour of these buildings, are available in [14, 15, 16]. In this section, only the main data regarding the case studies (three different ones, "rectangular", "Rabo de Bacalhau" and "corner" type building) and uncertainties definition are provided.

2.1 First case study and considered uncertainties

The first case study (herein refer as well as first building class) corresponds to the buildings which are quite standardized in terms of material, geometry, number of floors and structural details [16, 14]. Namely, the high percentage of these buildings consists of rectangular shape in plan, three floors with constant storey height, with two flats per floor and residential area in the ground floor. Façade walls thickness is 0.5 m on the ground floor, while they are thinner at the upper levels (walls thickness on the last floor is 0.4 m); side walls are with the thickness of 0.5 m without openings, constant in height. Rubble stone masonry and hydraulic mortar characterize the exterior walls, whereas the interior walls were built mainly with hollow bricks and cement mortar. The part on the façades below the window was constructed with hollow brick with 0.15 m thickness. RC elements are placed on the external walls, which are strengthened (belted) on all floors by RC beams at the height of the window lintels with the thickness of the wall and 0.2 m in height; small RC lintels were found of each doorway. There are two types of floor



(a) 3D Model (left); plan (middle) and main façade (right)



(b) Position of the RC elements and thickness of the walls of ground and top floor (left and right of each plan) for Models 1 to 4 (left), 5 to 8 (middle) and 9 and 10 (right)

(c) 3D Model (left); plan (middle) and main façades (right)

Figure 1 Case studies: (a) "rectangular"; (b) "Rabo de Bacalhau"; (c) "corner" (dimensions in [m]) construction used in these buildings: timber floors in the rooms and concrete floors in the services areas (denoted as HZ in Figure 1(a)). Figure 1(a) illustrates the representative structures with rectangular shape in plan, where the block of two prototype buildings is chosen with the aim to consider the effect of adjacent buildings. These structures are characterized mainly with the shared side wall.

Regarding the uncertainties, two types are considered: aleatory (related to the mechanical parameters) and epistemic (related to the structural details). In terms of aleatory uncertainties, eleven variables are considered for the sensitivity analyses (2N+1, N is the number of variables or group of variables) – see details in Annex A. These variables include mechanical properties in terms of Young and shear modulus and compressive strength of rubble stone and hollow brick masonry (aleatory uncertainties X1 and X3, respectively) and shear strength of rubble stone and hollow brick masonry (X2 and X4), then the parameters which control the drift and strength decay of piers and spandrels, respectively (X5 and X6), the parameters which control the degradation for the initial elastic stiffness (X7), the parameters related to the stiffness of the timber and RC floor, respectively (X8 and X9), the parameters which control the connection between external walls (X10) and the parameters which control the different thickness of the RC slab (X11). To each variable, it is defined a plausible range of variation – a minimum, median and maximum value – used to the sensitivity analysis, where in total 23 models were defined (M1 to M23). The mechanical properties were defined based on the values from Italian standard (as initial parameters) and updated by using the Bayesian approach with the values obtained from experimental tests performed on the similar buildings. Detailed explanation about this procedure, and gravity and live loads considered in the model are presented in [14, 15].

One of the main vulnerabilities of this building's typology is the connection between exterior/exterior and exterior/interior walls and between walls and floors. Thus, epistemic uncertainties considered in this study are related to the connections between exterior/interior walls. Thus, two models were adopted: (i) model with bad connections between exterior/interior and intermediate connections between exterior/exterior walls (model A) and (ii) model with good connections between walls (model B). Connection between exterior/exterior walls and between walls and floors, were considered as aleatory uncertainties, i.e. X10 and X8/X9, respectively. As model A is considered the more representative and realistic, in this work only the q-factor for this model is provided (for model B, see [13]]. For information about modelling of connections refer to [15].

2.2 Second and third case studies and considered uncertainties

For the second ("Rabo de Bacalhau") and third ("corner") case studies (i.e. second and third building classes), the variety in terms of material, geometry and constructive details is higher than for the first case study, as clarified more in detail in the following. Specifically, from the constructive point of view, the main variations inside the "Rabo de Bacalhau" (marked as B1 in Figure 1(c, left) and "corner" (marked as B2 in Figure 1(c, left)) types can be summarized:

- Ground floor occupation: (a) commercial or (b) residential;
- Side walls solution: (a) shared or (b) not shared between adjacent buildings;
- Façade walls materials: (a) solid brick; (b) hollow brick and (c) rubble stone masonry. Independently of the type of material, cement mortar was used;
- Side walls material: (a) solid brick; (b) hollow brick; (c) rubble stone masonry; (d) cement block and (e) RC wall. For all materials, cement mortar was used;
- Floors type: (a) only RC slabs; (b) combination of RC slabs (denoted as HZ in Figure 1) and timber floors;
- Position of RC columns and beams: (a) all over the height of the structure on both façades (Figure 1(b, left)); (b) only in the ground floor as external frame (Figure 1(b, middle)) or (c) only on the back façade (Figure 1(b, right)); (d) some internal RC elements are placed only in the ground floor.

Based on the Building Regulation from 1930 [17], for interior walls, the hollow brick was used on the last two floors and solid bricks on the lower floors of the building. The thickness of such walls varies between 0.25 m and 0.15 m.

Based on all these variations, in total 10 possible models for each building type (B1 and B2, considered as case studies) are defined and presented in the logic-tree (Figure 2). Correspondingly, Figure 1(b) exemplify the position of RC elements, as well as the thicknesses of the walls for defined building models for ground and top floor [17]. Despite, these data are presented only for B1, the same thicknesses and position of RC elements correspond to the B2. In this study, only buildings with the ground floor for shops/commercial area were examined, due to the higher vulnerability when compared with buildings with residential ground floors.

As for rectangular buildings, connections between exterior/interior and exterior/exterior walls for the "Rabo de Bacalhau" and "corner" buildings were considered as bad and intermediate, respectively. It should be mentioned that for the models defined in the logic tree (Figure 2), i.e. "Rabo de Bacalhau" and "corner" buildings, aleatory uncertainties (described by random variables, see Section 2.1) were not considered and all models were run considering only median values for the mechanical parameters.

It is worth noting that in order to explicitly model the interaction effect among buildings, the whole aggregate, which consists of four buildings, is modelled (Figure 1(c)). Afterwards, with the aim of defining the behaviour of the building by taking into account adjacent buildings, only the results for two buildings (B1 and B2) are considered.

3 Analysis of the global behaviour

3.1 Modelling approach

Nonlinear static analyses are performed to assess the global behaviour of the structures. Pushover analyses were performed by considering each main direction (parallel and perpendicular to the façades) including positive and negative orientation.

As recommended in EC8 [10], NTC [18] and [19], two load patterns

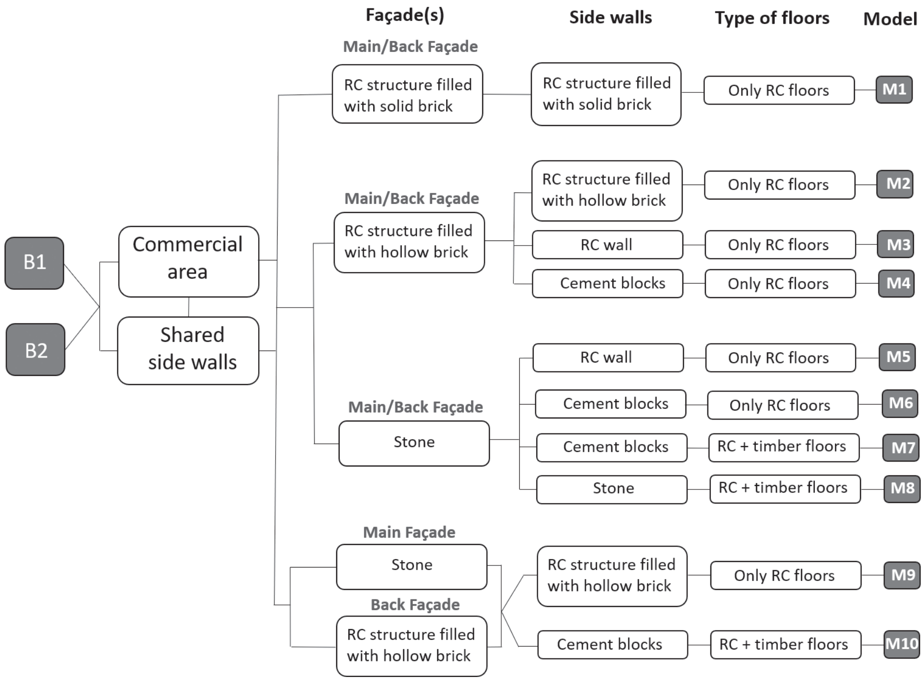


Figure 2 Logic-tree

(uniform, proportional to the mass, and triangular, proportional to the product between mass and height) are adopted. For the examined case studies, only the global seismic response is analysed, while the local flexural behaviour of floors and the out-of-plane walls' response are not explicitly computed due to the presence of RC ring beams which reduce the vulnerability to the out-of-plane failure modes of masonry walls.

To the aim to define the behaviour of the structure, three-dimensional models of the block of two and four buildings, for "rectangular" and "Rabo de Bacalhau"/"corner" shape in plan respectively, are defined in 3Muri [20] (used to generate the equivalent frame idealization of walls) and Tremuri [21], to perform the nonlinear analyses. The response of masonry panels (piers and spandrels) is described through nonlinear beams characterized by piecewise force-deformation constitutive law [22]. For detailed explanation of modelling and adopted criteria for strength refer to [15, 16].

It should be mentioned that results are presented only for the most representative case in terms of load pattern, as concluded in [15, 16]: (i) for the first building class, the triangular for the X direction (along the façades) and the uniform for Y direction (direction of side walls); (ii) for the second and third building classes, uniform is considered as more appropriate for both directions.

3.2 Definition of damage limit states and intensity measure

The definition of the Damage Levels (DLs) from the results of nonlinear static (pushover) analyses is a critical issue that is tackled in a different way in literature and codes.

In the presented study, four damage limit states (DLi, i = 1…4) are defined on the pushover curves according to the criteria proposed in [22] by correlating the behaviour of the structure at three scales (element, macroelement and global). It may be mentioned that reference is made to the attainment of damage levels 2, 3, and 4 assumed to correspond to damage levels defined in the part 3 of Eurocode 8 [23]. According to the multiscale approach, the DLi is defined by the minimum displacement threshold obtained from the verification of conventional limits at the three scales.

Among the possible choices for Intensity measure (IM_{DL}), Peak Ground Acceleration (PGA_{DL}) was selected that produces the attainment of specified damage states (DL_S).

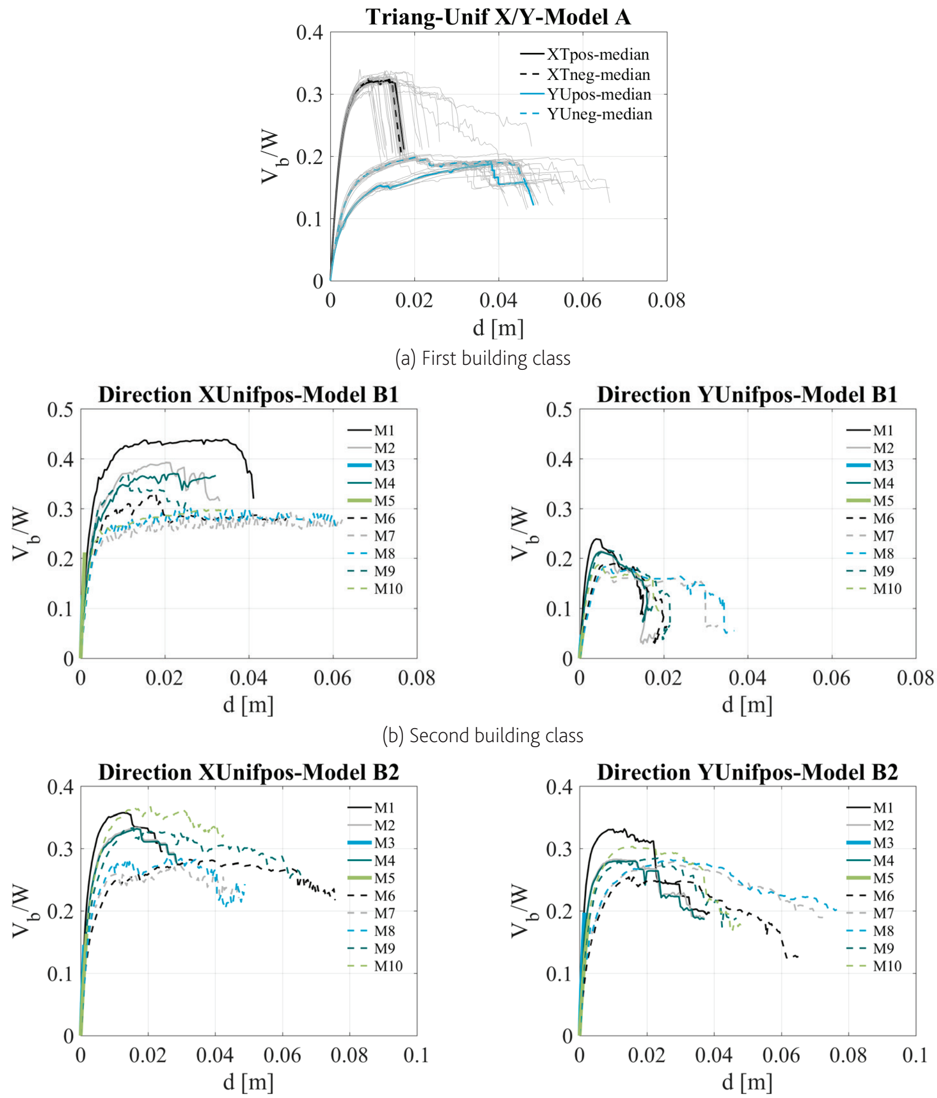
The Capacity Spectrum Method with overdamped spectrum is adopted to define the PGA. Detailed explanation about the definition of PGA is presented in [15, 16]. To calculate PGA_{DL}, the type of seismic action, type 1 (far-field, [24]) with PGA equal to 1.5 m/s^2 was adopted, since it corresponds to the more demanding in comparison with earthquake type 2 (near-field, [24]), as it was confirmed in [15, 16]. As concern the type of soil, types B (S = 1.29) and C (S = 1.5) are adopted, representing the types of soils for area of study.

3.3 Pushover curves

In this section, only the main results and conclusions are presented. For more detailed explanations refer to [16]. The resulting difference in terms of pushover curves between the adopted models for considered case studies, is plotted in Figure 3, considering directions X and Y and appropriate load patterns. Pushover curves are presented as the ratio between the base shear force (V_b) and the weight of the model (W), as a function of the average displacement of the roof weighted by the seismic modal mass of all nodes (d).

As mentioned, regarding the first building class, the values of mechanical parameters are varied between min-median-max values. In order to observe the variability of the behaviour of the structure, considering different values of mechanical parameters, next to the pushover curve obtained with median values of mechanical parameters (black and blue for X and Y direction, respectively), pushover curves obtained for all models (represented in grey colour) considered in the sensitivity analysis are also presented (Figure 3(a)). It is possible to observe that both, stiffness and base shear capacity, are higher in case of the X direction, whereas the higher ductility is obtained for Y direction due to the flexural behaviour (damage) of the walls in such direction. The comparison of the pushover curves obtained for the X direction (positive and negative) shows that the median curve is not so different in the two cases, since the building is quite symmetric in this direction, while in Y direction, the capacity slightly differs in terms of initial stiffness and maximum strength.

In general, for the second and third building classes (Figure 3(b)(c)), comparing the behaviour between the models defined above, it is concluded that M6, M7 and M8 are characterized with the lowest stiffness and strength, whereas the highest strength and stiffness are found in case of M1, M2 and M4 due to the material used for their exterior walls. For example, observing the second building class, Model M1 has respectively 3.4- and 1.6-times higher stiffness and strength in case of the Y direction in comparison to the M7. Comparing M1 and M9, around 1.2 times higher strength is obtained in case of M1 for both directions, while 1.2- and 1.9 times higher stiffness for X and Y direction, respectively.



(a) First building class

(b) Second building class

(c) Third building class

Figure 3 Pushover curves for considered case studies

Concerning the second building class, it is clearly noticed that the response in the X direction is characterized by higher strength and initial stiffness for all models due to the presence of side blind walls in this direction.

4 Structural behaviour factor

In general, the definition of factor q corresponds to the one semi-empirical value for a large group of structures, which does not always represent the real structural response. Thus, behaviour factor should be evaluated for a specific class and a particular structural building type. In the following the method used to evaluate behaviour factor in this study is explained briefly.

4.1 Procedures to define the behaviour factor

Based on the definition in Figure 4, where the resistance curve of an actual structure (pushover curve), idealized with ideal elastic-ideal plastic relationship, is compared to the response of equivalent ideal elastic one with the same initial stiffness characteristics, the behaviour factor is defined as:

$$q = \frac{V_{el,max}}{V_y} = q_0 \qquad (1)$$

where V_y corresponds to the strength of idealised bilinear system equivalent to the 'true' nonlinear behaviour and $V_{el,max}$ is the expected elastic strength capacity. The actual resistance (pushover) curve is idealized by mean of a bilinear approximation based on equal energy criterion. An equivalent initial stiffness is defined following the procedure in the NTC [18], as the secant stiffness in the first point of the seismic resistance curve attaining 70% ($V_{y,\,70\%}$) and 60% ($V_{y,\,60\%}$) of the maximum lateral strength. By adopting these two percentage (70% and 60%), it is observed how q-factor is influenced by the definition of V_y. The ultimate displacement (d_u) has been identified in correspondence with the value of DL4 defined by multilinear constitutive law. In this study this criterion refers to the first criterion.

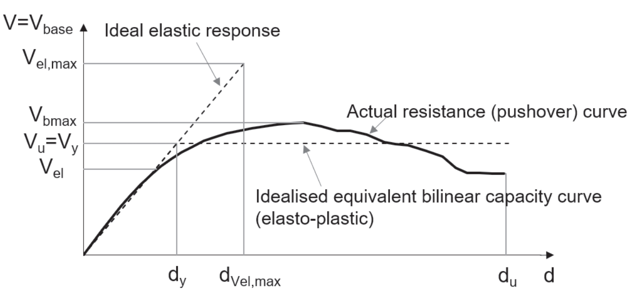


Figure 4 Parameters for the definition of the behaviour factor q

Since the elastic analysis methods do not take into consideration the redistribution of seismic loads after yielding of individual structural elements, the "ultimate" state of the structure, which corresponds to the attainment of the strength capacity in at least one structural element, is only approximation of the actual maximum resistance [25]. In fact, this state does not correspond to the ultimate strength of the system. The ultimate strength capacity (V_{max} or V_y) is reached at values higher than the base shear at which the element would reach its strength capacity according to linear elastic analysis (V_{el}) [25]. This is due to the limited (but existing) deformation capacity in the nonlinear regime, which is sufficient to allow the system to withstand increased seismic load. In this case, the reserve strength (overstrength), expressed in terms of overstrength ratio (OSR), results into increased value of behaviour factor. A correct definition of behaviour factor q would be:

$$q = \frac{V_{el,max}}{V_{el}} = \frac{V_{el,max}}{V_y}\frac{V_y}{V_{el}} = q_0\ OSR \qquad (2)$$

Moreover, the q-factor can be also defined as the ratio between the ground acceleration leading the structure to its ultimate limit state and the ground acceleration leading to the elastic limit (Criterion 2). Herein, the acceleration which corresponds to the ultimate limit state is related to the value of DL4 ($a_{g,DL4}$), whereas the value of DL2 is considered as the acceleration to the elastic limit ($a_{g,DL2}$). The q-factor (q_{ag}) would be as in Equation 3:

$$q_{ag} = \frac{a_{g.DL4}}{a_{g.DL2}} \qquad (3)$$

It is worth noting that DL1 could also be considered for the definition of the elastic limit of the acceleration; indeed, it is more similar with the concept of the first element that attains the nonlinear behaviour. In this study, DL2 was adopted as will lead to q-factor values on the safe side. For discussion about this issue refer to [13].

It should be noted that other approaches suggested for example by [2 and 26] are also appropriate for calculating the structural behaviour factor, but they are not examined in the present paper.

4.2 Evaluation of the behaviour factor for mixed masonry-RC buildings

In EC8-1, the ranges of values proposed for structural behaviour factor q, for unreinforced masonry, is between 1.5 and 2.5. Besides, the values for the structural behaviour factor were obtained also by other authors [7, 26] satisfying the range proposed by EC8-1 [10].

Even the use of values at the lower limits of proposal is recommended, the National Annexes may specify the values to be used in individual countries. Using advantage of the nonlinear static sensitivity analysis performed for the typology under study, an attempt has been made to propose the values for mixed masonry-RC buildings. The values of q-factors are defined from the pushover curves considering different sources of uncertainties (explained in Section 2) that influence the global seismic behaviour.

Figures 5 to 7 exemplify the values of behaviour factor for three building classes which belongs to the mixed masonry-RC buildings typology. It should be mentioned that q-factor was calculated for all models defined for each building class (see Section 2); however, herein only minimum and maximum values, as well as standard deviation are presented. Values of q-factor are presented only for the most representative cases in terms of load pattern, as previously defined.

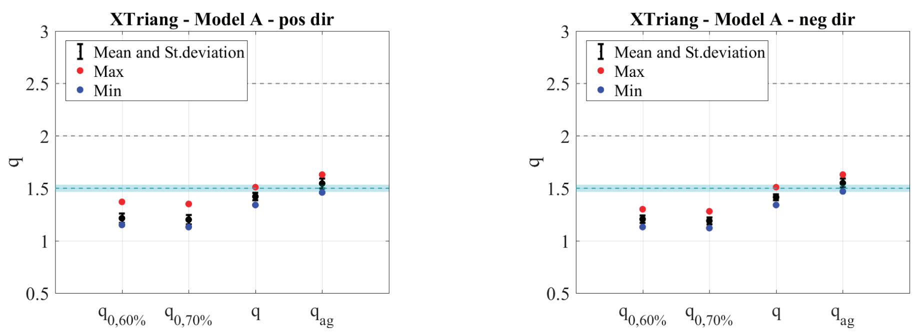
The obtained results in terms of q-factor, for three different classes of buildings indicate that different geometrical configurations ("rectangular", "Rabo de Bacalhau" and "corner" shape in plan), different masonry materials and structural system between these classes, influence (but not significantly in this case) the values of such factor. Explicitly, in the case of the third building class, the values are smaller than in the case of the first and second building class, particularly in the case of the Y direction (direction defined in Figure 1 (c, left)). However, if the values of the q-factor are evaluated with second criterion, the differences in factor *q*, evaluated for three building classes, are not significant. In terms of standard deviation, it can be seen that higher standard deviation is attained for the second and third building class, than for the first one. Actually, this was expected since, in case of the second and third building classes, adopted and analysed models (Figure 2) differentiate in terms of structural configuration, material and geometry for exterior walls, type of floors and position of the RC elements. On the other side, the first building class is more uniform and standardized, as defined above.

In general terms, the evaluation of the values of behaviour factor *q* on the basis of the ratio between accelerations (second criterion) resulted into smaller values than the evaluation on the basis of correlation of theoretical elastic and observed base shear responses (first criterion). This occurs particularly in Y direction for the first and second building class. This difference is due to the fact that in the first criterion the q-factor is calculated with the elastic force $V_{el,max}$ (defined for an elastic stiffness), while the values of the q-factor for the second criterion correspond to the stage of the buildings DL4 where the structural elements (side or façade walls) which mainly contributed for the global building's behaviour, in the *Y* direction, are damaged. Moreover, for the examined typology, the difference between the values calculated with $V_{y,60\%}$ and $V_{y,70\%}$ is irrelevant (Figures 5 to 7).

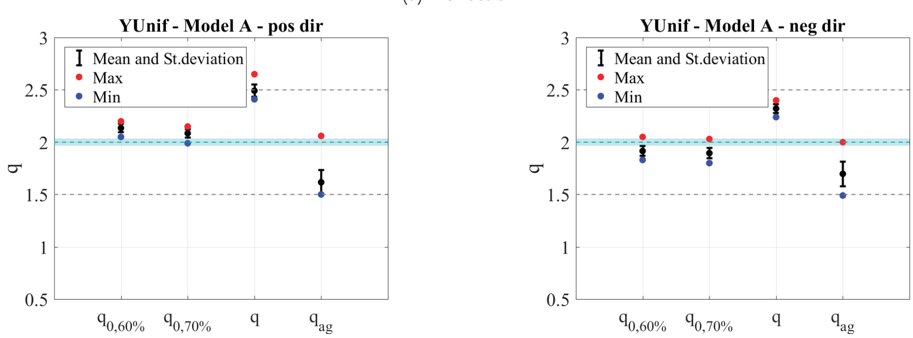
Regarding the first building class, the values of the standard deviation of the behaviour factor are greater in Y than in X direction, particularly in case of the second criterion. This is related to the different behaviour of the models analysed in the current study varying the mechanical parameters as abovementioned. In fact, the higher dispersion in terms of capacity was also obtained for Y direction [14].

Moreover, despite the differences observed in the behaviour between models considered as representative for the second and third building classes, i.e. M1 to M10 (see Figure 3), the range of values for behaviour factor is not significant. In fact, the maximum standard deviations of 0.25 and 0.17 are observed for second and third building class, respectively.

Lastly, the values proposed for behaviour factor for these building classes are: (i) for first building class the q-factor is equal to 1.5 in the direction of the façades and equal to 2 in case of the side walls; (ii) for the second building class, in the direction of the façades

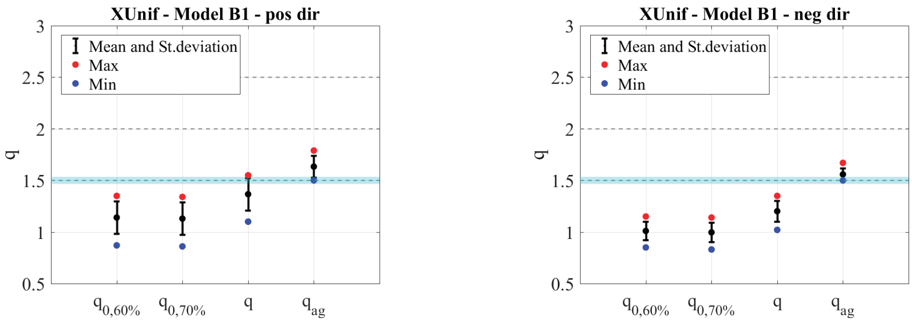


(a) X direction

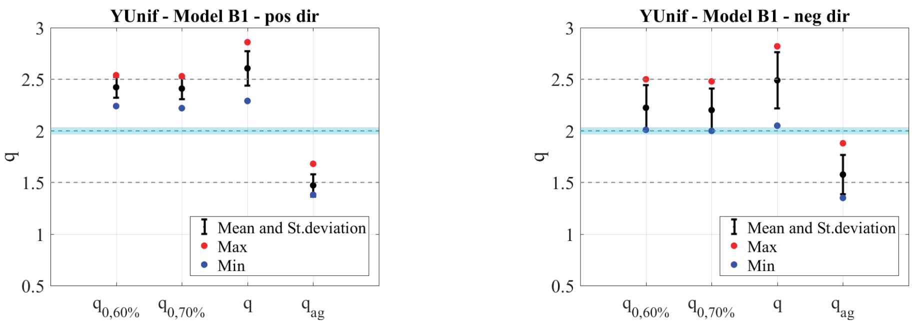


(b) Y direction

Figure 5 Mean, maximum and minimum values and standard deviation of the behaviour factor for first building class.Note: doted lines represent the values proposed by NTC; blue line represents the adopted value for q factor

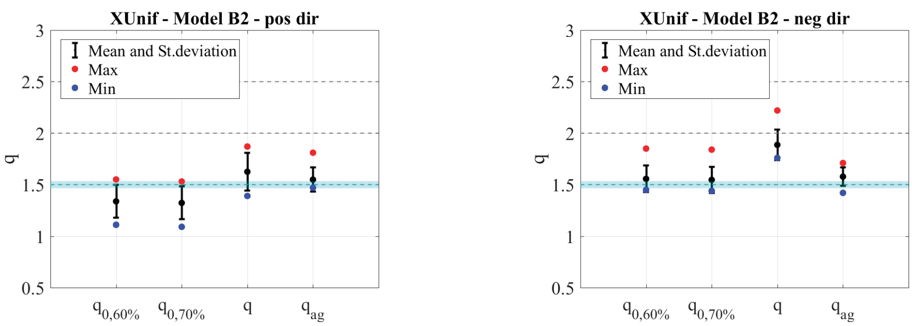


(a) X direction

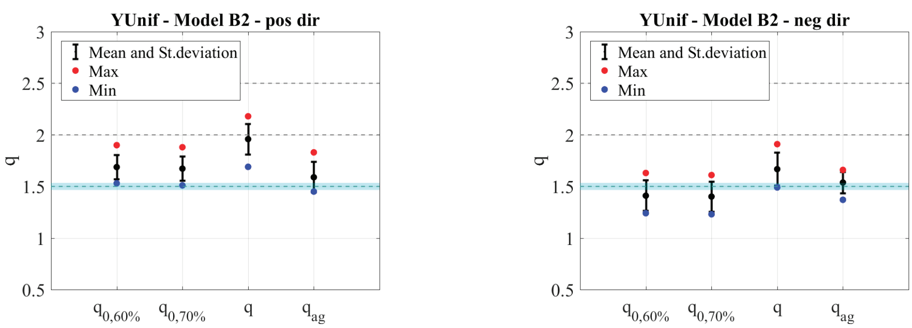


(b) Y direction

Figure 6 Mean, maximum and minimum values and standard deviation of the behaviour factor for second building class. Note: doted lines represent the values proposed by NTC; blue line represents the adopted value for q factor



(a) X direction

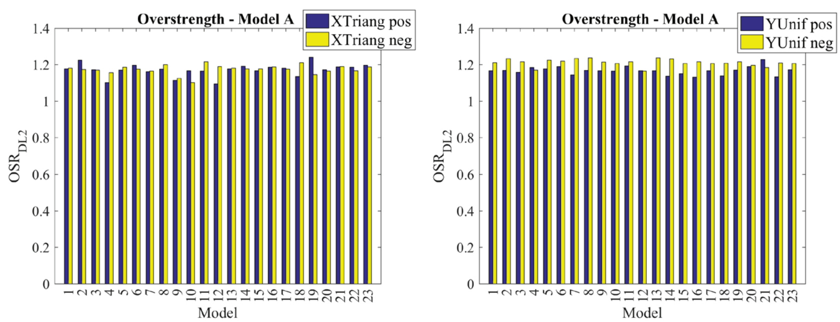


(b) Y direction

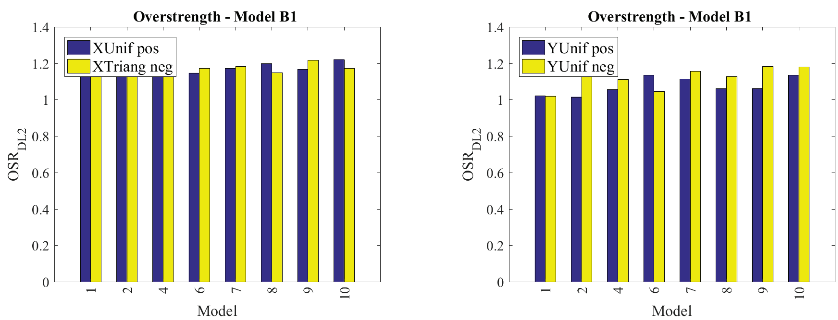
Figure 7 Mean, maximum and minimum values and standard deviation of the behaviour factor for third building class. Note: doted lines represent the values proposed by NTC; blue line represents the adopted value for q factor q-factor is equal to 2, while in the direction of the side walls q-factor is equal to 1.5; (iii) in case of the third building class the q-factor is equal to 1.5 in both directions.

Regarding the OSR, which depends on a series of factors varying from the structural configuration and associated redundancy to modelling assumptions [28], the current state of the art presents the different approaches for its evaluation. Namely, in [29, 30, 11], the OSR was evaluated numerically by nonlinear static analyses from nonlinear capacity curves, for several low-rise reinforced and unreinforced masonry buildings. Additionally, experimental evaluation of OSR have been also reported in the literature [5, 6, 7, 11, 25]. In this study, the availability of sufficiently reliable models for the nonlinear static sensitivity analyses of the most representative case studies allows to evaluate the OSR. Results are presented for first building class by considering all aleatory uncertainties (M1 to M23) and for all models defined by logic-tree in case of the second and third building classes (M1 to M10).

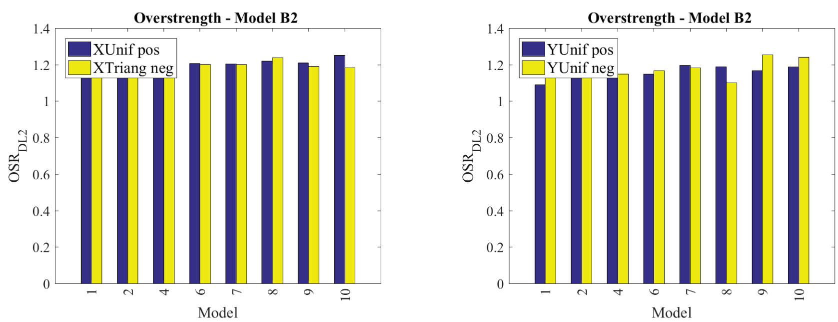
Figure 8 shows that based on analyses of three group of buildings which belong to the mixed masonry-RC structures, the values of overstrength factor is around of 1.2. As reported in [28], the unreinforced masonry structures are usually characterized by higher values of OSR. For example, it was reported that after twenty building configurations, the obtained range of OSR is wide, i.e. between 1.2 and 3.8. However, the structures under study are mixed masonry-RC buildings and all reinforced elements are characterized for weak concrete and very low ratios of vertical and transversal steel reinforcement which can decrease, in average, the typical values of OSR in unreinforced masonry buildings. Moreover, it can be noticed that different geometrical characteristics (shape of plan, different thickness of the walls) and variation of mechanical parameters of the materials do not affect the value of OSR in this typology. Thus, based on the obtained results, it is recommended to adopt an OSR = 1.2 for the mixed masonry-RC typology studied. Further research should be performed to consider the structures with different number of levels.



(a) First building class



(b) Second building class



(c) Third building class

Figure 8 Overstrength ratio for three building classes

5 Conclusions

Despite the significant progress in nonlinear methods of analyses of old building structures in the last decades, there is still considerable resistance to use nonlinear procedures in practical engineering offices. Thus, it would be important to define values for the behaviour factor (q) for each specific typology. Therefore, the main aim of this work is to contribute to the technical community and to attain a better insight on the value for behaviour factor for mixed masonry-RC buildings in Lisbon.

The results of nonlinear static sensitivity analyses have been used for the evaluation of values of structural behaviour factor (q factor) for these structures. Three building classes, represented by “rectangular”, “Rabo de Bacalhau” and “corner” shape in plan have been analysed, representing the most characteristic structures for this typology. The seismic behaviour of the buildings was evaluated by considering the aleatory and epistemic uncertainties. Then behaviour factor was defined by using two different criterions. Based on the obtained results, the value of behaviour factor was recommended for each building class and presented in the following:

- for first building class, the q-factor is equal to 1.5 in the direction of the façades and equal to 2 in case of the side walls;
- for the second building class, in the direction of the façades q-factor is equal to 2, while in the direction of the side walls q-factor is equal to 1.5;
- in case of the third building class the q-factor is equal to 1.5 in both directions.

As can be observed, the analysis of numerical results shows that the obtained values of structural behaviour factor are in the range with the values for unreinforced buildings proposed by EC8.

Moreover, as already mentioned, the definition of the behaviour factor must consider the overstrength ratio. Thus, this factor was also evaluated following the procedure based on nonlinear static sensitivity analyses and obtained value is equal to 1.2 for all building classes. Although this value is smaller than what was expected for masonry structures, the values of behaviour factor (q) are of the same order of magnitude as values proposed by codes and current state of the art.

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Annex A

In reference to Section 2.1, aleatory uncertainties used to perform nonlinear static sensitivity analysis and afterwards to define the behaviour factor and overstrength ratio, are defined in Table A.1.

Table A.1

Set Pk	Variable	X_{min}	X_{median}	X_{max}
X1	E [GPa]	0.69	0.82	0.98
	G [GPa]	0.23	0.27	0.33
	f_m [MPa]	2.07	2.33	2.63
X2	T_0 [MPa]	0.064	0.077	0.092
X3	E [GPa]	2.3	2.95	3.73
	G [GPa]	0.77	0.98	1.24
	f_m [MPa]	1.45	1.66	1.89
X4	T_0 [MPa]	0.24	0.28	0.32
X5	$\theta_{P,S3}/\theta_{P,S4}/\theta_{P,S5}$	0.0023/0.0039/0.0056	0.0029/0.0049/0.0069	0.0037/0.0061/0.0084
	$\theta_{P,F3}/\theta_{P,F4}/\theta_{P,F5}$	0.0046/0.0078/0.0120	0.0058/0.0098/0.0147	0.0074/0.012/0.01796
	$\beta_{P,S3}/\beta_{P,S4}/\beta_{P,F4}$	0.6/0.25/0.8	0.7/0.4/0.85	0.8/0.55/0.9
X6	$\theta_{S,S3}/\theta_{S,S4}/\theta_{S,S5}$	0.0015/0.0045/0.015	0.0019/0.0058/0.0194	0.0025/0.0075/0.025
	$\theta_{S,F3}/\theta_{S,F4}/\theta_{S,F5}$	0.0015/0.0045/0.015	0.0019/0.0058/0.0194	0.0025/0.0075/0.025
	$\beta_{S,S3}/\beta_{S,S4}/\beta_{S,F4}$	0.4/0.4/0.4	0.6/0.6/0.6	0.8/0.8/0.8
X7	$k_0 - k_{el}$	0.5 – 1.25	0.65 – 1.50	0.8 – 1.75
X8	G_{timber} [MPa]	6.136	9.88	15.91
X9	$G_{concrete}$ [MPa]	1208.3	3820.98	12083
X10	A [m^2]	0.001	0.000282843	0.00008
	I [m^4]	0.0005	0.000141421	0.00004
X11	p_{floor} [kN/m^2]	0.683	0.826	1

Legend:
E – Young Modulus; G – shear modulus; fm – compressive strength; T_0 – shear strength; $\theta(_{P, S/F})$ and $\beta(_{P, S/F})$ – drift and residual strength for piers; $\theta(_{S, S/F})$ and $\beta(_{S, S/F})$ – drift and residual strength for spandrels (shear (S) and flexural (F)); k_0 - value of the shear for which starts the degradation of stiffness, normalized to the ultimate shear and k_{el} - the ratio between the initial and the secant stiffness; $G_{eq,timber\ floor}$ and $G_{eq,RC\ floor}$ – equivalent shear modulus for timber and RC floor, respectively; A and I – area and moment of inertia of "equivalent" beam

Notes:
X1 and X2– rubble stone masonry; X3 and X4 – hollow brick masonry; X5 and X6 – drift and residual strength for piers and spandrels, respectively; X7 – degradation of the initial elastic stiffness; X8 and X9 – stiffness of the timber and reinforced concrete floor, respectively: represent the uncertainties of mechanical properties and the quality of wall-to-floor connection; X10 – connection between external walls; X11 – different thickness of the reinforced concrete slab (this uncertainty was applied by changing the masses of intermediate floors: permanent and accidental loads (factorized) p_{floor}).