Seismic assessment and retrofitting of *Pombalino* buildings by fragility curves

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15 WCEE
LISBOA 2012

SUMMARY:
The heritage value of the mixed wood-masonry 18th century Pombalino buildings of downtown Lisbon is recognized both nationally and internationally. The present paper focuses on the seismic assessment of a typical *Pombalino* building; seismic performances of different configurations (by considering also various retrofitting strategies) are compared by the definition of fragility curves and damage probability plots. The used software has been the structural program called Tremuri, which enables nonlinear static and dynamic analyses to be performed. Herein only nonlinear static analyses have been performed. The structure is modelled using non linear beams for masonry panels; in case of the internal walls (*frontal* walls) an original formulation has been developed in order to take into account their specific seismic behaviour. Seismic assessment has been evaluated by applying non-linear static procedures (in particular referring to the use of inelastic spectra as proposed in Eurocode 8). Finally, assuming a lognormal distribution probability function, fragility curves are obtained. The most important application of such curves is for seismic loss estimation studies. Furthermore, it has been evaluated the performance of the Pombalino buildings not only in its original state but also with some retrofitting schemes. Fragility curves and, consequently, the probability of reaching, or exceeding, the ultimate limit state are used in order to compare seismic performance of such different retrofitted schemes with the basic configuration.

Keywords: Pombalino buildings, retrofitting, fragility analysis, equivalent frame model

1. INTRODUCTION

The heritage value of the mixed wood-masonry 18th century *Pombalino* buildings in downtown Lisbon is recognized both nationally and internationally. In 1755 a catastrophic earthquake followed by a major tsunami struck the capital of Portugal causing severe damage to the city. The event completely destroyed the heart of the city, which was set on a valley area close to the river Tagus and is composed of a shallow layer of alluvial material. The disaster required an urgent solution. The Prime Minister at the time, Marquis of *Pombal*, was set in charge of rebuilding the city and restoring it back to normality as fast as possible. He delegated to a group of engineers the development of a structural solution that would guarantee the required seismic resistance of the buildings. Based on the know-how of that time and on the empirical knowledge gathered from the buildings that survived the earthquake a new type of construction was created, which is now generally referred to as *Pombalino* construction.

The work of Meireles and Bento (2010) was the first to test the *frontal* walls under static cyclic shear testing with imposed displacements, where a specific loading protocol was used and vertical loading applied to the specimen by four hydraulic jacks and rods. Therefore, the objective of the experimental work developed in the cited paper was, to obtain the hysteretic behavior of these *frontal* walls, by means of static cyclic shear testing with imposed displacements. These properties were used in developing an element for *frontal* walls, which was the scope of the paper Meireles *et al*. (2011) and is discussed also in the companion paper of this conference Meireles *et al*. (2012).

The formulation developed (implemented in a non linear beam) aims to reproduce the hysteretic shear response of *frontal* walls. The hysteresis model was developed based on a minimum number of path-following rules that can reproduce the response of the wall tested under general monotonic, cyclic or earthquake loading. It was constructed using a series of exponential functions and linear functions. The hysteresis rule is defined by 9 independent physical or mathematical parameters and incorporates stiffness and strength degradations and pinching effect. It is developed based on the experimental tests carried out (Meireles and Bento, 2010) and the parameters are calibrated by such results.

The modelling and the seismic assessment of a typical *Pombalino* building has been performed with a structural software where the previously described element has been incorporated. The program used is Tremuri which has been originally developed at the University of Genoa, starting from 2002 (Galasco *et al.* 2009). In particular, it works according to the equivalent frame approach and focuses only to the global building response (which is assumed to be governed only by the in-plane behaviour of walls). The program enables nonlinear static and dynamic analyses to be performed. The structure is modelled by using non-linear beams for the ordinary masonry panels and the abovementioned formulation for the *frontal* walls. By using Tremuri, pushover analyses were performed on different configurations of the examined building aimed to simulate the effect of various strengthening techniques. On basis of the capacity curves obtained, the seismic assessment has been evaluated by applying non-linear static procedures (in particular referring to the use of inelastic spectra as proposed in Eurocode 8). Finally, assuming a lognormal distribution probability function, fragility curves are obtained as well as damage probability plots in order to compare the effectiveness of different interventions analyzed.

2. THE EQUIVALENT FRAME MODEL OF THE EXAMINED POMBALINO BUILDING

The building that was chosen to be analysed in this study tries to replicate a typical *Pombalino* building. The building tipology and the definition of the typical *Pombalino* building that was developed is presented in Meireles *et al*. (2012) in detail. In this paper only some additional figures are presented.

A 3D view of the model of the building as it is shown in Tremuri is presented in Fig. 2.1. Herein, it is represented in grey the parts of the structure that are composed of rubble masonry; in purple the parts of the structure that are composed of stone masonry; in green (dark and light depending on the size) are the *frontal* walls and in light brown are the timber beams connecting the *frontal* walls. Moreover the alignments of the different structural vertical elements in the plan view of the building are illustrated in Fig.2.1.; they have been identified starting from buiding plans described in detail in Meireles *et al*. (2012).

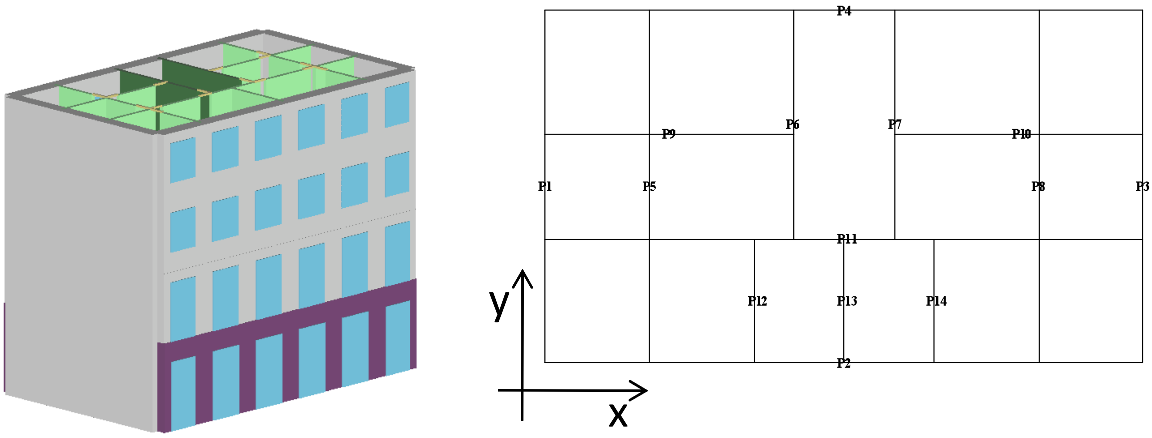

Figure 2.1. 3D model of the building (left); Numbers of the alignments of the different structural elements in the plan view of the building (right)

According to the modelling approach which Tremuri is based on, each wall is discretized by a set of masonry panels (*piers* and *spandrels*), in which the non-linear response is concentrated, connected by a rigid area (*nodes*). The equivalent frame idealization of the front and back façades is illustrated in Fig. 2.2. and Fig. 2.3., respectively. In particular, in red are the piers; in green are the spandrels and in light blue are the parts of the façade where no damage is foreseen (rigid nodes). A non-linear beam idealization is assumed for all ordinary masonry panels: thus the response is directly faced in terms of stiffness, strength and ultimate displacement capacity by assuming a proper shear-drift relationship.

In particular, the ultimate strength is computed according to some simplified criteria which are consistent with the most common ones proposed in the literature and codes (e.g. in Eurocode 8 2005 and in the Italian Code for Structural Design 2008) for the prediction of the masonry panels strength as a function of different failure modes which may occur (such as Rocking, Crushing, Bed Joint Sliding and Diagonal Cracking). The failure of the panel is checked in terms of drift limit values differentiated as a function of the prevailing failure mode occurred (if shear or flexural one). *Frontal* walls are instead modelled according to the formulation described in detail in Meireles *et al*. (2012).

Floor elements are modelled as orthotropic membrane finite elements, in particular: normal stiffness provides a link between piers of a wall, influencing the axial force on spandrels; shear stiffness influences the horizontal force transferred among the walls, both in linear and non-linear phases. For further details on Tremuri model see Galasco *et al.* (2004) and Lagomarsino and Cattari (2009).

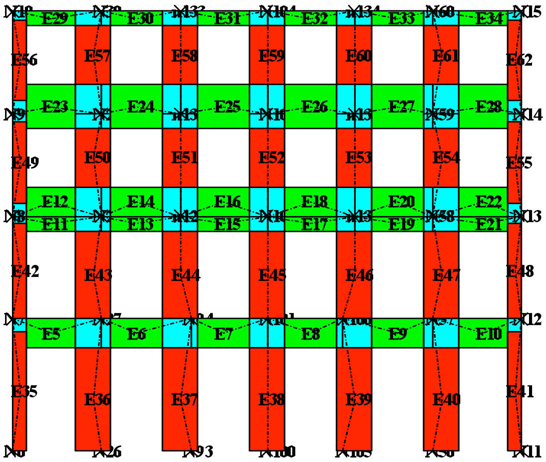

Figure 2.2. Equivalent frame idealization of front façade (P2)

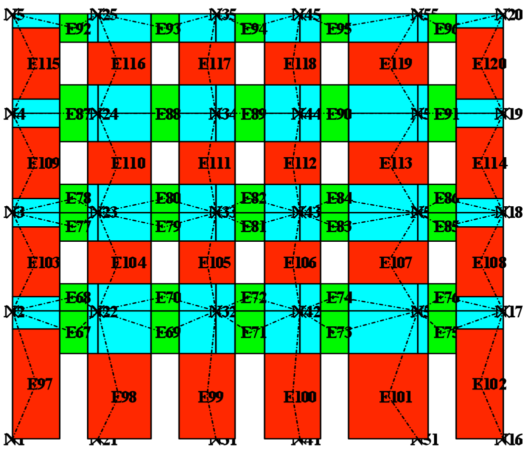

Figure 2.3. Equivalent frame idealization of back façade (P4)

3. STRENGTHENING SOLUTIONS

The basic configuration of the typical *Pombalino* building that has been analyzed is characterized by quite flexible wooden floors (not able to provide a satisfying seismic load redistribution among masonry walls in non-linear phase) and weak spandrels (without any tensile resistant element coupled to them). Pushover analyses perfomed on this basic configuration – as discussed in more detail in Meireles *et al*. (2012) - showed a significant difference between the seismic capacity of the buiding in xx and yy directions, in particular: the stiffness and strength is much higher in the yy direction than in the xx direction; but on the other hand, the ductility of the system is much higher on the xx direction and is practically non-existing in the yy direction. Due to the configuration of ground floor, a soft storey failure mechanism has been stressed. As known, all the above mentioned aspects - strength, stiffness and ductility - play a fundamental role in the seismic assessment and neither of two directions seems provide an effective system against the earthquake.

Due to this, the following retrofitting schemes have been proposed and analyzed:

1. Increase the in-plane stiffness of floors (transforming flexible floors into rigid floors);
2. Increase the in-plane stiffness of floors plus reinforcement of the five ground floor pillars;
3. Increase the in-plane stiffness of floors plus inclusion of four shear walls on the ground floor;
4. Increase the in-plane stiffness of floors plus inclusion of eight steel frames on the ground floor;
5. Increase the in-plane stiffness of floors plus inclusion of tie-rods at front and back façades.

The first one is the most evident and the one that will be seen to be the most effective and crucial improvement to the structure. The second one will be seen to bring not too much additional improvements to the seismic behaviour of the structure and the last three are seen to be added improvements to the structure if one desires to increase even more the earthquake resistance of the building. Paragraphs from 3.1 to 3.5 describe in more detail the abovementioned solutions; then, their effectiveness is compared with respect to the seismic demand proposed for Portugal in Eurocode 8 (2005) by introducing fragility curves (as discussed at paragraph 6).

3.1. Increase the in-plane stiffness of floors

Traditional timber floors are typically flexible. The increase of the in-plane stiffness of floors is an evident and most effective method of improving the seismic behaviour of old masonry structures. This is mainly because the increase of in-plane stiffness of floors enables the horizontal forces to be redistributed between the failing walls to the adjacent remaining walls and the structure behaves like a box. This could be obtained not only by fully replacing the original timber floors by more rigid structural solutions (such as the r. c. one, as often adopted) but – as preferable –by adopting more compatible solutions which also agree to conservation principle (e.g. by inserting new layer of wood planks and by guaranteeing a proper connection among different elements). For example, a discussion on the in plane stiffness of timber floor is illustrated in Brignola *et al.* (2009). In order to model the increase of in-plane stiffness of floors, the shear stiffness of the floors was increased by an order of 100.

3.2. Increase the in-plane stiffness of floors plus reinforcement of the five ground floor pillars

The pillars of the ground floor are a sensitive part of the masonry structure. One measure that was thought was to implement the reinforcement of the pillars on the ground floor as an additional measure, keeping the in-plane stiffening of the floors. To model this strengthened solution it was multiplied by a factor of 1.4 the stiffness and strength of the masonry associated to the pillars (starting from values described in Meireles *et al*. (2012). This factor comes from the Italian normative (Italian Technical Code, 2008; Circolare 2 febbraio, 2009, n. 617, table C8A2.2).

3.3. Increase the in-plane stiffness of floors plus inclusion of four shear walls on the ground floor

The inclusion of shear walls is a typical procedure for improving the seismic resistance of a building. It was decided to model the inclusion of four shear walls on the ground floor according to the scheme presented in Fig. 3.1. (shear walls in light blue). The shear walls have 48 cm in thickness and are composed of brick masonry. It was decided to place the shear walls only in the xx direction (direction of the façade and back walls, which has 18 m in length) since this direction is the most vulnerable one and is the weakest direction.

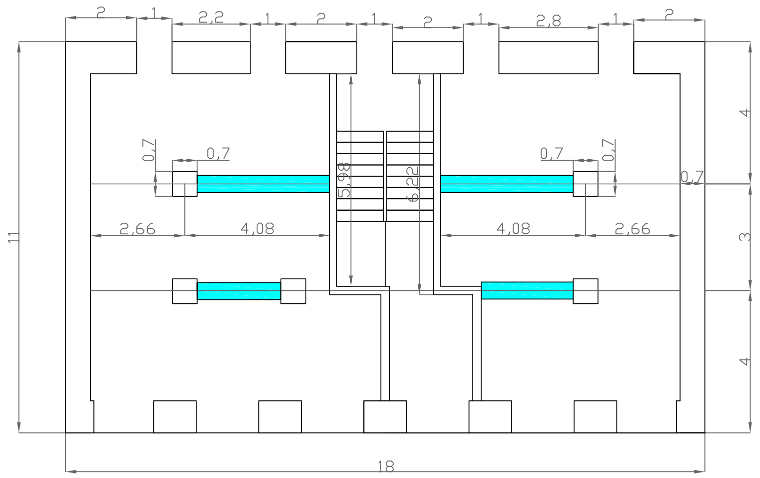

Figure 3.1. Positioning of the four shear walls on the ground floor – units in metres

3.4. Increase the in-plane stiffness of floors plus inclusion of eight steel frames on the ground floor

The inclusion of eight steel frames on the ground floor comes from the idea that including shear walls with no openings at ground floor is not a very much welcoming idea from an architectural and functionality point of view. The ground floors of these buildings are often used as restaurants, cafés or stores facilities and the inclusion of shear walls here is not very convenient from the point of view of the owners. The eight steel frames (pillars and beams) are each one composed of four HEA140 cross sections. Again, it was decided to place the steel frames only in the xx direction since this direction is the most vulnerable one and is the weakest direction. It was decided to model the inclusion of eight steel frames on the ground floor according to the scheme presented in Fig. 3.2. (eight steel frames in pink).

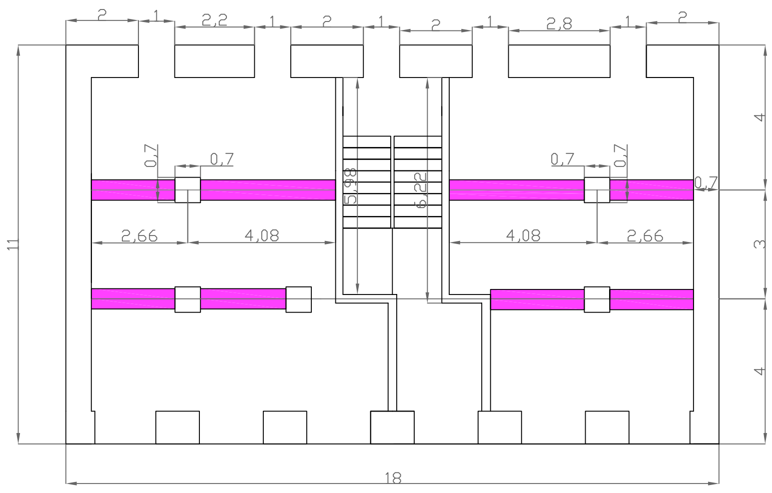

Figure 3.2. Positioning of the eight steel frames on the ground floor – units in metres

3.5. Increase the in-plane stiffness of floors plus inclusion of tie-rods at front and back façades

The input file of the software was prepared for the case of tie-rods at front and back façades. The tie-rods are placed at the top of the piers (placed along the spandrels), connecting the piers between each other. They are pre-stressed, pre-stressing the spandrels. The tie-rods are of 2,4 cm in diameter and composed of steel. An initial strain of 20% the yielding strain of the steel (0,02) was used. The tie-rods were only placed on the xx direction, the most vulnerable one and the only one with openings.

4. SEISMIC DEMAND

The seismic action for the downtown area of Lisbon is here presented. In Portugal, for the design and assessment of structures one must consider two types of seismic actions:

- Seismic action type 1 corresponding to a scenario of faraway earthquake;
- Seismic action type 2 corresponding to a scenario of nearby earthquake.

For each seismic action it should be selected a seismic zone depending on where our structure is located. For Lisbon city, and for normal residential buildings, the seismic zone 3 is defined for seismic action type 1 with design ground acceleration on soil type A (ag) of 1.5 m/s^2 and for seismic action type 2, seismic zone 1 is chosen with ag=1.7 m/s^2. The parameters for defining the configurations of the response spectra for the two types of seismic action, the referred seismic zones and for ground type C are presented on Table 4.1. and Table 4.2. (National Annex of Eurocode 8, 2004, version September 2007). The defined ground types for Lisbon downtown (Meireles and Bento, 2008). were A and C according to Eurocode 8 (2004) table. The ground type C was chosen since it is the most demanding situation corresponding to Lisbon downtown soil type.

Table 4.1. Elastic response spectrum parameters for seismic action type 1, seismic zone 3 and ground type C

Seismic action type 1 and seismic zone 3 -> *ag*=1.5 m/s^2				
Ground type	**S**	**T_B (s)**	**T_C (s)**	**T_D (s)**
C	1.5	0.1	0.6	2

Table 4.2. Elastic response spectrum parameters for seismic action type 2, seismic zone 1 and ground type C

Seismic action type 2 and seismic zone 1 -> ag=1.7 m/s^2				
Ground type	**S**	**T_B (s)**	**T_C (s)**	**T_D (s)**
C	1.5	0.1	0.25	2

The corresponding elastic response spectra for Lisbon downtown can be seen in Fig. 4.1. for the two types of seismic action.

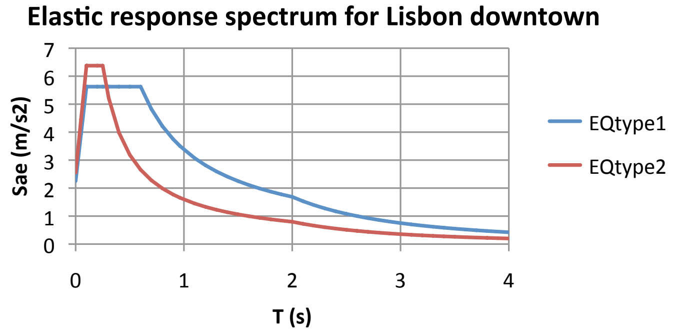

Figure 4.1. Elastic response spectra for Lisbon downtown
(Legend: EQtype1: Seismic action type 1; EQtype2: Seismic action type 2)

It can be seen that, for most of the range of periods of the structure, the seismic action type 1 is the most demanding but, for the range of low values of the period (high frequencies), the seismic action type 2 is seen to be more demanding. The two seismic actions must be considered in the subsequent analyses performed.

5. CAPACITY CURVES

Pushover analyses were carried out for both xx and yy directions and for the lateral load pattern proportional to the mass (uniform) for each configuration examined. In order to have a better perspective of the benefits that each of the retrofitting strategies has, a graph was defined, comparing the capacity curves for all the studied cases. Fig. 5.1. and Fig. 5.2. show the comparison of the pushover curves for the xx and yy directions, respectively.

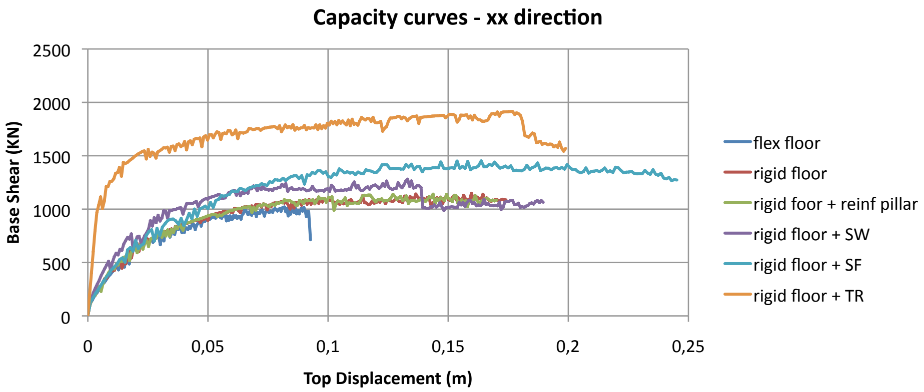

Figure 5.1. Pushover curves comparison in the xx direction

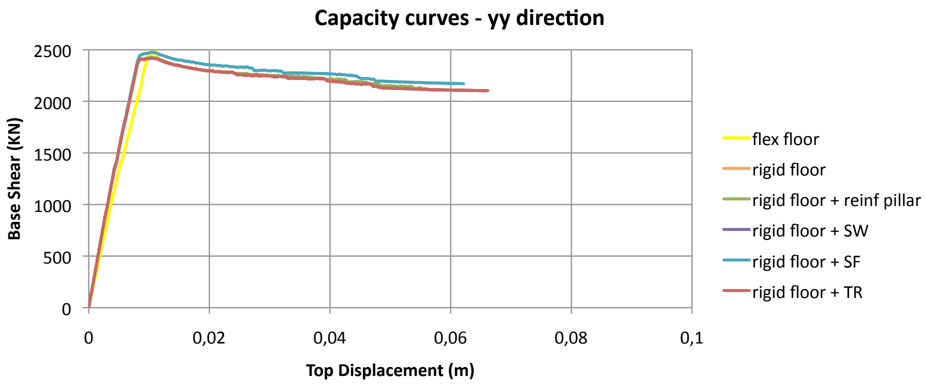

Figure 5.2. Pushover curves comparison in the yy direction

Based on the results obtained it is evident that the stiffness and strength is higher in the yy direction than in the xx direction. In the xx direction, going from the flexible floor situation ("flex floor") to the rigid floor situation ("rigid floor") one has an increase on the ultimate displacement, while the initial stiffness and strength are the same. Moving from the case of rigid floor to the case of rigid floor with reinforced pillars ("rigid floor + reinf pillar") brings no additional increase in ultimate displacement, stiffness or strength. Going from the case of rigid floor to the case of rigid floor with shear walls ("rigid floor + SW") one can observe that an increase of stiffness and strength is obtained. Moving from the situation of rigid floor to the case of rigid floor plus steel frames ("rigid floor + SF") one observes that the initial stiffness is maintained but the strength is increased and also the ultimate displacement. Finally, going from the situation of rigid floor only to the situation of rigid floor with tie-rods ("rigid floor + TR") a significant increase on the initial stiffness is reached as well as on the strength value, approximating the capacity curve in the xx direction to the values obtained for the yy direction. In the yy direction one can observe that all the curves are the same expect the curve of flexible floor since the only intervention influencing the yy direction has been the stiffening of the floors. All the other strengthen solutions have focused only on the xx direction.

Starting from pushover curves representative of the original multi degree of freedom, they have been properly converted into equivalent SDOF systems; to this aim, the criteria adopted in Eurocode 8 (part 3 -2005) have been assumed as reference.

6. FRAGILITY ANALYSIS

6.1. Definition of the damage limit states

Damage limit states have been established. The damage scale used in this work includes four levels of damage (plus the case of no damage): slight damage (1), moderate damage (2), heavy damage (3) and collapse (4). Damage limit states $S_{d,k}$ (k=1 to 4) are directly identified on the capacity diagrams in AD format as a function of the yielding displacement, S_{dy}, and the ultimate displacement, S_{du} (Eqn 6.1.). These are based on the proposal present in Lagomarsino and Giovinazzi (2006):

$$\begin{aligned} S_{d,1} &= 0.7 S_{dy} \\ S_{d,2} &= 1.5 S_{dy} \\ S_{d,3} &= 0.5\left(S_{dy} + S_{du}\right) \\ S_{d,4} &= S_{du} \end{aligned} \tag{6.1.}$$

Slight damage (1) indicates a condition still far from the reaching of the maximum strength and corresponds to local damage in few structural elements. Moderate damage (2) corresponds to the maximum value of the restoring force in the pushover curve, and is located, in terms of spectral displacement, after the yielding condition of the equivalent bilinear. Collapse (4) is defined on the basis of the ultimate displacement conditions for structural walls. Finally, heavy damage (3) lies in an intermediate position between moderate damage and collapse.

6.2. Fragility curves and damage probability plots

In order to compare the seismic performances of the different analyzed configurations, once the seismic input is fixed as reference (see paragraph 5), the probability of exceeding the ultimate limit state was evaluated by using the fragility curve concept. In particular, the conditional probability P [*ds* | S_d] of being in, or exceeding, a particular damage state (*d*), given the spectral displacement S_d, is defined by the following expression:

$$P\left[ds \mid S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{\overline{S}_{d,ds}}\right)\right] \tag{6.2.}$$

where: Φ is the standard normal cumulative distribution function; β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state *ds*; $\overline{S}_{d,ds}$ is the median value of spectral displacement at which a building reaches the threshold of damage state *ds*. Starting from spectral displacement (S_d) it is easy passing to a_g values. The values for the various uncertainties summarized by β_{ds} parameter are defined in Table 6.1. for each damage limit state.The uncertainties are various such as the uncertainty in the software model used (β_ε), the uncertainty associated to the variability of the input parameters (β_C), the uncertainty associated to the variability of the seismic action (β_D) and the uncertainty in the definition of the limit states (β_{LS}). The values of the β_ε, the β_C and the β_{LS} were all taken from the research paper of Pagnini *et al.* (2011). The β_D was conventionally assumed given not enough information was available on Eurocode 8 for a more precise estimation (e.g. related to the input definition for different percentile values); in general, in the case of a more detailed seismic hazard analysis, a wider scatter would be expected. Fig.6.1. shows the resulting fragility curve for the case of the original building for earthquake type 1 in the xx direction.

Table 6.1. Values of the various uncertainties for each damage limit state

β_{ds} contributions	damage limit state 1	2	3	4
β_{ε}	0.25	0.25	0.25	0.25
β_C	0.35	0.35	0.37	0.38
β_D	0.20	0.20	0.20	0.20
β_{LS}	0.24	0.26	0.18	0.14
$\beta_{ds} = \sqrt{\beta_{\varepsilon}^2 + \beta_C^2 + \beta_D^2 + \beta_{LS}^2}$	0.53	0.54	0.51	0.49

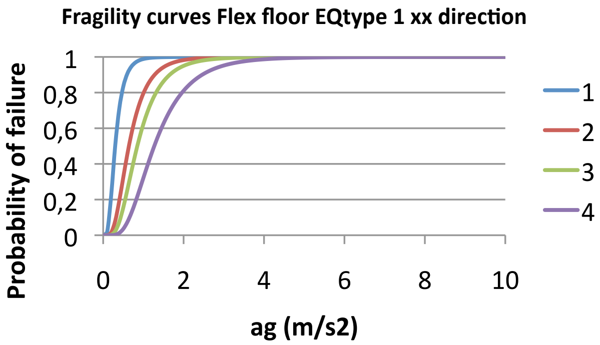

Figure 6.1. Fragility curve for earthquake type 1 in the xx direction for original building

The damage probability plots have been set all together for comparison purposes. Fig. 6.2. represents the probability of damage for earthquake type 1 in the xx direction for all the studied cases. In this figure P0 represents the case of having "no damage", P1 the probability of having "slight damage", P2 the probability of having "moderate damage", P3 the probability of having "heavy damage" while P4 the case of reaching "collapse".

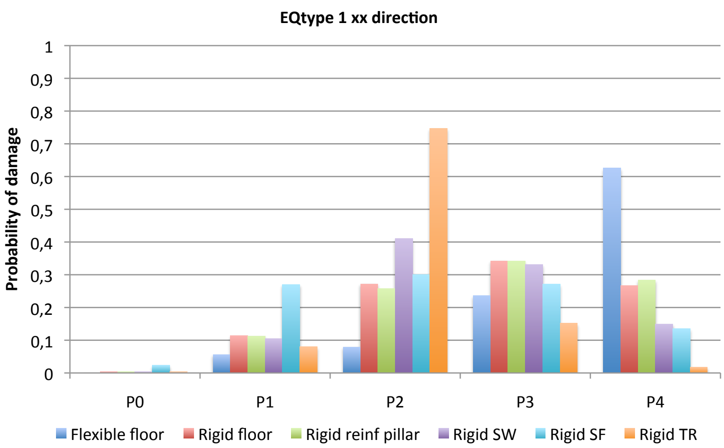

Figure 6.2. Probability of damage for earthquake type 1 in the xx direction

Based on the results obtained, it is clear that building without retrofitting presents the highest value of probability of damage P4 (collapse). Retrofitting the building by stiffening the floors enables reducing this value significantly. Retrofitting the building by stiffening the floors and reinforcing the five ground floor pillars does not improve any further the situation. Retrofitting the building by stiffening the floors and including shear walls or steel frames does improve the slightly the situation, reducing the value of P4 and spreading it more through P3 to P1. The retrofitting scheme that mostly improves the seismic performance of the building, with respect to the previous cases, is the case of the inclusion of tie-rods in the front and back façades. This reduces significantly the damage probability P4. Nevertheless, this retrofitting possibility seems to increase very much the damage probability P2 when compared to the other retrofitting strategies.

7. CONCLUSIONS

The building in its original state is very susceptible. It is thought to be because the floors are flexible. Simply by stiffening the floors one is able to have an improved structure. Additional retrofitting of the structure is possible and advisable if one wants to increase its resistance towards earthquakes. The most profitable solution (which is also the most complicated in terms of implementation) is the inclusion of tie-rods in the lintels at front and back façades. The solution of inclusion of shear walls is advisable also but has architectural drawbacks; the solution of the inclusion of steel frames in the ground floor is advisable and has not much architectural drawbacks. The solution of the reinforcement of the five ground floor pillars is seen to bring no additional benefit to the structure and is thus not recommended. It is possible to further ahead in the research conduct loss estimation studies with the results obtained. The current trend in seismic risk analysis and loss estimation involves the use of fragility curves derived from nonlinear static and dynamic analyses of representative structures. It is also possible, as a further research step, to improve the estimation of the beta coefficients with the response surface method.

AKCNOWLEDGEMENT
The authors would like to acknowledge the financial support of the Portuguese Foundation for Science and Technology (Ministry of Science and Technology of Portugal) through the research project PTDC/ECM/100872/2008 and through the PhD scholarship SFRH/BD/41710/2007.

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