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VULNERABILITY OF BUILDINGS IN HISTORIC TOWN CENTRES: A LIMIT-STATE APPROACH

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ABSTRACT

The paper presents a study carried out in the Alfama district of Lisbon, Portugal, funded by the EEC Environment Programme, DGXII Climatology and Natural Hazard, (project EV5V - CT93-0503). The aim is to identify retrofitting strategies which, while improving the seismic performance of historic buildings will not significantly alter their appearance. A street survey and the historical seismicity of the area represent the data on which the vulnerability assessment is based. Geometrical parameters and traditional building techniques are used to identify some idealised failure mechanisms and their distribution, leading to the definition of a vulnerability curve. This is compared with curves for masonry construction world-wide, as derived from The Martin Centre's database on earthquake damage. The results are mapped using a GIS application.

KEYWORDS

Earthquake Vulnerability, Historic Buildings, Limit-state Analysis, Strengthening and Upgrading

BACKGROUND

Buildings of historic town centres, for their age and stratification, represent a peculiar class of construction, whose interconnected structural behaviour has to be studied to define their vulnerability in earthquake-prone areas. Depending on the size of the sample and the aim of the study, the vulnerability can be assessed at different levels, involving different costs and yielding different types of evaluation. In the case of historic town centres, however, it is difficult to establish a one-to-one relationship between type of building, intensity of earthquake and type of damage, for a number of reasons:

- the single building would rarely comply with structural standards, for the vernacular quality of the fabric and its evolution in time, making any generalised assumption on structural behaviour rather unreliable;
- the high level of interconnection between adjacent buildings makes it difficult to identify and isolate structural elements and to correlate seismic input and levels of damage;
- the street survey can be misleading because both damage evidence and strengthening works might have been hidden by subsequent refurbishment.

It therefore becomes essential to couple the usual vulnerability assessment techniques, which in the case of r.c. rely on a century of codified practice, with the information that can be drawn from historical records on local building techniques, historical seismicity and recorded damage, and expertise acquired by professionals in the field of structural masonry. The approach adopted in the present study uses the data collected by a street survey of 200 buildings, theoretically leading to a Level 1 vulnerability assessment (i.e. classification of buildings according to some commonly recognised vulnerability indicators), to apply concepts typical of structural assessment techniques such as limit-state analysis. This approach allows the survey and structural treatment of specific constructional devices, highly influential on the aseismic behaviour of the building stock.

The underlying aim of the project is to develop more effective and less obtrusive ways of upgrading historic residential buildings in order to save life while at the same time preserving their value; and to provide a reference document to be used by local authorities in devising and planning conservation policies. The project is carried out by five European teams on four sites. The collection and analysis of the data obtained from the survey in Lisbon, have been discussed elsewhere (D'Ayala, *et al.* 1995). Here we present the methodology applied to define the vulnerability and the expected improvement obtained by simple upgrading work.

IDENTIFICATION OF COLLAPSE MECHANISMS

For the Lisbon site two classes of out-of-plane (OP) and in-plane (IP) mechanisms are considered and a value of equivalent shear capacity is calculated for each building in terms of A/g, where A is an equivalent static horizontal acceleration applied to the centre of gravity of the body under study. For each building, in relation to its position within the block, a two dimensional analysis is carried out relative to the most vulnerable wall. This is in most cases the facade of the building, which is often the main bearing wall. For buildings on corners or with more than one free wall, the analysis considers that with the highest percentage of opening surface.
Due to the assumptions of limit-state analysis to define the equivalent shear capacity only geometry and boundary conditions data are necessary, while the masonry, which cannot be reliably assessed with this level of survey, is taken as a perfect rigid-plastic material. This implies no elastic response amplification.

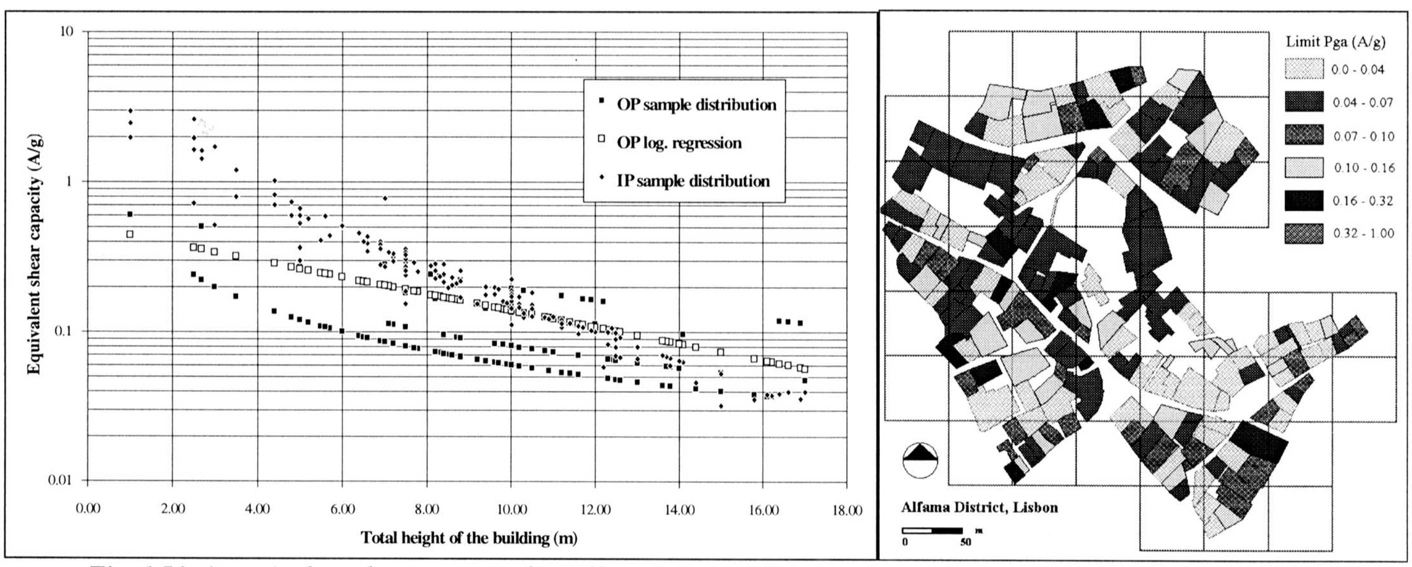


Fig. 1 Limit equivalent shear capacity for different classes of mechanism and map distribution over the sample

Fig. 1 shows the relationship between A/g and the building's height for the two classes of mechanisms. For the out-of-plane class of mechanisms, the parameters on which the limit-state analysis is based are the height of the building, an estimated average effective thickness. Four different cases are considered in relation to the boundary conditions of the wall:

a) OP1, no connection with the orthogonal walls or the floor structural system, corresponding to the structural scheme of a vertical cantilever (see Fig. 2a).
b) OP2, ties at the top floor connecting the facade to either the floor structural system or the lateral walls. This is a conservative idealisation, because in general for buildings with more than 3 storeys ties will be present on intermediate floors, representing additional restraint. A unit width of the wall is assumed over which the tie is effective. The critical value of A/g is calculated by optimising the position of the hinge which will form along the span for the mechanism illustrated in Fig. 2b (Giuffre' *et al.*, 1993).

c) OP3, quoins connecting the facade with the lateral walls over the entire height. This is equivalent to a reduction of the free deflection horizontal width which can be computed as a proportional increase in the restoring moment (Spence *et al.*, 1992a). The resulting mechanism is qualitatively drawn in Fig. 2c.

d) OP4, connection by ties and quoins simultaneously. This is a fairly common configuration in the sample. The resulting mechanism can be obtained as a combination of the two previous ones.

For each facade, having considered a scheme of assembly of rigid blocks (Fig. 2d), also the in plane mechanism, (IP), and its ultimate load factor are computed with a linear programming procedure elaborated elsewhere (D'Ayala 1993), taking into account the influence of width of piers, inter-storey height, percentage of opening and height/width ratio.

The geometric parameter values are based on visual estimates. The verification with a number of surveyed buildings shows a mean error of 5%. The value of the A/g function, however, is only affected by an error of 0.005 g for a 10% difference in height, for instance, well within the margins of precision of macroseismic ground motion evaluation.

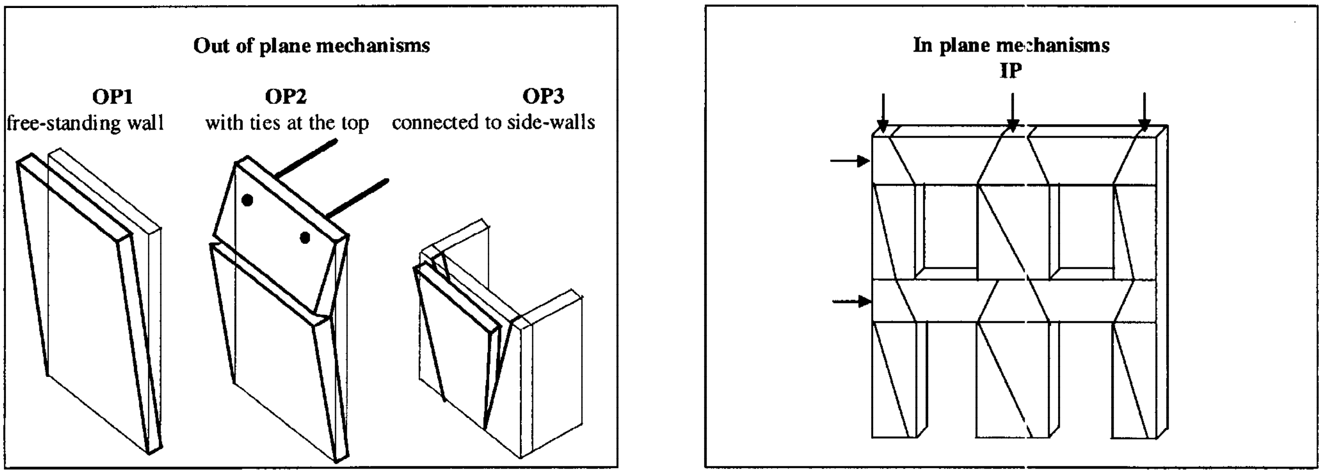


Fig. 2a, b, c, d. Idealised mechanisms for different boundary conditions of the wall.

DEFINITION OF DAMAGE LEVEL DISTRIBUTION

The histogram distribution in eight classes of equivalent shear capacity has been analysed taking into account the minimum and maximum level for each building, as obtained by the two classes of mechanism (Fig. 3). Although the value associated with the in-plane mechanism is not necessarily always the greater (compare with Fig. 1, i.e. cases in which the connection with the lateral walls is effective, or the facade presents slender piers), this can be assumed to occur in most cases, and therefore a first qualitative estimate of the benefit of inserting some form of transversal connection can be carried out by considering the two corresponding cumulative frequency distributions in Fig. 3. If, by means of such an upgrading, for each building the second type of mechanism is more likely to occur, then for a level of $A/g = 0.16$, a reduction of 30% of buildings with triggered mechanisms will be obtained.

In the present conditions, instead, for a threshold of $A/g \geq 0.04$, for more than 30% of the buildings a mechanism will occur, pointing out an intrinsic vulnerability to low levels of seismicity which, although conservatively estimated, is not acceptable by present safety standards. It is important to remember, however, that these values do not indicate that the single building cannot withstand a certain level of seismic action and hence will face collapse. In fact, the values obtained only indicate a likelihood that a number of buildings in the sample will overcome a limit equilibrium configuration. In order to quantify the loss associated with the overcoming of an A/g threshold, a relationship between level of A/g, distribution of mechanisms in the sample and corresponding level of damage needs to be established. The method applied has then been checked *a posteriori* with data from other sources including that available for the 1755 Lisbon earthquake.

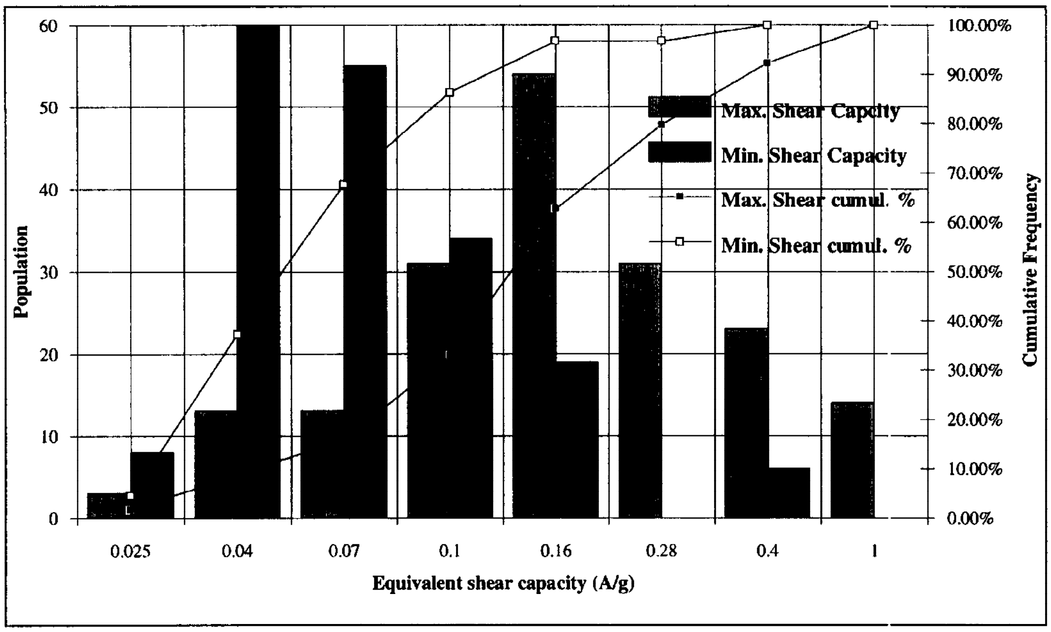


Fig. 3. Distribution of minimum and maximum equivalent shear capacity

The method followed starts from the assumption of the MMI definition for a certain level of intensity. For instance level VIII is identified by extensive heavy damage in the masonry buildings (20 - 50%) and single partial collapse (10%). These definitions are valid for brick-masonry buildings of up to 3 storeys. A building stock of rubble masonry with an average height of four storeys, as in the Alfama district, would probably experience a wider extent of damage and a higher average damage. In order to quantify the proportion of buildings experiencing a certain level of damage in relation to a particular level of overcome equivalent shear capacity, they have been grouped by number of storey and type of laterla restraint. Table 1 shows the range of equivalent shear capacity (in A/g) for each class of storeys and type of lateral restraint, and the percentage of building belonging to each class. It can be assumed that a damage level D2 will be associated with the initial overcoming of the minimum value of A/g, for the low level of acceleration associated, and for the dynamic nature of the action (the calculation here assumes an equivalent steady state). For the out of plane mechanism flexural subhorizontal cracks will show where the hinge forms. When passing the next threshold of A/g, the crack pattern will widen and deepen and the buildings will move to the next level of damage. In this way, cumulative distribution of damage levels D1 to D5 for an interval of A/g covering from intensity VI to IX have been computed as shown in Fig. 4. In Table 1 the level of damage for each group associated with intensity VIII is also shown. A similar distribution can be assumed for the in plane mechanism. In this case the tresholds of A/g also represent somehow different types of mechanism, from the ones dominated by shear failure in the upper spandrels (high number of storeys), leading to partial collapse, to the ones with flexural and diagonal cracks in the piers (depending on the pier's geometry), causing a more stable type of damage.

The cumulative distribution of the minimum shear capacity coincides with the cumulative distribution of level D2 according to the assumption above. Only for a small percentage of the building stock (6 floors with no restraint, accounting for 2.2 % of the sample) can the initial damage be considered severe (level D3). Overcoming the maximum shear capacity will produce damage level D3 and D4 in most buildings with a certain percentage of collapse (D5) for those building for which the two classes of mechanism show very similar equivalent shear capacities (i.e., 5 and 6 storeys with little lateral restraint and slender piers). The maximum shear capacity curve therefore intersect the cumulative damage D3 and D4 curves across an interval of acceleration which corresponds to intensity from VII to IX MMI.

Table 1. Distribution of mechanisms and damage level for class of storey

General Data			**Free wall**			**Walls with ties**			**Walls with quoins**			**Walls with ties and quoins**		
Number of storeys	Total % of Building	Max Min. Height (m)	% of Building	Min Max A/g	Damage Level	% of Building	Min Max A/g	Damage Level	% of Building	Min Max A/g	Damage Level	% of Building	Min Max A/g	Damage Level
1	0.07	3.20 1.00	0.85	0.188 0.6	D2	0.0			0.07	0.425 1.35	D0	0.08	0.61 1.95	D0
2	0.11	6.50 4.40	0.95	0.092 0.13	D3	0.0			0.05	0.21 0.29	D2	0.0		
3	0.3	8.50 6.50	0.75	0.070 0.092	D3/D4	0.15	0.091 0.12	D3	0.04	0.16 0.21	D2/D3	0.06	0.23 0.30	D2
4	0.32	12.20 8.50	0.56	0.049 0.070	D4	0.250.	0.064 0.091	D3/D4	0.12	0.11 0.16	D3	0.07	0.16 0.23	D2/D3
5	0.15	14.00 12.00	0.42	0.042 0.050	D4/D5	0.42	0.065 0.055	D4	0.04	0.095 0.113	D3/D4	0.12	0.13 0.16	D3
6	0.05	15.00 17.00	0.44	035 0.040	D5	0.23	0.045 0.052	D4/D5	0.0			0.33	0.11 0.13	D3/D4

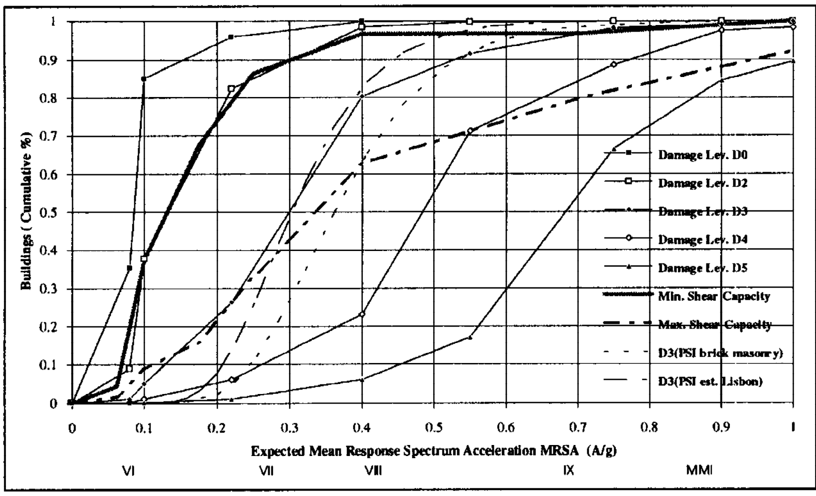
VALIDATION OF RESULTS

In order to validate the results obtained with the method described above, the equivalent shear capacity has been converted in mean response spectrum acceleration (Mrsa) which takes into account the mean value of elastic response spectrum with 5% damping in the range of period typical of masonry buildings. The two values can be considered related by an amplification factor of 2.5. The Mrsa also relates to the MMI macroseismic scale. In the case of Portugal it is shown (from Oliveira *et al.*, 1994) that VI corresponds to $Mrsa \cong 0.10\ g$, VIII to $Mrsa \cong 0.40\ g$ and IX to $Mrsa \cong 0.64\ g$.

In this way it become possible to compare the cumulative damage distribution with the curves of normal distribution for building masonry (PSI curves) elaborated over the database collected by the Martin Centre (Spence *et al.*, 1992b) through in situ survey of post-earthquake damage in 20 different locations world-wide. In Fig. 4 the PSI curve of damage level D3 for brick masonry is plotted. It can be observed that 50% of cumulative damage occurs for a value of $Mrsa = 0.36\ g$, 20% greater than the value obtained for the D3 damage distribution in the Lisbon sample. The D3 corresponding normal distribution curve, in Fig. 4 labelled *PSI est. Lisbon*, is obtained with a difference in central value which corresponds to a 25% difference in extent of damage for a level of intensity VIII.

The curve computed with the limit-state analysis method shows a dispersion higher than the PSI estimate. This can be explained by the relatively small number of buildings considered for the analysis, and by the fact that the explicitly assumed differences in state of conservation, out of verticality and lateral restraint, all contribute to widen the spectrum of structural responses within the sample, so that an equivalent normal distribution will have a greater standard deviation.

One other way of confirming the results obtained with the present analysis is to compare them with the evidence drawn from the historical records of the 1755 Lisbon earthquake. This earthquake is recognised as the most destructive in Lisbon, caused by a relatively distant seismic source in the Atlantic Ocean South-West of Lisbon with an intensity IX in Lisbon and a return period of 1000 years (Oliveira 1994). The most complete work on this event and the damage caused in Lisbon is the one by Pereira de Sousa (1923). For the more important buildings, i.e. churches, monasteries and other institution's buildings, Pereira takes into account three levels of damage (damage without collapse, partial collapse, total collapse) which can be associated with levels D3 to D5, respectively, of the present study. The percentage of buildings in each class is 23%, 45% , 32 %, respectively, with virtually no building left undamaged. Such a distribution is instrumental for the attribution of intensity IX to this event. Intensity IX corresponds to $Mrsa \cong 0.64\ g$ which according to Fig. 4 gives cumulative damage values of 95% D3, 80% D4 and 40% D5, with a mean error of 5% compared with 100%, 77%, 32% obtained by Pereira's estimate. In comparing the two sets of results, it has to be considered that while Pereira's data refer to the better constructed buildings of the time, the results of the present analysis refer to vernacular buildings, partially upgraded as an effect of that same earthquake. While one case is not sufficient to prove the general validity of the distribution, the independent way in which the two sets of data have been obtained, suggests that the correlation is quite satisfactory.



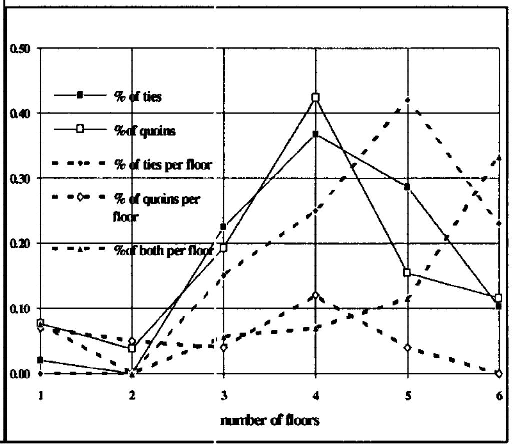


Fig. 4a: Equivalent shear capacity and damage level cumulative distribution *Fig. 4b: Present distribution of ties and quoins*

PROPOSED UPGRADING STRATEGY

The main criteria which ruled the choice of the upgrading techniques are, to be as homogeneous as possible with the present fabric of the buildings, to be relatively low cost and relatively easy to realise, while maximising the damage reduction (Spence *et al.*, 1994). A first suggestion of how to achieve this goal comes from the analysis of the present situation (Table 1 and Fig. 4b). Data show that ties are present only in buildings of three or more storeys, in increasing proportion up to five storeys. Quoins alone, although more evenly spread, are less common and completely absent in 6 storey class, while quoins in conjunction with ties represent an high proportion in this same class. However, high percentages of buildings in every class do not show any type of aseismic structural device. This percentage is maximum in the 2 storey class and decreases with the increase in number of storeys, showing some awareness by the builders of the increasing vulnerability with height.

While the limit-state analysis shows that quoins will provide an increase in equivalent shear capacity greater than ties, it is evident that in an existing building the latter device is much easier to realise than the former, and less expensive. Our proposed strategy is therefore to insert 2 ties per floor every two floors to all building of 4 storeys or more without any form of strengthening already in place, in so reducing the distribution of minimum equivalent shear capacity as shown in Fig. 5 and reducing the maximum damage to level D3 for intensity VIII. This ensures that there will be no loss of life for this level of intensity.

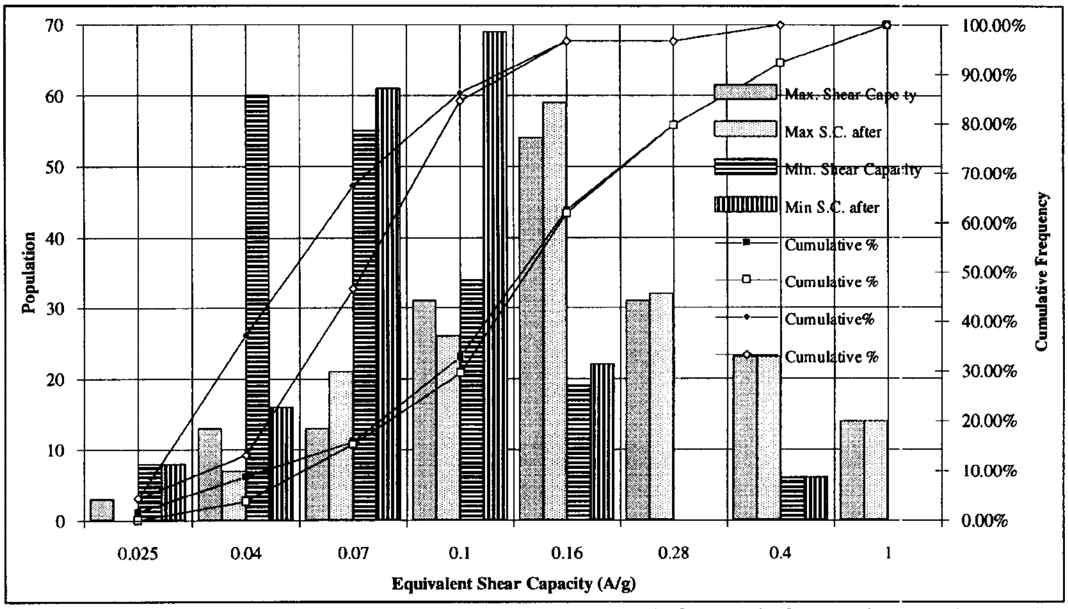


Fig. 5 Distribution of minimum and maximum shear capacity before and after implemented upgrading

COST BENEFIT STUDY

The effectiveness of the proposed strategy can be investigated by means of a simplified cost-benefit study. An estimate of the costs of introducing ties has been obtained using data from specialist contractors, based on the costs of analogous programmes of masonry upgrading. They are presented in terms of cost per m^2 (in $US equivalent) in Table 2. Considering a nominal cost of $ 150 per tie, the estimate of the cost of this programme over the Lisbon sample is of 1.16 $\$/m^2$ of upgraded lettable area, for a total value of $ 16650. The upgrading of the 3 storey class of building (for which one set of ties is planned) will cost additional $ 12000, corresponding to 1.41 $\$/m^2$. The total number of upgraded buildings would be 46 in the first instance and 86 in the second, the 25% and 47% of the sample respectively.

Table 2. Cost and benefits of strengthening programme.

Intensity	VI	VII	VIII	IX
Exceedence period (yrs)	20	100	1000	10000
Annual recurrence probability ($x10^{-4}$)	400	90	9	1
Annual loss, unstrengthened ($x10^{-4}$)	7.0	26.1	4.6	0.9
Total annual loss, unstrengthened (10^{-4})	38.6			
Annual loss, strengthened ($x10^{-4}$)	0.3	12.3	3.6	0.8
Total annual loss, strengthened ($x10^{-4}$)	16.7			
Saved annual loss ($x10^{-4}$)	21.9			
Assumed reconstruction cost ($\$/m^2$)	600			
Saved annual loss ($\$/m^2$)	1.31			
Average cost of strengthening ($\$/m^2$)	1.5			
Payback period (yrs)	1.37			

The benefits of the programme include direct benefits (future rebuilding costs), and indirect benefits including the saving of human lives, the preservation of the city's architectural heritage, and saved losses through business interruption and relief and rescue operations. Only the direct losses can be quantified with any confidence, if approached on a probabilistic basis, by calculating the annual probability of loss before and after strengthening. Studies by Oliveira (1994) indicate that the annual occurrence probabilities of earthquakes with intensities VI to IX are as shown in Table 2. Using the estimates of vulnerability above and assuming loss ratios of 5%, 20%, 50%, 80% and 100% (Coburn *et al.*, 1992)for each of the damage level D1 to D5, the annual expected loss for each intensity, and the overall total annual loss are shown in Table 2. It is interesting to observe that in each case, the expected losses are dominated by the 100-year event, an earthquake of intensity VII.

The annualised reconstruction cost before and after strengthening have been calculated on the basis of construction costs typical of Lisbon ($\cong$ 600 \$/m^2), showing that the annualised saving from the strengthening programme is about 1.3 \$/m^2. This can be compared with an average cost of strengthening (using ties alone) of \$1.5/m^2. The 'payback' period is calculated as 1.14 years.

CONCLUSIONS

- Substantial improvement to the seismic performance of historic stone masonry buildings is possible without serious alteration to the integrity of the fabric.
- A method based on limit state analysis has been developed to estimate the vulnerability of the sample in its present state to earthquake actions of different spectral acceleration and MMI Intensity. The method has been extended to show how the vulnerability would be reduced by an upgrading strategy involving the use of additional ties in the facade wall for the most vulnerable buildings.
- A cost-benefit analysis has indicated that the cost of the proposed upgrading strategy would be about \$1.50 per m^2 lettable area while the annualised loss reduction would be about \$ 1.3. The proposed strategy would pay for itself in about 1 year, constituting only about 1% of a full upgrading programme.
- The implementation of such a research programme has provided benefits to the ongoing programme of upgrading by the local authority responsible for the preservation and improvement of the Alfama District.

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