ESTABLISHING CORRELATION BETWEEN VULNERABILITY AND DAMAGE SURVEY FOR CHURCHES

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SUMMARY

The correlation between vulnerability and damage of churches, is the object of the paper. Damage data on this type of buildings have been collected in a systematic fashion following the Serravalle di Chienti earthquake of September-October 1997 and they have been statistically analysed, relating damage levels and patterns to typologies and presence of strengthening devices. A vulnerability analysis has also been carried out, based on the calculation of collapse load factors associated with mechanisms of collapse which best represent the surveyed damage patterns. The correlation between vulnerability and damage is discussed. The paper outlines the different levels of vulnerability of the churches with respect to residential buildings and the implications of these results on the assumed macroseismic intensity of specific events.

INTRODUCTION

The level of vulnerability shown by churches and other monumental buildings with respect to seismic action has been recognised as different and usually higher than the one associated to ordinary houses (Doglioni et al., 1994). Especially in respect to moderate earthquakes. This observation can be ascribed to the intrinsic greater structural vulnerability due to open plan, greater height to width ratio and often the presence of thrusting horizontal structures and the smaller tolerance to cracking if the decoration apparatus is not to be damaged.

This problem is usually tackled by advocating the unique character and value of a given church and pursuing a policy of one assessment and one strengthening project specifically tailored for it. However the recent Umbria-Marche earthquake of September-October 1997 has shown that the problem is generalised and that structural typologies and associated type and distribution of damage are fairly recurring. Figures provided by the Regional Sovrintendenza show that in Umbria the number of damaged churches is 1815 and, although complete collapses are not numerous, the loss of important artefacts due to highly developed crack patterns and partial collapses, is quite considerable. Many of those churches had already undergone forms of strengthening, borrowed from the general repertory used for residential housing, and damage patterns were highly dependent on these. It is therefore evident that the problem can not be solved with a one to one approach, but more general rules should be laid out. The first step in this direction is the assessment of the specific vulnerability of this class of buildings and then the specification of appropriate measures to reduce that vulnerability.

Furthermore, the general problem of the quantification of the historical seismicity and hazard of a region, and how it relates to instrumental more recent data, is also highly affected by the assumed vulnerability of churches. Indeed, the intensity of past earthquakes is derived mainly on the basis of historic accounts of their associated damage and these usually relate to churches and other “emergent” buildings. It is evident that assuming the wrong level of vulnerability might lead to actually overestimate the intensity of a particular earthquake and overall the hazard of a given region. This in turn could prompt the development of strengthening policies and techniques, which are over designed and might contribute to the loss of important heritage in an attempt to mitigate a risk, which is actually not realistic.

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The paper presents the analysis of damage in historic masonry churches in Umbria, caused by the sequence of shocks 26 September 1997-20 October 1997, with epicentres in Serravalle di Chienti, and correlate it to seismic macro intensity measures. A general approach for the assessment of the vulnerability of homogeneous classes of buildings, based on the limit state analysis method, is then used to model surveyed collapse mechanisms and to define a vulnerability function. The final part of the paper evaluates the correlation between calculated vulnerability and observed damage, and discuss these results.

VULNERABILITY AND DAMAGE TO CHURCHES

The necessity of developing ad hoc procedures for the assessment of the vulnerability of churches has been identified by Doglioni and al.(1994) since the Friuli earthquake of 1976. While the architectonic and historic value of this class of buildings in general requires a specific seismic analysis for each building, recurring typologies and numbers involved suggest that a statistical approach could yield useful results, both in the immediate aftermath of the earthquake, and previous to an event for identifying groups of buildings particularly vulnerable and defining priority criteria for structural improvement. Studies of this kind and further interest for the seismic behaviour of churches was raised in the aftermath of the September 1997 earthquake of Serravalle di Chienti, Italy, and were made feasible by the development of a database on damage and vulnerability of churches in Umbria (Lagomarsino et al. 1998), commissioned by the Civil Protection Authority and the Cultural Heritage Authority, and carried out by the National defence from Earthquake Group, part of the National research Council.

The database contains data on more than 1000 churches in the Umbria region, organised by Comuni. The present analysis focused on the Comuni with a territory along the border with the Marche region, namely Assisi, Foligno, Gualdo Tadino, Nocera Umbra, Sellano, Spello and Valtopina, which enclose a geographic area along the Appenini mountains, with a north south dimension of approximately 40 Km. Along these main ridge where located the shifting epicentres of the major events. The area extends westward from the Appenini with a steep change in levels occurring in few miles, from a height of 1700 m. in Colfiorito to few hundreds m. above sea level in Foligno. This area was chosen for the range of intensity recorded, spanning from 6 EMS in Assisi to 8.5 EMS in Colfiorito, as well as for the great presence of churches of all types. The analysis has been restricted to churches with the typology of single nave, with or without transept and apse, with or without lateral chapels. This typology, evolved from the archetype of the so called hut church, typical of the region, accounts for more than 90% of the churches contained in this sample and includes the small country hermitic chapel as well as the imposing basilica Superiore di San Francesco in Assisi. Differences are not only of scale or lay out, but most importantly of arrangement of structural elements, quality of masonry, level of conservation. The sample contains 360 churches, of which 155 in an area of intensity EMS 6, 163 in area of EMS 7 and 42 in area of EMS 8 –8.5. From the database information were drawn on layout, gross geometric dimensions, presence of strengthening device, type of masonry, type of roof structure, type and level of damage. Photographic documentation was also available for approximately one third of the sample.

Distribution of damage among churches

One useful feature of the GNDT database is the classification of damage by structural macroelements and associated mechanisms of collapse. For the purpose of this study, five mechanisms were analysed with reference to the macroelement façade: global overturning of the façade, overturning of the upper part, in plane failure of associated with membrane behaviour (arch effect), flexural failure due to lateral behaviour, diagonal crack pattern associated with predominantly shear behaviour. Their cumulative distribution with respect to 4 levels of damage are shown in fig. 1, irrespective of the level of seismic intensity at the site. For the four levels of damage (D1 to D4) a mean damage ratio is calculated, which takes into account extent and structural relevance of damage. The distribution in fig.1 appear to be rather insensitive to the type of damage, and the proportion of light damage (D0-D1) accounts for 40 to 60 % of the sample, in all cases. It should be considered that this class also includes cases in which the specific mechanism has not been identified.

While it is outside the remit of this paper to discuss method and reliability of the mechanisms reconnaissance within the database, it is worth analising the data to see whether there is indeed a correlation between intensity and specific mechanisms.

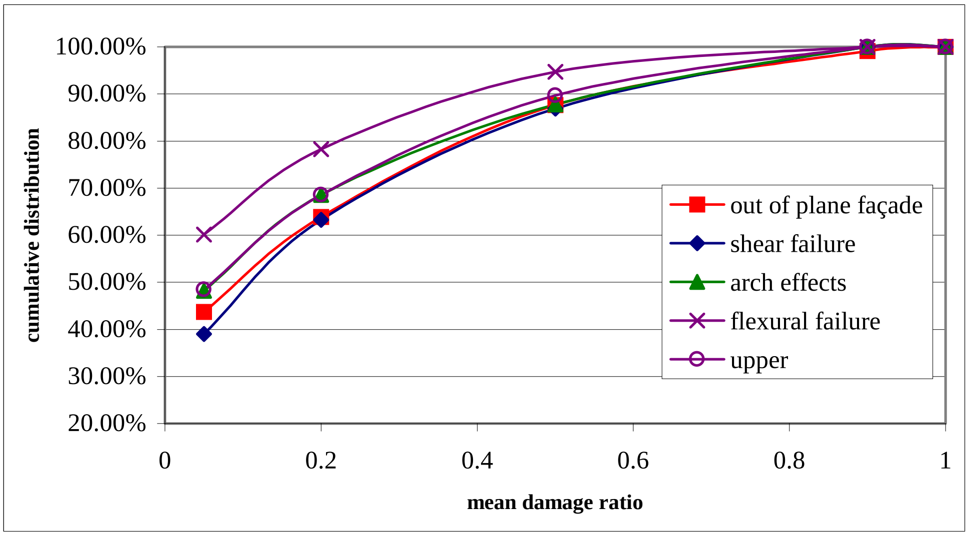


Fig. 1 Cumulative distribution of damage levels for different damage types

The results of this query are summarised in percentile terms in table 1. For low intensity there is not substantial difference in the occurrence of one mechanism in preference to another and on distribution within classes of damage, as the intensity increases there is an increment in the occurrence of shear failures with higher level of damage, while flexural failures tent to reduce. No marked difference in occurrence and level of damage of the other mechanisms can be identified. However, more than one mechanism will generally be recognisable for each church, and therefore in order to correlate macroseismic intensity and damage, a global measure of damage has been sought. The total damage ratio for each church is calculated as the sum of the damage ratio associated to each of the previous five mechanisms divided by the number of triggered mechanisms. Furthermore, in order to take into account a non linear superposition of effects associated with the presence of increasing numbers of mechanisms, a parabolic variation of weighting ranging between 1 and 1.4 for damage level D2 to D4 was introduced. The weighting was calibrated so as to yield total damage ratio of 1, equivalent to collapse, when the instance of more than two separate cases of damage D4 (partial collapse) resulted for the same building. Results of cumulative distribution are shown in fig.2 for four classes of macroseismic intensity.

Table 1 Distribution of damage by type of mechanisms and intensity levels

Mechanisms	Global overturning			Upper overturning			Arch effects			Flexural failure			Shear failure		
Intensity level / Damage level	VI	VII	VIII	VI	VII	VIII	VI	VII	VIII	VI	VII	VIII	VI	VII	VIII
D0-D1	0.56	0.35	0.31	0.65	0.37	0.31	0.52	0.48	0.34	0.60	0.58	0.63	0.57	0.3	0.07
D2	0.2	0.22	0.12	0.16	0.24	0.15	0.24	0.18	0.15	0.22	0.15	0.15	0.24	0.26	0.15
D3	0.18	0.28	0.24	0.16	0.26	0.22	0.19	0.18	0.24	0.16	0.18	0.12	0.17	0.24	0.46
D4-D5	0.05	0.14	0.27	0.02	0.13	0.31	0.05	0.16	0.27	0.01	0.08	0.10	0.1	0.19	0.31

In terms of global damage, the proportion of cases with little or no damage is now substantially reduced, accounting for 20% in areas of intensity EMS 6 and decreasing to not for intensity EMS 8 and above. Global mean damage ratio for the four intensity areas are 0.41, 0.59, 0.75 and 0.8 respectively. This data should be compared with the mean damage ratio obtained for distribution of single mechanisms. Given the very close level of occurrence shown in fig,1 and table 1, the average has been calculated, which provide a cumulative figure of 0.28 for the all sample, and values of 0.25, 0.29 and .40 for respectively EMS 6, EMS7 and EMS8 or greater.

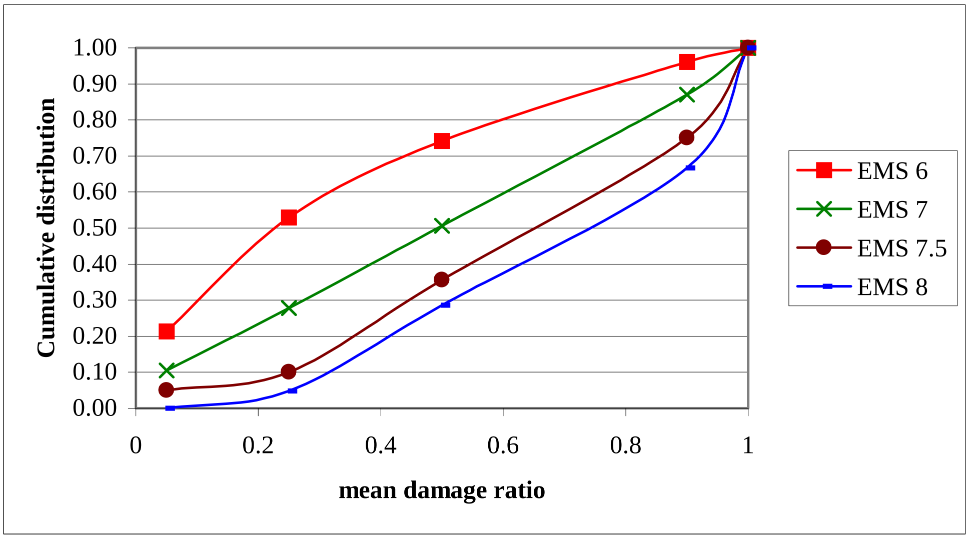


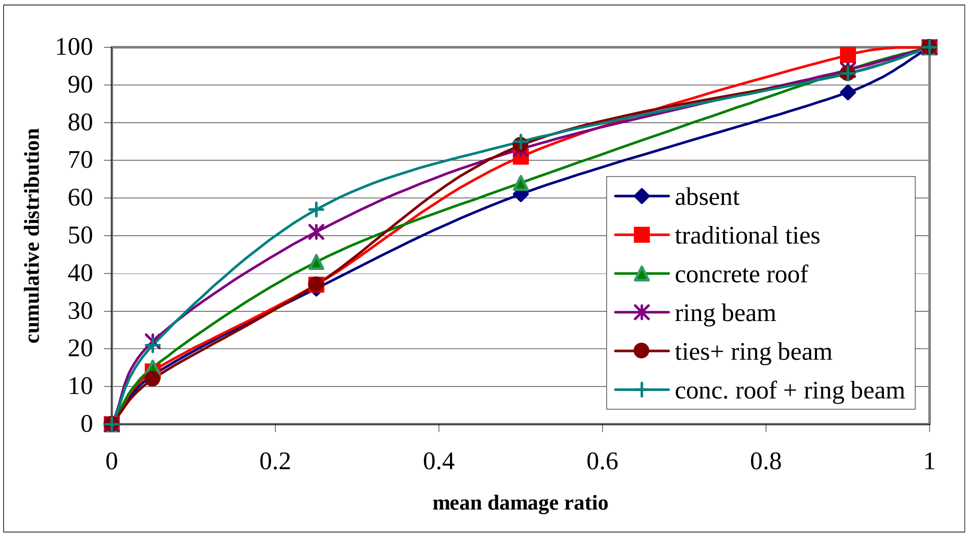
Fig.2 Cumulative damage ratio distribution for different level of intensity EMS

In order to better understand the correlation between measures of ground motion and damage to a particular sample of buildings, it is essential to characterise them structurally. This can be done by analysing the presence of specific structural elements, which are significant for their seismic behaviour. In table 2, the aggregate percentile of different strengthening systems within the sample, and then for areas of homogeneous intensity, are shown. The distribution of strengthening device is substantially independent from the level of seismicity recorded except for the area with seismicity EMS 7.5. On average up to two thirds of the buildings are unstrengthened. This value, however, would also include cases for which there was no record. The proportions of traditional ties, concrete roof and ring beams, is generally similar, around 20%. Traditional ties are slightly more common than other devices. Occurrence of more than one device is never greater than 11%, and in no way correlated to greater seismic intensity. A completely different pictures would be obtained if the query is run, using the territory of the Comuni: then, in Nocera only 24% of the churches are unstrengthened, while up to 50% have ringbeams; in Foligno, instead, 71% are unstrengthened and only 9% have ringbeams. The presence of traditional ties is somewhat more even with 4 Comuni with 14% and a maximum of 33% in Nocera. It would therefore appear that the difference substantially lies in the implementation of recent strengthening policies by the local authorities.

Table 2 Correlation between strengthening and seismic intensity

Type of strengthening	Totals (%)	EMS 6 (%)	EMS 7 (%)	EMS 7.5 (%)	EMS 8 (%)
Absent	64	58	56	80	57
Transversal ties	17	20	20	5	23
Longitudinal ties	3	2	5	0	5
Concrete roof	16	20	16	10	24
Ring beam	16	17	20	5	14
Ties + ringbeam	4	4	5		9
Conc. roof + ringbeam	7	11	9		5

The relevance of strengthening devices to the seismic performance of buildings is measured by the value of associated mean damage ratio, as shown in fig. 3. Traditional ties, whether or not in connection with ringbeams, while maintaining same level of occurrence as the unstrengthened case, for damage up to level D3, are very effective in preventing partial and total collapses (D4). Concrete roof, effective at the lower level of damage, has proportions of high damage comparable with the unstrengthened case, while ring beams coupled with concrete roof seems to be the most efficient option to prevent medium to serious damage, but not necessarily to prevent collapse. In the region of high damage the ties results most efficient, because they are more sympathetic to buildings of poor quality, which are also the most likely to experience high damage. The average damage ratio for each class of reinforcement further proves the point. These are 0.54, 0.49, 0.44, 0.41, for unstrengthened, concrete roof, traditional ties, ring beams respectively. The traditional ties allow a reduction of average damage ratio of 10%, to level of damage D2-D3 rather than D3-D4. Ring beams are marginally more effective, while the implementation of concrete roof, often introduced to reduce fire risk, is not on its own substantially beneficial. In connections with ringbeams however the average damage ratio drops to 39%. Of course these results should be considered in light of the relatively moderate level of shaking experienced by the sample.



.Fig. 3 Cumulative distribution of damage ratio for classes of strengthening.

Vulnerability analysis for churches

In agreement with the observed damage patterns relevant models of failure mechanisms have been developed and their associated equivalent shear capacity calculated with an approach based on limit state analysis. The procedure, presented in details by the author elsewhere (D'Ayala et al 1996) looks in detail at each building to establish its equivalent shear capacity, by means of collapse load factor, calculate a vulnerability function as the inverse of the shear capacity, and then analyse this with a statistical approach over the sample, to draw conclusions of general nature.

Fig. 4 Typical mechanisms surveyed in Sellano and Nocera Umbra

Some of the most recurring damage patterns are illustrated by photos of churches in Sellano and Nocera Umbra (Fig.4). The corresponding mechanisms chosen, schematically depicted in fig. 5, are the global overturning of the façade Ch1, according to three different constraint conditions (of which only the non constrained one is depicted in fig 5, the other two being determined by presence of longitudinal ties or presence of ring beam at roof level), the overturning of the upper part of the façade, Ch2, the development of vertical cracks in association to horizontal membrane behaviour (arch effect) Ch3. These correspond to the surveyed damage mechanisms referred to in fig.1 and table 1. These three classes of mechanisms are not mutually exclusive, and, as proven in the survey, they can to a certain extent coexist on the same structural element. The openings' position and size within the facade assume an important role in the determination of the actual mechanism and its load factor. For the purpose of the present study a simplified scheme with main central door and in axis rose window has been assumed, as the most common arrangement. The failure load factor for each of the five mechanisms has been first calculated, choosing the maximum among the mechanisms in class Ch1, according to the particular constraint condition. Then this value is compared with the failure load factors associated with the other two mechanisms and the minimum value is chosen as the most likely to be triggered, as the one involving the least amount of energy input.

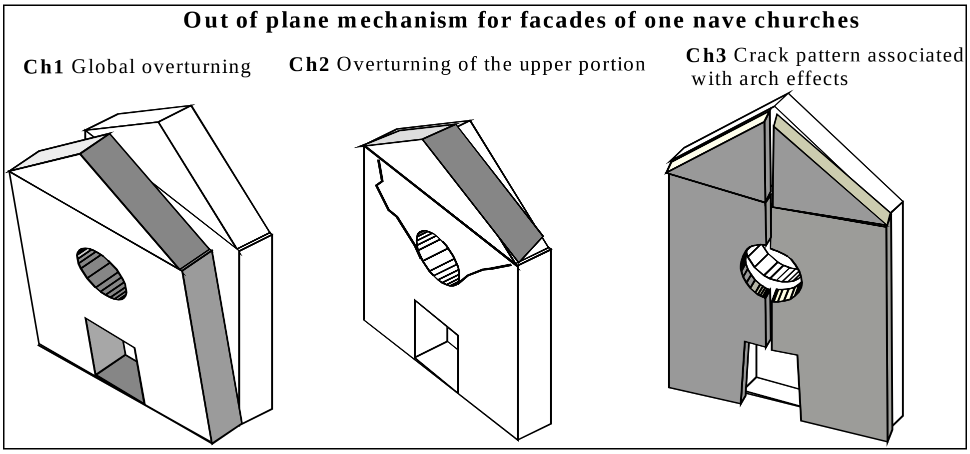


Fig. 5 Collapse mechanisms associated with out of plane action for the façade of churches.

The vulnerability of each church, in terms of equivalent shear capacity as a function of the slenderness of the façade has been plotted in fig. 6. The slenderness here is intended as the ratio between the major dimension of the façade and its effective thickness. As information on the quality and maintenance of the masonry is rather sparse, the effective thickness has been calculated extrapolating from a reduced quantity of data and based on visual judgement.

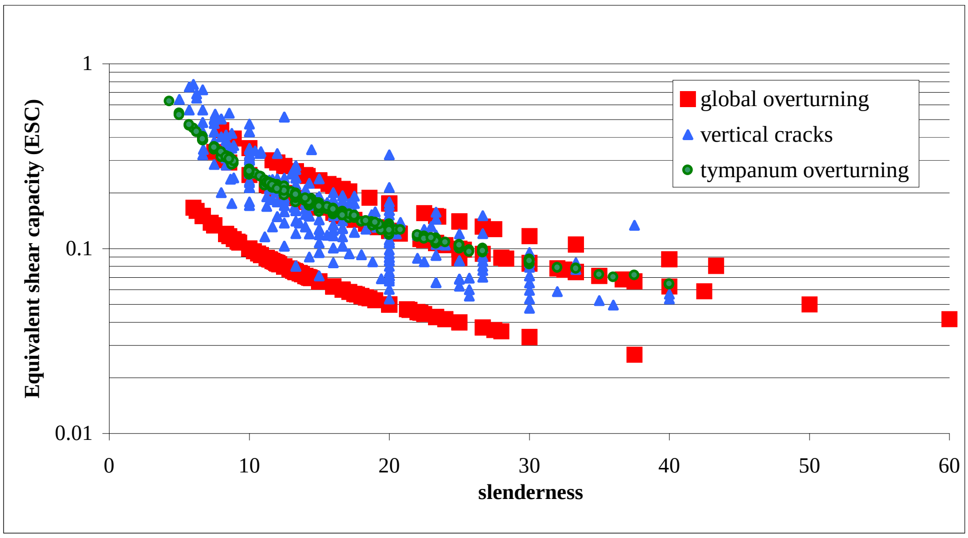


Fig. 6 Equivalent shear capacity of the sample for different mechanisms of collapse.

Fig. 6 shows that for many churches the global overturning will develop in preference to other mechanisms in the absence of ties or other structural devices, which will connect the façade to the rest of the structure. However if any of these devices is present then in general the load factor associated with the arch effect (Ch3), either in the horizontal or vertical direction is smaller than the collapse load factor associated to other mechanisms and hence this will occur in preference. This result is in agreement with the distribution of damage associated with mechanism 3 for Sellano, where the presence of ties was poor. It is also evident from fig. 6 that global overturning in presence of ties and overturning of the upper part of the façade (Ch2) have similar collapse load factor and this would explain the similar incidence of this type of mechanisms in the sample, especially in Nocera Umbra where an high proportion of churches have ties. Finally in the case of presence of ring beams at roof levels the local collapse of the upper part of the façade will take place in preference, as this is associated with lower collapse load factors. This is very well exemplified by the second picture in fig. 4, which refers to the Church of St. Stefano in Parrano, Nocera Umbra, for which an "antiseismic roof" had been implemented shortly before the earthquake.

CORRELATION BETWEEN VULNERABILITY AND DAMAGE

To evaluate the relationship between vulnerability and damage distribution for the churches the greater vulnerability value was chosen among the ones calculated with the approach defined before. In other words, the mechanism chosen as the most likely to take place is he one with the lowest shear capacity. This value was correlated to the associated value of total damage to the façade. These data and their logarithmic regressions are plotted in Fig.7 for different levels of seismic intensity EMS. Correlation coefficients and curves equations are summarised in table 3, where D is the mean damage ratio and V is vulnerability. The data distribution shows higher correlation in terms of vulnerability and damage associated with greater seismic intensity.

Table 3

Sample	Logarithmic regression equation	Correlation factor R
EMS 6	D =0.301ln(V) –0.40	0.50
EMS 7	D =0.415 ln(V) –0.50	0.73
EMS 8 – 8.5	D =0.474 ln(V) –0.57	0.77

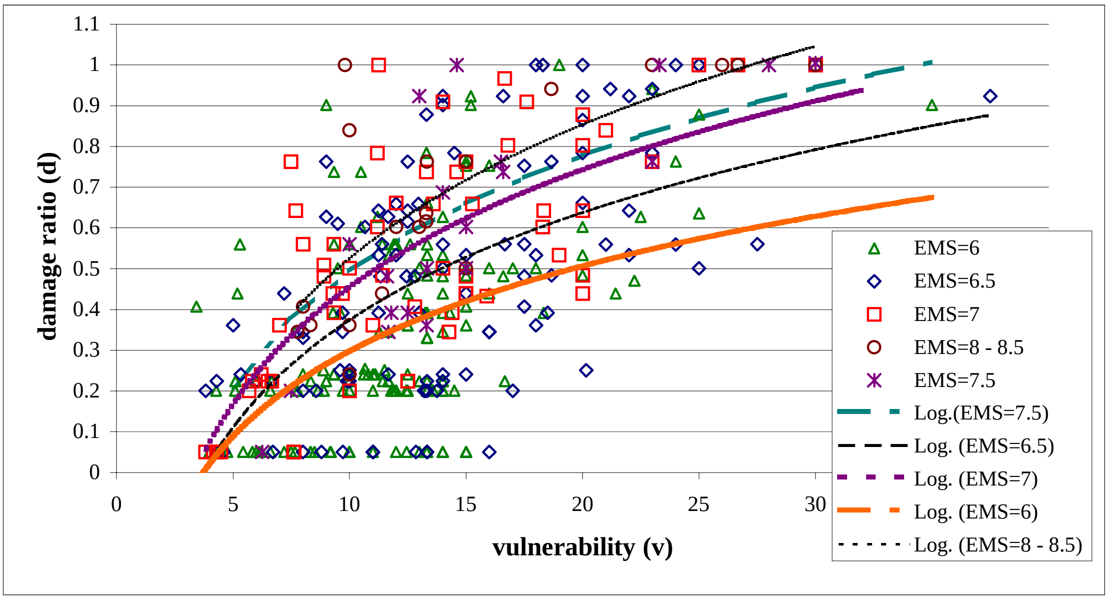


Fig.7 Correlation between vulnerability and damage for churches and houses.

CONCLUSIONS

A simple model which predicts vulnerability and damage through a calculation of the building's equivalent shear capacity seems to be effective in identifying the general performance of a group of buildings, though anomalous cases exist. The model has been applied to a sample of churches in 7 Comuni of Umbria hit by different intensity levels. The calculation of the equivalent shear capacity is carried out by identifying mechanism of collapse associated with structural and constructional details, and a vulnerability function is derived, as the inverse of the minimum of the calculated shear capacities.

The vulnerability analysis proves that the presence of traditional ties and other "seismic" constructive features are essential in the quantification of the vulnerability, together with the presence of more recent strengthening implementation. The good level of correlation obtained over the samples between the computed vulnerability and the observed structural damage, over a range of intensity, shows that the methodology is sound, and can be used as a predictive tool. Application to other database of damage would strengthen its reliability.

To this respect, it should also be noted that churches that had undergone strengthening, except few cases, in general performed well in terms of limited damage. However where the intensity of the earthquake was greater serious shear crack patterns developed, and while these represent a stable type of damage, from a structural point of view, they can actually threaten mural paintings and other wall supported artefacts. Hence different type of strengthening should be sought, which do not increase the stiffness of the overall structure. More research is needed in this field.

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