THE PERFORMANCE OF STRENGTHENED MASONRY BUILDINGS IN RECENT EUROPEAN EARTHQUAKES.

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SUMMARY

This paper assembles and reviews new information on earthquake damage to strengthened masonry buildings. It considers how the performance of strengthened buildings compares with that of unstrengthened buildings in the same location and what can be learnt about the relative performance of different techniques of strengthening. The paper then explores what can be deduced about the cost-effectiveness of strengthening programmes for existing masonry structures and identifies some lessons learnt from this experience in devising future strengthening programmes for use in Europe and elsewhere

INTRODUCTION

Virtually all buildings in Europe of architectural importance built before the middle of this century are of some form of masonry. Not only the numerous notable individual monuments, but the entire historic centres of most European cities are now recognised as vital elements of this architectural heritage. In many areas, these buildings are in need of protection from natural as well as man-made hazards, of which perhaps the earthquake hazard is the most destructive.

The idea of strengthening masonry buildings to improve their resistance to earthquakes is not new. Throughout the history of architecture there have been many devices used to enhance earthquake resistance, particularly in those parts of Europe where the occurrence of damaging earthquakes is more frequent. Over most of the last 150 years, the use of iron or steel ties at floor and roof levels connecting the external walls together has been common practice in some areas. More recently, there has been a growing practice of introducing reinforced concrete slabs and roofs supported on and attached to the external walls; and other techniques, such as jacketing external walls with a thin layer of mesh-reinforced plaster have been used in some places.

These techniques have been supported by the results of laboratory studies and design procedures which indicate that they should improve performance, even if not attaining the level of resistance required of new buildings. Regulations have been developed (e.g. in Italy) to permit improvement strategies to be employed on a local basis.

Until recently, however, there was little direct evidence of masonry buildings which have been strengthened and subsequently tested by earthquakes. A paper presented to the Tenth European Conference on Earthquake Engineering was able to identify only a handful of instances from Italy, Romania and Croatia [Spence, D'Ayala et al, 1994]. Since that time, studies have been carried out into the 1980 Irpinia earthquake and several recent events, which have considerably enhanced the amount of data available. This paper presents this new data, from the 1980 Irpinia and the 1997 Umbria-Marche earthquake, both in Italy, and from the 1998 Faial, Azores earthquake; and by comparing it with earlier data, draws conclusions on three important questions:

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How did the performance of strengthened buildings compare with that of unstrengthened buildings in the same location ?

How have strengthening interventions been designed and implemented, and what can be learnt about the relative performance of different techniques of strengthening?

What lessons can be deduced concerning the cost-effectiveness of strengthening programmes for existing masonry structures?

RECENT DATA

2.1. The September 26 1997 Umbria-Marche Earthquake

This earthquake took place in a region much affected by previous earthquakes and in which masonry buildings of traditional form predominate, while reinforced concrete buildings are relatively few. Because of the history of earthquakes of moderate magnitude, it has become common practice over the years to reinforce existing buildings by a variety of retrofit techniques. After the most recent earthquakes of 1979 and 1984 new standards were introduced in some parts of the area to facilitate and regulate such strengthening. The latest earthquake therefore represented an opportunity to re-examine the earthquake vulnerability of unreinforced masonry construction and to test the effectiveness of reinforcement by a variety of techniques, both recent and traditional, under earthquake loading.

On September 26 1997, two earthquakes of magnitudes Ms = 5.5 and 5.9 occurred on the same day. These were followed by a sequence of damaging shocks which included three shocks with magnitudes greater than 5.0 within the next three weeks. To examine the damage to strengthened and unstrengthened buildings, buildings were surveyed in three locations: Assisi (EMS Intensity 6), Sellano (EMS intensity 7-8), Nocera Umbra (EMS Intensity 8). Each sample consisted of buildings within a small area, and all buildings were included, whether damaged or undamaged. The purpose of these surveys was to relate the level of damage to the presence of strengthening devices and to the vulnerability of the sample.

Nocera Umbra and Sellano are roughly equidistant from the first epicentre, about 15 km, while Assisi is further, approximately 25 km. Table 1 shows the percentage of buildings constructed before the area was classified as seismic in 1981. It is clear that very few buildings in any of these towns are built to formal earthquake codes.

Table 1: General data on classification of building stock in the three town centres

Town	Intensity EMS	Total dwellings	% built pre classification	Buildings in town centre
Assisi	6	8892	86	about 500.
Nocera Umbra	7-8	2823	89	210
Sellano	8	1039	80	150

In each centre, most buildings can be dated back to the middle ages, although many have undergone major alterations, most following serious earthquakes during the 17th and 18th centuries, and some early this century. The bearing walls are generally of double-leaf stone masonry with or without an inner rubble core, the outer layer consisting of roughly dressed limestone blocks set in coarse lime mortar. The quality and conservation of the masonry work varies from coarse rubble with poor mortar in thick joints, to well dressed stone ashlars with mortar joints of a few millimetres thickness. Insertion of horizontal layers of brickwork, window surrounds in brickwork and stone lintels in local tuff stone is common. Many of the buildings are plastered.

The original horizontal structure consists of brick vaults for the lower floors and timber joists for the upper floors, with or without main timber beams. Some of the original structures have been replaced during this century by I-beams and jack-arches. The roof structure is normally made of timber rafters simply supported on the top of the walls.

Traditional strengthening devices such as ties and dressed stone corners are present in each of the three towns. More recent strengthening work has been carried out to a different extent in the three towns, following damage caused by previous earthquakes in 1979 and 1984. In Sellano a campaign of *upgrading* to the current code requirement for new buildings was implemented from 1986 onwards. In Nocera Umbra and Assisi strengthening works have been carried out either by the city council or privately on single buildings, generally improving the behaviour of the building, without upgrading it to the required levels in the code of practice. Under such *improvement* programmes, strengthening generally consists of substitution of timber floors and roofs with lightweight concrete slabs set into ring-beams which are cast into the inner leaf of the bearing walls. In some cases more ties have been introduced, either because of the presence of vaults, or to anchor the head of timber beams onto the façade walls. Some examples of wall strengthening by wire mesh were noticed in the epicentral area. Table 2 gives a breakdown of the frequency of improvement and upgrading within the samples.

Table 2: Extent and types of strengthening in the three town centres

Town	Number of buildings in sample	Traditional ties %	Dressed stone corners %	Improvement %	Upgrading to code level %
Assisi	60	36	30	10	unknown
Nocera Umbra	56	28	26	26	5
Sellano	45	24	17	6	60

2.1.1 Damage assessment

For measurement of damage the survey used the 6-point damage scale of the Europe Macroseismic Scale [Grünthal, 1993], approximately defined in the first two rows of Table 3. The damage distributions for Sellano and Nocera Umbra were similar, while that for Assisi was much lower (Table 3). Although total collapses were few, the number of partial collapses and seriously damaged buildings was significant.

Table 3: Distribution of damage in each of the three town centre samples

Damage type and level	No damage% D0	Light damage% D1	Moderate damage% D2	Serious damage% D3	Partial collapse% D4	Total collapse% D5
Mean damage ratio	0	0.05	0.20	0.50	0.90	1.00
Assisi	22	29	28	16	5	0
Nocera Umbra	5	30	28	16	17	4
Sellano	20	7	37	23	6	7

A more detailed analysis of the data collected on strengthened buildings for Nocera Umbra (Table 4) shows that buildings without any strengthening device experienced, on average, damage D2 or above, with 51% damage D3 and 31% damage D4. Conversely of those buildings with ties and roof strengthening none had damage greater than D2. The majority of buildings strengthened with ties only, whether of older or more recent implementation, show damage D2 (43%) with a significant proportion of damage D3 (27%). 8% collapsed (D5). Although a minority of buildings with roof strengthening and no ties showed a moderate damage D2 (37%), there was an even spread of other level of damage up to D4.

Table 4: Performance of buildings in Nocera Umbra: Sept 1997 Umbria-Marche earthquake

	Damage Level %							
Type of strengthening	D0	D1	D2	D3	D4	D5	Number of buildings	Mean damage ratio
None			17	51	31		29	0.57
Ties		19	43	27	3	8	37	0.33
Ringbeam	12.5	25	37	19	6		16	0.24
Ties and ringbeam	20	4	4				5	0.10
All strengthened	5	22	41	22	4	5	58	0.29

Table 4 shows that the mean damage ratios for strengthened buildings, whether with ties or roof strengthening, are significantly lower than those for unstrengthened buildings, while those buildings with ringbeams and ties performed best.

A similar analysis of the data in Sellano (Table 5) yields the following mean damage ratios: 0.61 for buildings with no strengthening, 0.08 for buildings with ties, 0.30 for building with roof strengthening and 0.30 for buildings with ringbeams and ties. It is worth noticing that the number of buildings with ties in this sample is very small, while the majority of buildings have roof strengthening, and that the intensity was slightly higher.

Table 5: Performance of buildings in Sellano: Sept 1997 Umbria-Marche earthquake

	Damage Level %							
Type of strengthening	D0	D1	D2	D3	D4	D5	Number of buildings	Mean damage ratio
None	25			25	12.5	37	8	0.61
Ties	6		4				5	0.08
Ringbeam	16	12	32	32	8		25	0.30
Ties and ringbeam			66	33			6	0.30
All strengthened	19	8	39	28	6		36	0.27

2.1.2 Vulnerability

An alternative way of presenting damage is as a function of vulnerability. The authors have developed a new limit-state approach to vulnerability assessment of historic buildings based on the resistance of the expected failure mechanism [Spence and D'Ayala, 1997], and taking account of strengthening techniques used. This approach was used for comparison with the damage, building by building, in the Sellano and Nocera Umbra samples. In this method the measure of vulnerability (V) is the reciprocal of the equivalent shear capacity (ESC), where ESC is the horizontal static acceleration level required to trigger the expected collapse mechanism, expressed as a ratio of the acceleration due to gravity. Table 6 summarises this data, indicating that there is a reasonable correlation between damage ratio and vulnerability. Fuller details are given elsewhere [Spence and D'Ayala, 1999].

Table 6: Sellano and Nocera Umbra: damage and vulnerability by vulnerability group

Vulnerability group	Number	Average damage ratio	Standard deviation of damage ratio
Low vulnerability: V<5	48	0.24	0.23
Medium vulnerability: 5<V<10	30	0.21	0.23
High vulnerability: 10<V<20	16	0.69	0.35
Very high vulnerability: V>20	8	0.87	0.11

2.2 The 1998 Azores Earthquake

On the morning of July 9, 1998, the islands of Faial, Pico and São Jorge in the archipelago of Azores were shaken by a m_d = 6 earthquake with its epicentre localised in the northern sector of the canal which separates Pico from Faial. This earthquake, with a focal depth not larger than 10 km, was felt with maximum EMS Intensity 7-9, causing 8 deaths, 150 injuries and important damage to the built housing stock, roads and monuments. About 1500 persons were made homeless. The observed damage is located essentially in the island of Faial, close to the epicentral region, and in the island of Pico. In São Jorge the extent of the damage was much smaller. [Oliveira and Malheiro, 1999]

The consequences of this event are of great interest. It was the third serious earthquake to occur in the area this century. After the events of 1926 and 1973, the building stock was repaired using different techniques. In the 1998 earthquake it was observed that the repaired buildings behaved differently as compared with the unrepaired buildings.

In the island of Faial as a whole, from a total of about 3950 housing units, 54% experienced no damage and 10% suffered total collapse; the remaining 36% had some type of damage statistically distributed with a mode around the 25% damage ratio.

The damage inflicted to the housing stock was very selective. The older, rubble, 1-2 storey high masonry units, in the rural areas between 5 and 10 km from the epicentre, were very much affected by the shaking. In some places almost 100% collapsed. In the town of Horta, 15 km away from the epicentre, the housing stock of 3-4 storey high old masonry suffered moderate damage with a slight opening of walls towards the exterior.

Horta suffered intensity EMS 6. Most buildings in the centre of Horta date from the last century. They are 3-4 storeys high. After the 1926 earthquake, tie-rods (rods with screw bolts) were applied. These were much more successful than the collar beam technique used after the 1973 earthquake. Collar beams were widely used but in an incomplete way; they were placed only in the walls with damage and did not close the loop.

Throughout Horta, masonry is of relatively poor quality. It is made of volcanic tuff, 66 cm thick almost constant in height, partially organised, with a modulus of elasticity E=0.1 to 0.3 GPa, depending on the quality. Large window openings are present, attaining a ratio of 1:1 along the length of the street facades. Floors are made of wooden beams simply supported inside the walls. Roofs are supported by wooden gable structures.

The damage which occurred in Horta had the following features:

• There were no collapses.

• A few buildings need a great deal of repair as the peripheral walls opened up several centimetres per storey, by rotating to the exterior around the foundation. These buildings suffered interior damage: plaster failed in walls and ceilings and tiles dislodged.

• There was moderate damage in tens of houses including shear cracking of exterior walls, collapse of parapets and deformation of walls out of plane.

Statistical data on damage in the island as a whole and in the three districts of Horta is shown in Table 7.

Table 7: Summary of damage for Horta: July 1998 Azores earthquake

District	No damage (%)	Light damage (%)	Moderate damage (%)	Heavy damage or collapse (%)
All Faial	54	10	17	20
Horta- Angustias	78	11	10	1
Horta- Conceição	75	10	10	5
Horta-Matriz	65	17	18	1

Unfortunately, detailed analysis of the data, correlating damage to the type of structure, has not been completed at the time of submission of this text, but will be reported in due course. It is expected to confirm the impressions reported above, namely that, while strengthened buildings performed better than unstrengthened buildings generally, those strengthened with ties (post 1926 earthquake) performed better than those strengthened with collar or ring beams (post 1973), despite the greater age of the intervention.

2.3 The 1980 Irpinia Earthquake: the TOSQA Project

In 1986 a large survey of the buildings in the Historic Centre of Naples was carried out by the Italian GNDT to establish data to estimate their earthquake vulnerability. The survey included some 700 buildings representing roughly 10% of the whole Historic Centre, much of which dates from the Viceregal period from 1501 to 1734 AD [Barratta et al, 1996].

The data collected on each building included information on the form of construction, floor by floor, divided between 11 vertical structural typologies and 8 horizontal structural typologies, and also indicated the existence of strengthening techniques (mostly iron tie-rods). The database also contains information on evidence of earthquake damage to each building. It can reasonably be assumed that observed earthquake damage in the 1986 survey related to damage from the 1980 Irpinia earthquake; the intensity of the earthquake in the Historic Centre was about EMS = 7. In 1986 many buildings were still unoccupied and undergoing or awaiting repair following the 1980 earthquake. It can also be assumed that where the observed strengthening was with tie rods, this had taken place before the 1980 earthquake, since any strengthening after that event would have used reinforced concrete techniques; indeed most of the tie-rods observed in Naples were introduced following the previous disastrous earthquake of 1930 which damaged buildings throughout the city.

In the GNDT database, damage level has been assessed according to a five-point scale, each point of which corresponds to the following damage ratio: slight damage = 0.2, moderate damage = 0.4, heavy damage = 0.6, partial collapse = 0.8, collapse = 1.0.

The EU-funded TOSQA Project (EV5V-CT93-0305, 1994-1996), [Spence, 1996] made use of the GNDT database for a more detailed investigation of the performance of buildings in a more limited case study area of 270 of the previously studied 700 buildings for two purposes:

• to compute the average damage on all buildings within a particular case study area according to the last type of strengthening observed.

• to investigate whether the presence of tie-rods at floor level correlates with reduced average damage.

The GNDT database identifies a number of different categories of strengthening intervention, including “raising”, “restoration” and “repair”, which are not specifically related to earthquake resistance action, and “seismic repair” and “seismic strengthening” which are specifically designed for earthquake resistance. It also identifies the particular type of intervention visible. It is thus possible to compare the performance of buildings with no strengthening with those strengthened in any way, those strengthened specifically for improving seismic resistance, and those improved with tie-rods. Table 8 gives the results of the analysis.

Table 8: Comparison of average damage in the Historic Centre of Naples after the Dec 1980 Irpinia earthquake, strengthened and unstrengthened buildings

Type of strengthening	Number of buildings	Average damage ratio (DR)	Standard deviation on DR
None	28	0.233	0.181
All types	251	0.063	0.081
Seismic strengthening only	113	0.057	0.063
Tie rods only	159	0.091	

Table 8 clearly shows the performance of buildings which had undergone some type of strengthening to have been very much better than those without any improvement; those (about 40% of the sample) with seismic strengthening performed even better. The use of tie rods produced an improvement which was not as good as that of all seismically strengthened buildings, but still very much better than that of buildings with no seismic strengthening.

The database confirms the generally held view that the widespread introduction of iron tie-rods following the 1930 earthquake resulted in a considerable reduction in the damage caused to the Historic Centre of Naples from the 1980 earthquake.

DISCUSSION

Limitations

Before presenting any conclusions from these studies, some limitations need to be pointed out.

1. Even though this is a considerably larger dataset than was available five years ago, the number of buildings in each location whose performance has been assessed is still rather small, and insufficient for statistically robust conclusions to be drawn.

2. All the example surveys are drawn from areas which have been affected by earthquakes of moderate intensities, ranging from EMS = 6 for Horta to EMS = 8 for Sellano. The effectiveness of strengthening when intensities reach EMS = 9 or 10 remains untested.

3. The three surveys reported are not entirely consistent with each other in terms of aims, timing in relation to the earthquake, or methodology of damage assessment; in particular the formal GNDT methodology of damage assessment, though used as a guide, was not followed in every respect.

4. In this paper the results of each survey have been presented as a damage ratio, but there are somewhat different approaches to determining state of damage and calculating damage ratio between the two Umbria surveys and the Naples survey.

5. The principal variables which are examined in this paper are whether the building has been strengthened or not, and by which method the strengthening has been carried out. Surveys were restricted to a limited area, and all masonry buildings in the area at the time of the survey, whether damaged or not, have been included. It can be expected that other important variables such as age of the buildings, condition and degree of maintenance and ground conditions will have had as important effects on the performance of the buildings as that of strengthening. These have not been separated, because, even where this is possible, the resulting sample sizes would then have been very small.

3.2 Effectiveness of strengthening

The evidence from the three Italian surveys for which statistical data is available strongly supports the conclusion that prior strengthening reduces damage in earthquakes at these intensity levels. In the most recent cases, for the 1997 Umbria-Marche earthquake, the damage ratio has been reduced from 57% to 30% in Nocera Umbra, and from 61% to 30% in Sellano, comparing unstrengthened with strengthened buildings. In the Naples survey, seismic strengthening reduced the damage survey from 23% to 6%.

In terms of threat to human life, the most important measure of damage is the proportion of buildings suffering partial or total collapse. In Sellano and Nocera Umbra combined this proportion was 35% for unstrengthened buildings and 7% for strengthened buildings. In Naples the statistics are complicated by the delay between the earthquake and conducting the survey. The low level of damage suggests that there would have been few if any collapses, but any collapses that had occurred would have been removed before the survey and thus would not have appeared in the database.

3.3 Comparison of different strengthening techniques

Because numbers of buildings in each sample are smaller, the comparison of the performance of buildings strengthened by different techniques is even more uncertain. In the Naples study, it appears that the performance of buildings with tie-rods only was slightly worse than that of buildings with other types of seismic strengthening or seismic repair. In the Nocera Umbra survey, it appears that buildings strengthened with ringbeams only had a slightly better performance than those strengthened with tie-rods only; by contrast, in Sellano, the buildings strengthened with ringbeams performed on average worse than those strengthened with tie-rods only, although the number of the latter group was very small. Finally, in Horta, it was reported, though without statistical evidence as yet, that the buildings reinforced with tie-rods performed better than those with ring-beams. This is explained by the fact that the ringbeams on the two main bearing walls were not interconnected through the crosswalls.

On the basis of the available data it is therefore is difficult to assess the relative efficiency of one technique by comparison with the others. This in general is highly affected by the original fabric of the building and its state of conservation and by the accuracy and care in the implementation of the strengthening devices. In fact anomalous cases of partial or total collapse, where ties or roof strengthening techniques have been used, are always associated with lack of proper detailing, such as insufficient anchorage and connection with the rest of the masonry structure, or in the case of ties, insufficient number or irregular location.

However, there is no convincing evidence that the more modern and more expensive technique of reinforcing with concrete ring-beams has performed better than the older system of reinforcing with tie rods. This has implications for the cost-effectiveness of strengthening strategies.

The study in Sellano gave an indication of the possible reasons for this. Most buildings in this village had been reinforced prior to the earthquake with ringbeams at first floor and roof level, and with roof-level slabs; but substantial damage nevertheless occurred as a result of shear failure of the principal load-bearing walls. Without a programme of strengthening walls, the wall shear resistance therefore represents an upper limit to what can be achieved by strengthening; the evidence suggests that this resistance can be mobilised as effectively by the use of tie-rods as with a concrete ring-beam. As well as being cheaper, tie-rods are less intrusive into the existing masonry structure, and make less impact on the external appearance of the building.

3.4 Cost-effectiveness of different strengthening techniques

Under the TOSQA Project [Spence, D'Ayala et al, 1997] a compilation was made of the costs of a range of different strengthening techniques, and estimates were made of the reductions in vulnerability resulting from some of these techniques. Using a group of old masonry buildings in the city of Lisbon as a case study, it was estimated that a programme of strengthening using tie-rods alone would reduce the average damage ratio at intensity EMS = 7 from 29% to 14% and at intensity EMS = 8 from 52% to 40%. It was calculated from this that the payback period for a programme of strengthening with tie-rods would be between 5 and 15 years, depending on the seismicity model assumed.

It was also shown, using recent Italian data, that the cost per m^2 of strengthening with insertion of ringbeams (at \$20 / m^2), was about 2.5 times higher than the cost of strengthening with tie-rods. Thus a substantially greater reduction in expected losses would be needed to achieve an equivalent payback period for the investment in strengthening. Although no cost data is available for the buildings in the present damage surveys, we can note that the reduction of losses achieved by strengthening in the intensity 7 areas is greater than estimated for the Lisbon buildings, while the performance of buildings strengthened with tie-rods or concrete ringbeams is not much different.

Thus two tentative conclusions are:

1. In an area where the expected return period of intensity EMS = 7 is no greater than 100 years, the payback period for a programme of strengthening using tie-rods can be expected to be not greater than 15 years.

2. The additional cost of strengthening with reinforced concrete ringbeams is not justified, since there is no evidence that the loss-reduction achieved by this method is any greater than that achieved with tie-rods.

CONCLUSIONS

The results presented on the actual performance of strengthened old masonry structures in earthquakes constitute a significant increase in the data previously available on this subject.

Broadly these results, though not of uniform quality or constituting a large database, do confirm the indications of previous studies, both theoretical and laboratory derived, that strengthening is effective, reducing average damage ratios at intensity EMS = 7 to 8 by about 50%, and reducing the probability of collapse to a low level.

They also suggest that the performance of buildings reinforced with tie-rods prior to the 1980s is not very much different from the performance of buildings reinforced more recently with reinforced concrete ring-beams. Thus the additional cost of strengthening with reinforced concrete ring-beams compared with reinforcing with tie-rods may not be justified.

The study has thus lent further weight to the view expressed in an earlier paper [Spence, D'Ayala et al, 1994] that the use of tie-rods for strengthening old masonry buildings is cost-effective, makes minimal intrusion into the existing structure, has the least possible impact on the external appearance of buildings, and should be encouraged. The conclusion has importance for the many thousands of masonry buildings in the older areas of European towns and cities which will need to be strengthened in the future if major destruction and loss of life is to be avoided.

REFERENCES

Barratta, A., Colletta, T., and Zuccaro, G. (1996), *Seismic Risk of Historic Centres: a Preliminary Approach to the Naples Case*, Istituto Italiano per gli Studi Filosofici, Naples.

D'Ayala, D., Spence, R., Oliveira, C. and Pomonis, A. (1997), "Earthquake Loss Estimation for Europe's Historic Town Centres", *Earthquake Spectra*, Volume 13, No 4, pp 773-793, November 1997.

Grünthal, G. (1993), *European Macroseismic Scale 1992*, European Seismological Commission, Luxembourg.

Oliveira, C.S. and Malheiro, A.M. (1999), "The Faial-Pico-Sao Jorge Azores Earthquake of July 9, 1998", *Second International Conference on Geothermal and Earthquake Engineering*, Lisbon, June.

Spence, R., D'Ayala, D., Coburn, A. and Pomonis, A. (1994), "Earthquake Protection of Historic Town Centres", *Proceedings of the Tenth European Conference on Earthquake Engineering*, pp 891-896, Vienna.

Spence, R. (ed) (1996), *The TOSQA Project: Earthquake Protection for Historic Town Centres, Final Report*, The Martin Centre, Cambridge University.

Spence, R. and D'Ayala, D. (1999), "The Umbria Marche Earthquakes of September 1997: Damage Assessment and Preliminary Analysis", *Structural Engineering International* (to be published).