Seismic Analysis and Frp Jacketting of 4 Storey Rc Building

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ABSTRACT
In this paper, the specific details of a 4-storey, 3-bay reinforced concrete frame test structure with unreinforced brick masonry (URM) infill walls are described along with estimates of its likely weaknesses with regard to seismic loading. The construction details for this building are typical of construction more than 40 years old in Mediterranean European countries. The concrete frame is shown to be essentially a "weak-column strong-beam frame" which is likely to exhibit poor post- yield hysteretic behaviour. Based on the results of an extensive literature review, the building is expected to have maximum lateral deformation capacities corresponding to about 2% lateral drift. The unreinforced masonry infill walls are likely to begin cracking at much smaller lateral drifts, of the order of 0.3%, and to completely lose their load carrying ability by drifts of between 1% and 2%. Three seismic retrofit schemes were identified, based on the literature review, for further study. The effectiveness of each retrofit scheme will be tested using full-scale pseudo-dynamic tests at the European Laboratory for Structural Assessment (ELSA) at the European Commission's Joint Research Centre in Ispra, Italy. The results of the detailed analyses and tests will be reported elsewhere.

INTRODUCTION

There has been much research on the topic of seismic retrofit of structures in recent years. Attention has been focussed world-wide on both building and bridge structures and with the widespread damage to older structures in the relatively recent Loma Prieta, Northridge, and Kobe earthquakes, owners are increasingly taking action to prevent similar damage to existing structures in future earthquakes.

The purpose of the present study was to investigate possible seismic retrofit options for use in a project using full-scale pseudo-dynamic tests at the European Laboratory for Structural Assessment (ELSA) at the European Commission's Joint Research Centre in Ispra, Italy. This project is being conducted as part of the overall European effort to develop seismic retrofit guidelines in the form of Part 1-4 of Eurocode 8 (CEN, 1998). To that end, the present study investigated potential seismic retrofit techniques for use in a "typical" reinforced concrete frame building with brick masonry infill walls. While the building was designed according to the state- of-the-art over 40 years ago, it does not meet the present day seismic design requirements. The building is representative of many other buildings of its era constructed in European Mediterranean countries such as Greece, Italy and Portugal.

LITERATURE REVIEW

In the course of this investigation, a review of the broader literature in the area of seismic rehabilitation was undertaken (Griffith and Pinto, 1999). Over 200 papers, mainly journal articles and World Earthquake Engineering Conference papers and European Conference on Earthquake Engineering papers, published since 1980 were reviewed as part of this project. In the interest of space, only a brief summary is given here.

A review of the current EC8 practice in repair and strengthening of concrete structures in Europe (Elnashai and Pinho, 1998) discussed the need for the design philosophy underpinning the assessment and strengthening of buildings to be consistent with that for new buildings. While different design target performance limits may be allowed for new and existing construction, the basic design philosophy should be consistent. It was concluded that there is a need to explicitly include deformation-related performance objectives in retrofit design guidelines in view of the trend towards deformation-based seismic design of new structures. Consequently, the lateral drifts and structural deformations reported for concrete frames and masonry infill are specifically summarised here.

Considering first the concrete frame on its own, it may be expected that it will withstand a lateral drift of the order of 1.5% to 2% before the beam-column joints and/or columns fail (Beres et al, 1992a). The maximum base shear strength for the concrete frame will likely be of the order of 15% of its weight and occur at roughly 1% drift (Bracci et al, 1995a,b).

As for the masonry, the ultimate strength of the

masonry infill may be estimated using a value for the maximum shear strength of URM of $\tau_u = 0.4MPa \pm 0.2MPa$. There is a wide range in the values reported in the literature for the shear strain (or lateral drift angle) at which the maximum shear stress occurs. Nevertheless, based on the literature it appears that this maximum stress may be estimated to occur at a lateral drift angle of approximately 0.3% (see for example Pires and Carvalho, 1992; Valiasis et al, 1993; Fardis and Calvi, 1995; Zarnic and Gostic, 1997; Schneider et al, 1998).

Turning to failure modes, a recent review by Dyngeland (1998) found that the most common failure mechanisms for concrete buildings due to seismic loading are: (1) beam-column joint failures; (2) column failure due to inadequate flexural or shear strength; (3) shear wall failure; or (4) infill wall failure due to inadequate shear strength or inadequate out-of-plane flexural strength. Bruneau (1994) provides a similar overview of the seismic vulnerability of masonry (brick and concrete block) buildings and their most common failure mechanisms. Of particular interest to this project are those due to in-plane forces.

Many of the structural failures during earthquakes in the early 1970s were due to inadequate shear strength and/or lack of confinement in concrete columns. Hence, early column strengthening procedures typically involved increasing the concrete column's cross-section. The main problem with this approach is that it often unacceptably increases the dimension of the column, rendering the retrofit impractical. The use of thin carbon fibre composite sheets avoids this problem and has consequently gained acceptance over the past 10 years. It was noted that steel and composite jacketing was particularly useful for correcting inadequate lap splice problems and that anchor bolts can be used to improve confinement away from the corners of rectangular columns (eg, Priestley et al, 1994; Saadatmanesh et al, 1997). Nevertheless, concrete jacketing of concrete columns has been shown to be very effective in improving strength and ductility and converting strong-beam weak-column buildings into buildings with a strong-column weak-beam mechanism (Choudhuri et al, 1992; Rodriguez and Park, 1994; Bracci et al, 1997; Bush et al, 1990).

Where the masonry infill is susceptible, it can be retrofit in a wide variety of ways. Crack injection grouting is often used to return a masonry wall to its "original" condition whereas the use of so-called "jacketing" techniques adds both strength and stiffness to the infill. In this context, jacketing consists of encasing the existing element by an additional structural component. For masonry infill walls, jacketing can take the form of:

- shotcreting – the application by spraying a thin layer of concrete onto the face of the brickwork. Reinforcement may or may not be attached to the brickwork before spraying;
- prefabricated reinforced concrete panels attached, normally, with dowels through the brickwork;
- steel plates or fibre composite sheets glued/bonded onto the brickwork; or
- steel strip bracing attached to the brickwork using either through-bolting or some form of chemical bonding agent.

While recent research has focussed on the use of advanced fibre composites, energy dissipation and seismic isolation devices for the seismic retrofit of buildings, the more traditional methods should not be overlooked when considering which system(s) to employ. In practice, the optimal scheme will depend upon many factors, some of which are non-technical such as aesthetics and the level of disruption to occupants (Jirsa, 1994)

DESCRIPTION OF ELSA TEST BUILDING

The ELSA test building is a 4-storey, 3-bay reinforced concrete frame with unreinforced brick masonry infill walls. The concrete frame was designed for gravity loads and a nominal lateral load of 8% of its weight, W (Carvalho, 1998). The reinforcement details were specified to be representative of buildings constructed over 40 years ago in European Mediterranean countries such as Italy, Portugal and Greece. In the following sections, the building is described and the results of a preliminary assessment of its likely seismic behaviour are presented.

Frame Geometry, Section Details and Material Properties

The dimensions of the building and section details are shown in Figures 1 – 3. It can be seen in the elevation and plan drawings (Figure 1) that the storey heights are 2.7m and there are two 5m span bays and one 2.5m span bay. Brick masonry infill (200mm thick) is contained within each bay. The left-hand bay contains a window (1.2 x 1.1m) at each of the 4 levels. The central bay contains a doorway (2.0 x 1.9m) at ground level and window openings (2.0 x 1.1m) in each of the upper 3 levels of the building. The right-hand (2.5m span) bay contains solid infill. The beam reinforcement details are shown in Figure 2. It should be noted that the longitudinal reinforcing steel consists of smooth round bars which are terminated with 180° bends. All beams in the direction of loading are 250mm wide and 500mm deep. The transverse beams are 200mm wide and 500mm deep. The concrete slab thickness is 150mm. The column reinforcement details are shown in Figure 3. The

column stirrup detail with a 90° curtailment bend should be noted in particular. Preliminary calculations have been carried out in order to establish which failure mechanisms are most likely to occur under seismic loading. In order to do this, the mean values for the respective material strengths shown in Table 1 were used.

Table 1 – Material properties (mean values).

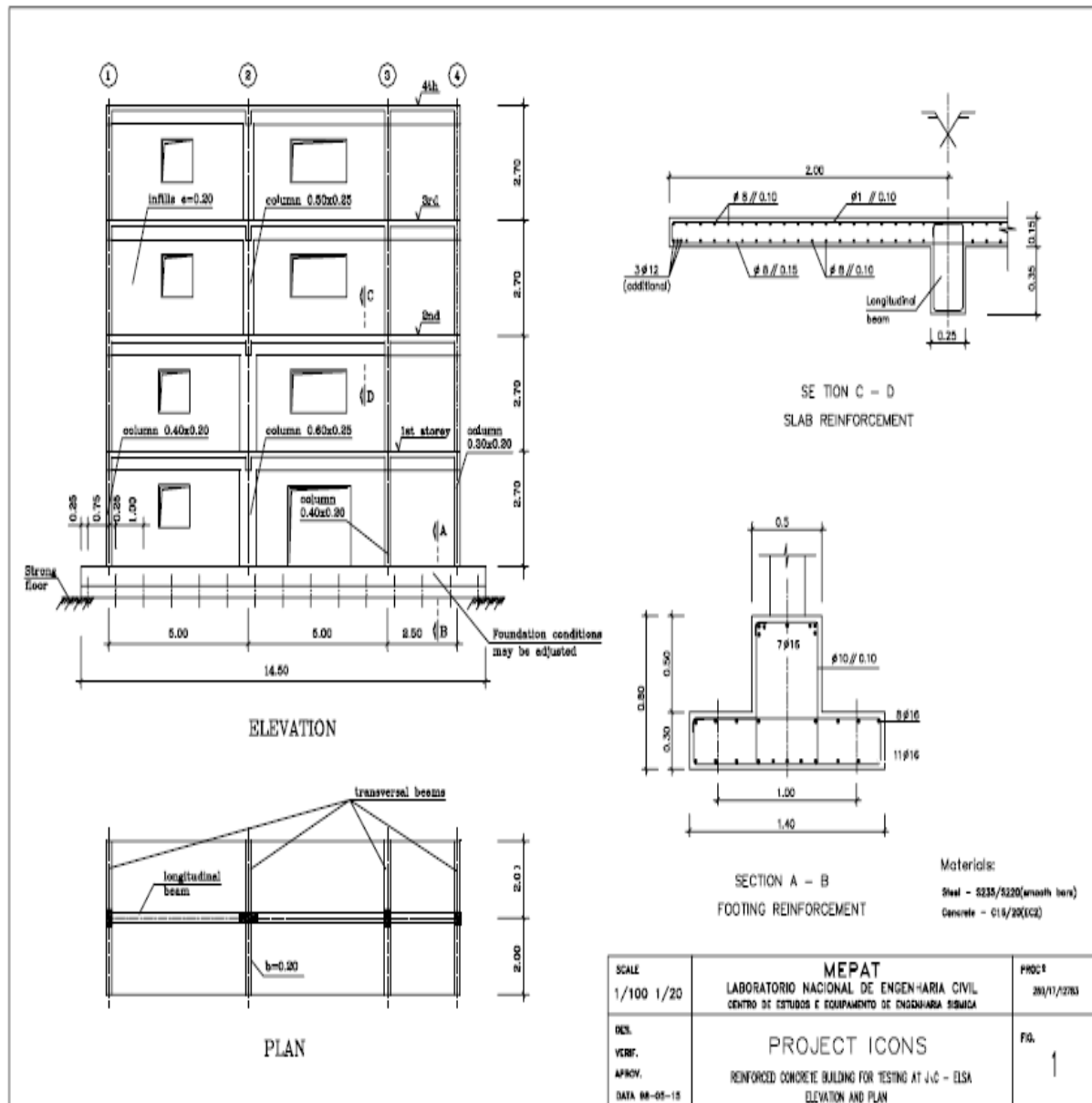| Steel	$f_{sy} = 235 MPa$, $E_{st} = 200x10^3 MPa$
Concrete	$f_c' = 24 MPa$, $\varepsilon_{cu} = 0.003$, $E_c = 20x10^3 MPa$
Brick Masonry	$\tau_u = 0.4 MPa$, $\gamma_u = 0.003$

Figure 1 – Plan and elevation views of concrete frame plus masonry infill building.
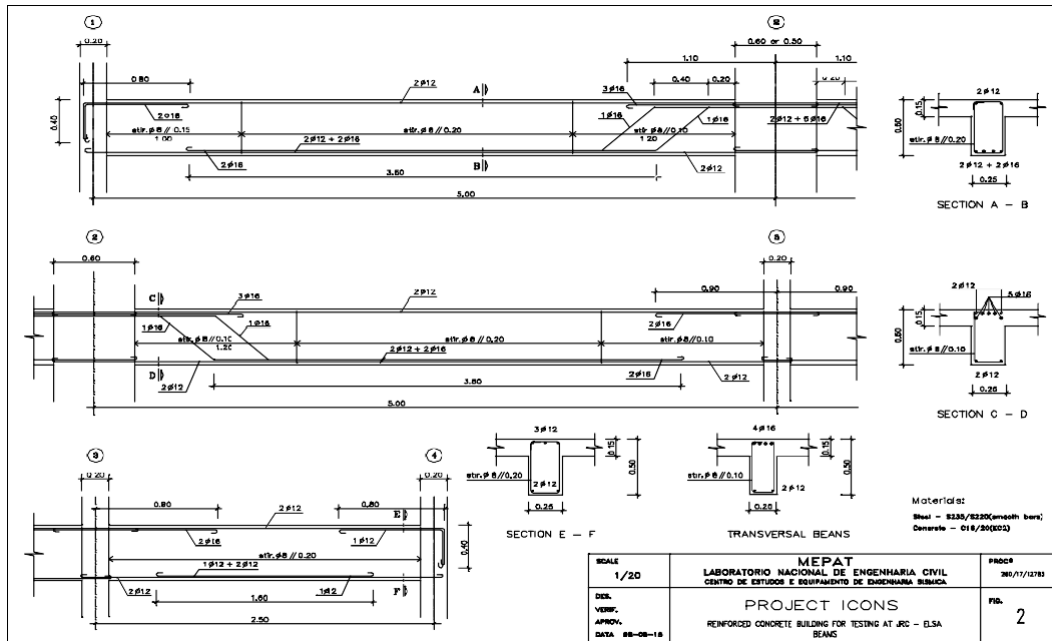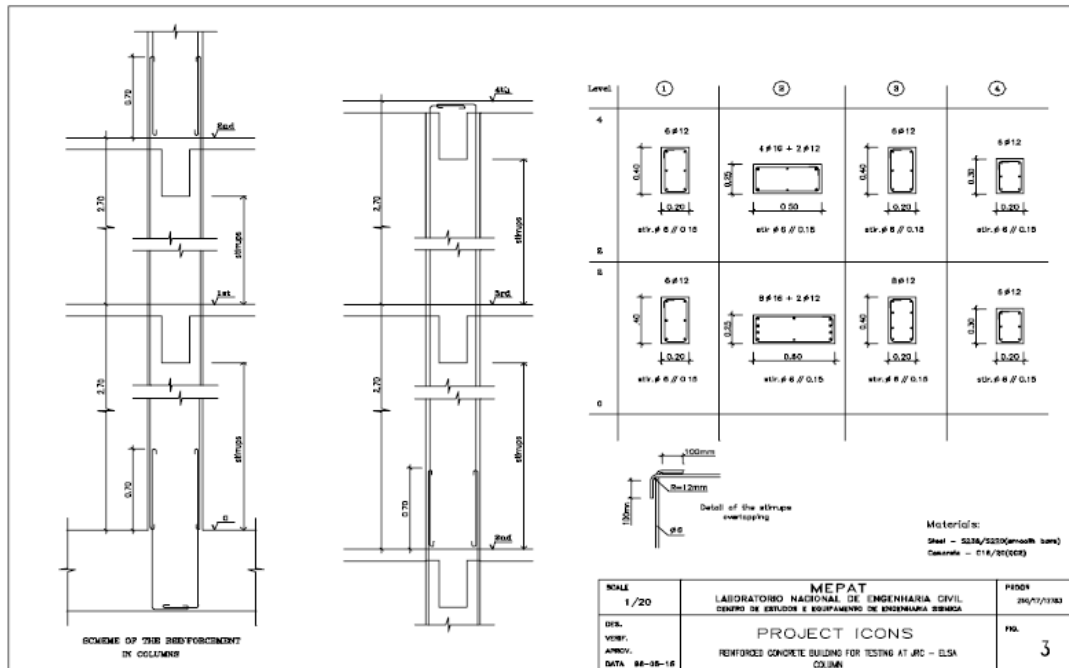
Figure 2 – Beam reinforcement details.

Figure 3 – Column reinforcement details.

Frame Sidesway Potential and Column Shear Strength

The ultimate moment capacity, M_u, was first calculated for each beam and column cross-section using conventional rectangular stress-block theory and the mean values for the respective steel yield and concrete compression strength properties shown in Table 1. In order to assess whether a column sidesway mechanism was likely to occur, the sum of the moment capacities of the columns at each level were divided by the sum of the moment capacities of the beams at each level using equation (1).

$$\frac{\sum M_{u,columns}}{\sum M_{u,beams}} \quad (1)$$

Even though the slab contribution to the beam capacity was ignored, the value given by equation (1) was still less than one for each level of the structure, indicating that the structure is highly susceptible to column sidesway collapse. (Note: in practice, the value given by equation (1) should be markedly greater than 1, say 1.4 for example, to ensure that a column sidesway mechanism is not likely to occur.)

The shear capacity of each column, V_u, was then

estimated to determine whether the columns were likely to suffer shear failure before reaching their maximum flexural strength. These calculations indicated that no columns are expected to suffer premature shear failure.

Masonry Infill and Concrete Frame Shear Strength and Stiffness

Next, the relative shear strength of the masonry infill walls were estimated and compared to the estimated ultimate shear strength for each storey of the bare concrete frame. The values suggest that the ultimate strength of the brick walls is approximately four times that of the bare frame. Of course, the column storey shears will not achieve their maximum at the same lateral drift as will the masonry since the lateral stiffness of the brickwork is also much greater than that for the frame. The retrofit scheme therefore needs to be capable of accommodating the differences in both strength and stiffness. It should also be noted that the shear strength of the concrete frame above level 2 is only 60% of the frame's shear strength below level 2.

Section and Joint Details

There are two main aspects of concern for the beam-column details shown in Figures 2 and 3. The first is related to the possibility of premature joint failure due to poor anchorage of the bottom beam steel that terminates in the beam-column joint region. The behaviour of smooth round bars with 180° bends in the joint region is likely to be better than the behaviour observed by Beres et al (1992) who tested joints with deformed bars which terminated in the joint region without any bends. In those tests, exterior joints failed at lateral drifts of between 1.5% and 2% of the storey height. Interior joints failed at lateral drifts between 2% and 2.5%. Therefore, it is expected that the joints in the test building will perform adequately, at least up to drifts of 2% to 2.5%.

However, the use of the 90° stirrup "overlapping details" (Figure 3) in the curtailment of the shear reinforcement for the beams and, more particularly, for the columns is another matter. Experience has shown that columns will collapse if their stirrups are not located sufficiently close to confine the concrete core and prevent buckling of the longitudinal steel. The stirrup spacing used in this project is 150mm in all columns ($s = 10d_b$ or $12.5d_b$) and either 100mm or 200mm in the beams ($s = 8.3d_b$ or $16.7d_b$). These are remarkably close spacings in view of the age of the design. Nevertheless, the stirrup detail shown in Figure 3 is not expected to withstand repeated large cycles of lateral loading once concrete crushing has occurred.

Summary

In summary, the main issues to be addressed by potential seismic retrofit schemes are:

- The large difference in strength and stiffness between the concrete frame and the brick masonry infill. For example, the bottom storey of masonry infill is estimated to reach its ultimate shear strength of about 660kN at a storey deformation approximately 0.3% drift. On the other hand, the bottom storey of the frame is estimated to reach its ultimate strength of 150kN at a drift of between 1% and 2%.

- The large change in strength and stiffness of the frame at level 2. The strength of the bare concrete frame above level 2 is estimated to be only 60% of the strength of the frame below level 2. Similarly, the lateral shear stiffness for the frame storeys above level 2 are estimated to be 65% of the stiffness for the frame storeys below level 2.

- The seismic behaviour of the bare concrete frame is likely to be that of a "weak-column, strong-beam" mechanism under ultimate deformation conditions.

- The stirrup curtailment detail (Figure 3) which uses 90° hooks is unlikely to withstand repeated cycles of large deformation (say ±2% drift).

RETROFIT STRATEGIES

In this section a number of seismic retrofit strategies are discussed. Each has its relative merits. For example, the retrofit choice will depend to a large extent upon the seismic performance level that is required during the design basis earthquake (DBE). Predominately elastic response so that the masonry infill walls are protected during the DBE will require a much different retrofit than if the walls are allowed to fail to permit ductile moment or braced frame behaviour. Hence, several retrofit options were considered for this project.

Option 1: Replacement of URM infill with damped bracing

In this option, the URM infill would be replaced with K-bracing in the 2.5m span bay at each level of the building. The bracing would incorporate energy dissipation devices that would help reduce the seismic demands from the levels corresponding to the 5% damped design spectrum for the DBE. With the addition of the damped bracing, the frame can be designed to provide more uniform storey stiffnesses and strengths and the energy dissipation devices can be designed to "yield" at appropriate force amplitudes. In this way, ductile bracing can limit the forces that they attract and help limit the maximum base shear reaction. In practice, any retrofit solution that

increases the base shear reaction will incur additional expenses due to the need to improve the building's foundation.

Option 2: Composite jacketing of columns and selected masonry infill

Composite jacketing can be used to strengthen the columns and infill walls and to improve the ductility of the columns and the post-cracking behaviour of the URM infill walls. However, this retrofit option will cause a modest increase in the lateral strength and stiffness of the building. If the consequences of this are acceptable (e.g. foundation strengthening) then the composite jackets should be easily capable of addressing the potential column stirrup weakness and the poor post-cracking behaviour of the masonry infill. Since the jacketed infill would be able to carry sizeable loads after cracking, the change in strength and stiffness of the concrete frame itself would not be critical and probably need not be specifically modified.

Option 3: Retrofit of concrete frame elements only

In this option, only the concrete frame elements would be repaired. The masonry infill would essentially be ignored. The assumption being that the deformations in the building during the DBE would be between 1.5% and 3% drift and that the URM would have completely failed at that point. The likely maximum sustainable drift for the concrete frame was estimated to be approximately 2%. With this in mind, the column hinge zones would need to be confined (composite or concrete jacketing) to maintain confinement during large reversals of displacement (in excess of 1.5% drift). Furthermore, the change in strength and stiffness at level 2 must be addressed in this option.

CONCLUSIONS

In summary, a comprehensive literature review was performed in order to gain a better insight into the key issues relevant to seismic retrofit of concrete frame buildings with unreinforced masonry infill walls. Based on this review, it was concluded that buildings having details typical of construction of more than 40 years ago in Mediterranean European countries are likely to have maximum lateral deformation capacities corresponding to about 2% lateral drift. The unreinforced masonry infill walls are likely to begin cracking at much smaller lateral drifts, of the order of 0.3%, and to completely lose their load carrying ability by drifts of between 1% and 2%.

In this paper, the specific details of a 4-storey, 3-bay reinforced concrete frame test structure with unreinforced brick masonry (URM) infill walls were described and estimates of its likely weaknesses with regard to seismic loading were outlined. The concrete frame was shown to be essentially a "weak-column strong-beam frame" which is likely to exhibit poor post-yield hysteretic behaviour. To make matters worse, there is a decrease in strength and stiffness of the concrete frame of the order of 35-40% at level 2. This particular problem is not critical as long as the URM infill retains its load-carrying capacity since the strength and stiffness of the URM infill is estimated to be much larger than that of the frame. However, even if the infill were designed to respond elastically in the DBE its ductility is much worse and in the event of a larger than expected earthquake the infill strength is likely to be lost. Hence, three retrofit options were selected for further detailed investigation.

The effectiveness of the retrofit schemes eventually selected from among the options discussed here will be tested using full-scale pseudo-dynamic tests at the European Laboratory for Structural Assessment (ELSA) at the European Commission's Joint Research Centre in Ispra, Italy. The results of the detailed analyses and tests will be reported in future publications.

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