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Seismic Retrofit of RC Frame Buildings with Masonry Infill Walls: Literature Review and Preliminary Case Study

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ABSTRACT

There has been a substantial increase in the topic of seismic retrofit of existing buildings in recent years as evidenced by the growing number of research papers published in this area. Attention has been focussed world-wide on both building and bridge structures and with the widespread damage to older buildings and bridge structures in the relatively recent Loma Prieta, Northridge, and Kobe earthquakes, owners have begun to take 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 the seismic upgrade of a reinforced concrete frame building with brick masonry infill walls. The building is typical of a Mediterranean European country (e.g., Greece, Italy, Portugal) and while designed according to the state-of-the-art over 40 years ago, it does not meet the present day seismic design requirements and contains a number of now "well-recognised" seismic design deficiencies and problems such as:

- inadequate beam-column joint details (discontinuous, inadequately anchored bottom beam steel and inadequate shear reinforcement);
- inadequate confinement of columns (stirrups with 90° bends and spacing of $10d_b$ to $12.5d_b$);
- inadequate column splice joint details;
- weak-column strong-beam frame collapse mechanism; and
- brick masonry infill wall interaction with frame response.

The overall aim of this project was to identify the optimal combination of retrofit options that would enable the building to meet the present-day "life-safety" performance criteria for buildings subject to a design magnitude earthquake. As part of this study, a detailed review of the broader literature in the area of seismic rehabilitation was undertaken in conjunction with a preliminary assessment of the building's seismic capacity. Based on these findings, a number of retrofit schemes will be investigated analytically in order to identify the most suitable course of action.

In the present paper, a summary of that literature review is given, followed by the results of the preliminary assessment of the seismic resistance of the building, and a description of several seismic retrofit scheme options for further detailed study. The effectiveness of the retrofit scheme eventually selected from among the options discussed here will be tested using full-scale pseudo-dynamic tests at the ELSA laboratory of the European Commission's Joint Research Centre in Ispra, Italy. The results of these detailed analyses and tests will be reported in future publications.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

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This research was 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 possible seismic retrofit techniques for use in the seismic upgrade of 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.

In the course of this investigation, a review of the broader literature in the area of seismic rehabilitation was undertaken. 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. Hence, the rather lengthy review. On the other hand, many more papers than those reviewed have been published on this topic. In view of the timetable for the project, however, not all of these papers could be reviewed. Nevertheless, some of these publications have also been listed in the Bibliography, Appendix B. Due to the large amount of information covered, the most important quantitative results of this part of the study are summarised in tabular form in Tables 1 and 2 in Sections 2.3 and 2.4, respectively. An overall summary of the literature review is also given in Section 2.10 of this report.

The particular building of interest is described in detail in Section 3. A simple preliminary analysis of the building's likely behaviour under seismic overload conditions is then presented. The methods of "analysis" used were necessarily simple; mostly "plastic" collapse analysis and simple statics methods. The relative strengths and stiffnesses of the concrete frame and brick masonry infill walls were calculated in order to gain an insight into the most appropriate seismic retrofit options for the building. The construction details (beam-column joints, column splice joints, and stirrup spacing and curtailment) were discussed in the context of the results of the literature review which indicated deformation levels at which the various details might be expected to fail. Section 3 concludes with a "prediction" of the lateral drifts at which the building might be expected to reach various damage states, including collapse.

In Section 4 several options for the seismic retrofit of the masonry infilled RC frame building are presented. The options cover a range of seismic performance levels. Further work is required to perform detailed analyses and design of each option in order to identify the most appropriate seismic retrofit option. The report concludes in Section 5 with an overall summary of the results of the literature survey and retrofit recommendations.

2. LITERATURE REVIEW

There are many seismic strategies for retrofit in use and/or under development. For the purposes of the present investigation, the literature was broken down into 8 areas. These were: (1) seismic assessment; (2) frame plus infill behaviour; (3) beam-column joints and connections; (4) column strengthening; (5) bracing with or without energy dissipation; (6) masonry strengthening; (7) walls; and (8) seismic isolation.

Before discussing each of these topics in detail, a general overview of the topic and literature is given in Section 2.1. Detailed discussion of aspects relevant to the seismic retrofit of concrete frame buildings with masonry infill walls is then given in Sections 2.2 – 2.9. A bibliography of publications relevant to seismic retrofit of concrete/masonry buildings is included at the end of this report.

2.1 Introduction

A review of recent publications relevant to the retrofit of bridges and building structures (Dyngeland, 1998) indicated that the most common failure mechanisms for concrete buildings due to seismic loading are:

- beam-column joint failures due to
  - inadequate joint reinforcement and/or
  - improper anchorage of longitudinal beam reinforcement;
- column failure due to
  - inadequate flexural strength,
  - inadequate shear strength, or a
  - combination of the two;
- shear wall failure due to
  - inadequate reinforcement or
  - inadequate connection to the surrounding/adjacent frame members; and
- 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. It must be understood, however, that for masonry walls to maintain their in-plane load carrying ability, they must not collapse due to out-of-plane forces. To prevent out-of-plane collapse, adequate connection details must be provided (see Negro and Taylor, 1996). Anicic (1995) reports that horizontal ties, tie beams and/or columns and “rigid” floor diaphragms are normally effective at preventing out-of-plane collapses of masonry walls provided the span-height ratio and height-thickness ratio are kept within normal design limits.

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.

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).

Other more "high-tech" retrofit solutions include seismic isolation and/or the use of energy dissipation devices (friction-based, viscoelastic-based or hysteretic) which are most commonly incorporated into additional bracing. Experimental tests of such systems are widely reported in the literature (e.g. Griffith et al, 1990; Aiken et al, 1993).

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 trends are towards the use of advanced fibre composites, energy dissipation devices and seismic isolation schemes for the seismic retrofit of buildings, the more traditional methods should not be neglected when considering which system(s) to employ. The optimal scheme will depend upon various factors, many of which are non-technical such as aesthetics and the level of disruption to occupants (Jirsa, 1994).

Research in the area of seismic retrofit of historical monuments has been reported at the last four European Conferences on Earthquake Engineering (Syrmakezis et al, 1998, 1995, 1990; Syrmakezis, 1986). In these reports some of the special considerations when working with historical buildings and monuments have been highlighted.

A "state-of-the-art report" on the seismic performance of URM buildings in recent North American earthquakes by Bruneau (1994) highlights the seismic risk posed by URM in central and eastern North America. It was reported that the analytical procedure in the Uniform Code for Building Conservation requires modification.

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 that are reported in the literature for concrete frames, masonry infill and the like are explicitly noted wherever possible in this literature review.

A recent review by Badoux (1998b) of developments in seismic retrofitting research in the USA highlighted the use of:

- precast reinforced concrete panels for retrofitting frame structures;
- jacketing for reinforced concrete bridge and building columns with inadequate confinement or lap splice details; and
- composite overlays for retrofitting masonry buildings.

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. The use of jacketing of concrete columns to address shear capacity problems was also discussed.

Seismic retrofit practice in Japan has been reported periodically by Sugano (1980; 1981b; 1982), Sugano and Endo (1983), Endo et al (1984), Higashi et al (1984), and more recently by Kabayama et al (1998). Early methods commonly involved the addition of concrete shear walls and/or steel bracing. The paper by Endo et al (1984) summarises data collected by the Japanese Concrete Institute on 157 retrofitting projects in Japan. The seismic performance before and after retrofitting was also discussed. Higashi et al (1984) reported on tests of concrete frame models strengthened by post-cast shear walls, adding pre-cast concrete walls, adding steel bracing, adding steel frames and by casting the walls monolithically with the frame. Sugano (1980) gives an excellent review of the "then" existing practice of seismic retrofit for concrete buildings in Japan. He discusses in particular the use of infill walls, bracing, buttressing, wing walls and column strengthening. It is of interest to note that at that time, column strengthening consisted of encasement in steel jacketing, steel straps or additional concrete. The use of composite materials was some years away. Rogdriguez and Park (1991) reviewed the literature on the repair and strengthening of reinforced concrete buildings, with emphasis on reinforced concrete columns.

Finally, many case studies have appeared in the literature highlighting the world-wide activity in this area (e.g., Holmes, 1979; Knoll, 1983; Zezhen and Dinggen, 1988; Del Valle Calderon et al, 1989; ATC, 1992; Moehle et al, 1994; Rutherford and Chekene, 1994; Wasti et al, 1998). The various key issues relevant to seismic retrofit, in general, and concrete frames with masonry infill walls, in particular, are reviewed in the following sections.

2.2 Seismic Assessment

2.2.1 Overview

A long-term area of research activity has been the topic of seismic assessment. Nevertheless, recognition of the seismic risk posed by the many non-seismically designed non-ductile buildings in the central and eastern United States and many parts of Europe has led to continued research in this area.

The importance of proper seismic assessment as part of the seismic upgrading process can not be underestimated (Bertero, 1992). To that end, the most important issues to be addressed by engineers preparing repair and strengthening strategies for RC buildings were outlined by Pilakoutas and Dritsos (1992). Recent overviews on the current state of seismic assessment and repair in the US (Rojahn and Comartin, 1998), Japan (Kabayama et al (1998), and Europe (Fardis, 1998) were given at the 11th European Conference on Earthquake Engineering. Current techniques which have been or are being used in Japan include the use of additional

RC walls or steel plates or jacketing (RC or metal) of concrete columns. Some of the newer retrofit techniques include composite fibre jacketing, isolation and energy dissipation (mainly in the form of viscoelastically damped braces). In the overview by Fardis (1998), key issues were discussed with reference to deformation-based design rather than the traditional force-based design concepts. It was noted that the foundation might require strengthening if the super-structure is strengthened or the load-path altered.

Other papers on the subject of seismic assessment include that by Bonacci (1994) where the use of the substitute structure approach in calculating design forces for drift and damage control is described. Hassan and Sozen (1997) present a simplified method of ranking RC, low-rise, monolithic buildings according to their seismic vulnerability. This procedure requires only the dimensions of the building structure and is based on the damage that occurred during the 1992 Erzincan earthquake.

Park (1997) presented a "force-based" procedure for assessing the seismic resistance of concrete frame buildings, which is based on determining the probable strength and ductility for the collapse mechanism of the structure. The method accounts for beams, columns and beam-column joints with substandard reinforcement details. By determining the lateral strength and displacement ductility of the frame, a designer can estimate the likely seismic performance from acceleration response spectra for various ductility levels.

In contrast, Priestley (1997) presented a paper that discusses the use of a displacement/deformation-based procedure for the seismic assessment of concrete frame and shear wall buildings. The paper employs basic capacity design principles to develop a systems approach for assessment that results in a displacement-based determination of available seismic capacity. It appears that this approach, accounting for column and beam-column joint shear strength, gives less conservative estimates of performance than would result from the application of existing (force-based) code rules.

Other recent papers have covered such details as the use of the capacity spectrum method for comparing retrofitting strategies, seismic assessment as a low-cycle fatigue process, seismic assessment of ancient churches and other historical monuments, and performance evaluation of buildings during recent earthquakes (Sucuoglu and Erberik, 1997; Petrini et al, 1998; Olaru, 1998; Lagomarsino, 1998; Kratzig and Meskouris, 1998; Badoux, 1998a). Badoux (1998a), in particular, gives an excellent outline of some of the key issues designers must address when selecting a seismic retrofit scheme. Technical and non-technical factors are discussed and four basic retrofitting strategies are used to illustrate the use of the capacity spectrum method for comparing the effectiveness of the various strategies under consideration. The efficiency of the capacity spectrum method as a "displacement-based" analysis tool for seismic assessment was illustrated using the results of a study into the seismic resistance of existing RC frame plus bearing (shear) wall buildings in Switzerland (Peter and Badoux, 1998).

Finally, Dovich and Wight (1996) conducted tests on a 2-storey, 2-bay concrete flat slab frame and two connection subassemblies with reversing cyclic lateral loads. The results indicated that for reasonable gravity loads, the lateral response was more ductile than anticipated, giving hope that seismic retrofit of such structures might be less expensive than commonly assumed.

2.3 Reinforced Concrete Frames with Masonry Infill

2.3.1 Overview

There has been much work conducted into the seismic behaviour of infilled frame buildings (e.g., Abrams, 1996, 1994; Chrysostomou et al, 1992; Zarnic and Tomazevic, 1984). Some of the most recent work published has focussed on refining modelling techniques for such buildings, taking into account parameters such as wall openings, wall thickness, pier width, gaps between brickwork and frame, etc. (Schneider et al, 1998; Mosalam et al, 1998; Kappos et al, 1998; Griffith and Alaia, 1997). The results of this and previous work suggest that the brickwork can be accurately modelled using "equivalent struts". However, the most significant outcome is perhaps the general consensus that brickwork infill can have a beneficial effect on the overall seismic performance of the building if it is properly tied into the rest of the building. The notable exception to this is, of course, partial-height infill walls that often times cause columns to experience non-ductile shear failures rather than respond in a ductile, predominately flexural manner (Zarnic and Gostic, 1997). In such instances, a sufficient gap must be preserved between the infill and the column face to prevent interaction or the column must be detailed to prevent premature shear failure.

2.3.2 Seismic Behaviour of RC Frames with Masonry Infill

The complex nature of the interaction between concrete frames and masonry infill wall panels is reflected perhaps in the large number of experimental studies conducted on this topic. For example, Bertero and Brokken (1983) summarised work in which the effects of masonry and lightweight concrete infills on RC moment resisting frame buildings were studied experimentally and analytically. The experimental investigations consisted of quasi-static cyclic and monotonic load tests of 1/3-scale models of the lower 3.5 storeys of an 11 storey, 3-bay RC frame infilled in the outer two bays. The RC frame was designed for high rotational ductility and resistance to degradation under reversed cyclic shear loads. For reasons of economy, ease of construction, favourable mechanical properties and efficiency of different types of masonry infill, it was concluded that the most promising panel configuration consisted of solid brick laid in mortar reinforced with two mats of welded wire fabric, one bonded to each size of the wall in a layer of cement stucco (mortar).

Around the same time, Zarnic and Tomazevic (1984) summarised the results of their experimental and analytical investigations into the seismic behaviour of masonry infilled RC frames. It was found that the infill began to crack at approximately 0.2% lateral drift and the system exhibited satisfactory behaviour up to 2% drift. Small amounts of horizontal reinforcement of the infill were found to have little effect. Subsequently, experimental tests on seismic retrofit methods suitable for RC frames with masonry infill were performed (Zarnic et al, 1986a). The repair methods consisted of epoxy grouting of cracks in the concrete frame and masonry infill elements plus strengthening of the masonry infill by reinforced concrete jacketing of the infill panels.

Valiasis and Stylianidis (1989) conducted tests of a concrete frame with URM infill walls. The infill wall was not "connected" to the surrounding frame, as is often the case in existing buildings. The infill increased the building strength by 50%; however, this additional strength disappeared at comparatively small lateral drifts. In contrast, Carydis et al (1992) conducted shake-table tests of a steel frame filled with brickwork but where the gap between the brickwork and frame was filled with a nonshrinkable grout. It was reported that the grout-filled gap was largely responsible for the frame's good behaviour although test results were only reported for lateral drifts up to 0.14%.

In Portugal, Pires and Carvalho (1992) conducted experimental tests of seven 2/3-scale models of a 1-storey, 1-bay concrete frame. Six of the frames had brick masonry infill. One was tested as a bare frame. Quasi-static horizontal cyclic loading was used. Three of the infill wall frames were constructed such that the concrete frame was constructed after the brick infill was built and the other 3 had the infill constructed afterwards. During tests, a loss of stiffness due to masonry infill cracking was observed at about 0.1% drift where the maximum strength was recorded. The strength decreased approximately linearly from that point to about 50% of the maximum strength at a drift of 1%. An analytical model was subsequently developed and calibrated against the experimental data (see Figure 1) to represent the hysteretic behaviour (Pires et al, 1995).

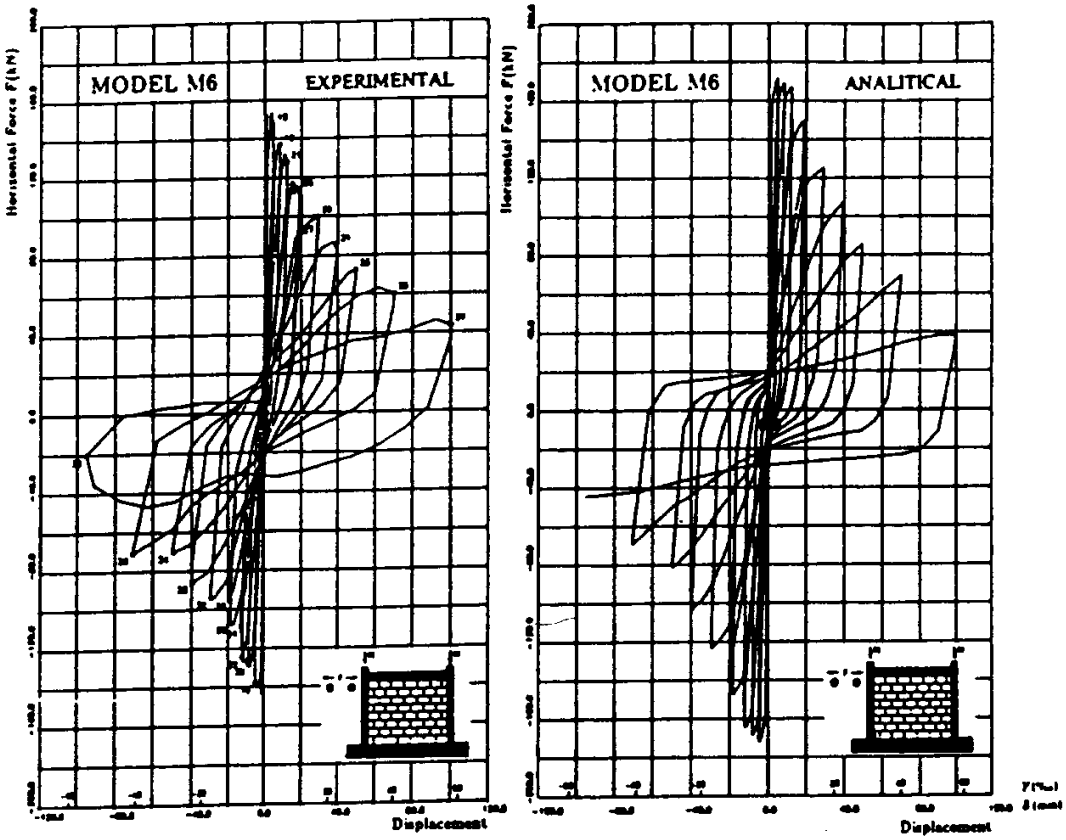

Figure 1 – Horizontal force-displacement test results.
(after Pires et al, 1995)

Valluvan et al (1994) conducted tests to study the shear transfer across frame-wall joints in RC frame buildings. The tests helped clarify (1) the benefits of compression across the joint, (2) the role of reinforcing bars as dowel shear connectors across the joint, (3) the influence of strength of concrete in the existing frame, and (4) the poor performance of grouted frame-wall joints.

Manos et al (1995) reported on the results of experimental and analytical investigations into the seismic behaviour of RC frames with masonry infill. Bare RC frames reached their maximum strength at drifts of 1%. The infill reached its maximum at 0.3% drift and was able to maintain it until about 2% drift. Based on this work, it was concluded that 3D panel elements could be used to model infill effects but that realistic material properties are required in order to obtain accurate models of prototype structures.

The ability of masonry infill walls to increase the strength of concrete frames was further highlighted in an experimental study of the seismic behaviour of a 1/3-scale concrete frame (1-bay, 4-storey) with infill walls by Liauw and Kwan (1992). Two models were tested. In the first model, the frame was infilled with reinforced concrete shear walls. Brick masonry was used for the infill in the 2nd model without any connectors to the surrounding frame. Both structures had the same static strength. This was even more dramatically demonstrated in a full-scale PSD test conducted on a 4-storey RC frame (Negro and Verzeletti, 1996). The frame was first tested as a "bare frame" with a nominal acceleration 50% larger than that used for design. The structure responded as a weak-beam strong-column mechanism. The fundamental frequency was half after the tests. The maximum base shear force was 140*kN* and the maximum drift was about 2%.

The frame was then tested with all bays in the frame infilled with unreinforced hollow block masonry. The same loading histories were used for all tests. The ground-storey infill was severely damaged by the 1.50 times design earthquake test. The maximum drift during this test was 1.1% (see Figure 2) and the maximum base shear was 2000*kN* (approximately 50% stronger than the bare frame).

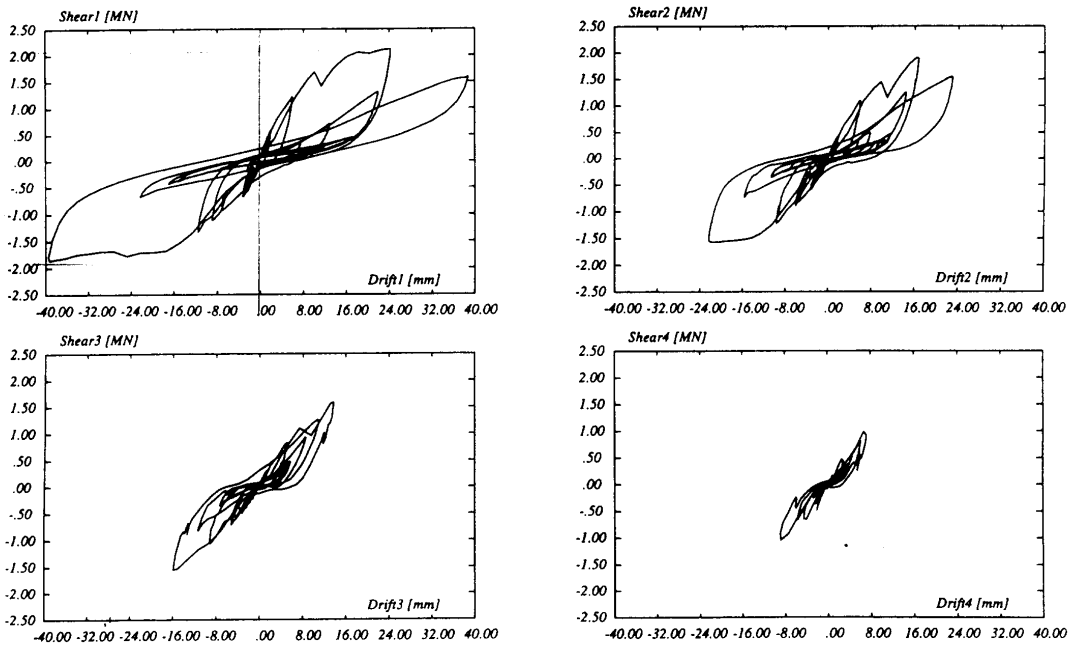

Figure 2 – Storey shear force versus storey drift test results.
(after Negro and Verzeletti, 1996)

The infill was then removed from the ground storey level to create a "soft" storey frame. The tests were repeated once again. The energy dissipation, frame deformation behaviour and strength/stiffness degradation characteristics were all discussed.

The results of tests conducted on 12 separate 1-bay, 1-storey RC frames with masonry infill were reported by Mehrabi et al (1996). The experimental results indicate that the infill can improve the seismic response. Specimens with strong frames and strong infill exhibited much better behaviour. The least ductile specimen performed satisfactorily up to 2% drift. For the bare frame, V_{max} occurred at 3.1% drift and still retained 80%V_{max} at 6.8% drift. In contrast, the infilled frames reached V_{max} at 0.5% drift and 0.8V_{max} at 1.5% drift.

Mosalam, White and Ayala (1998) used pseudo-dynamic testing to study the response of a 2-storey, 2-bay steel gravity load designed steel frame with unreinforced concrete block masonry walls. They showed that it could be successfully modelled analytically using "equivalent" struts with gap elements to model "lack of fit" between the frame and infill. Hence, the wall did not act until between $\pm 0.1\%$ and $\pm 0.2\%$ drift since the hysteretic storey shear versus drift loops all had bi-linear stiffening shapes. The infill walls were badly cracked by drifts of about 0.5% in the PSD tests.

Schneider et al (1998) conducted experiments to investigate the in-plane seismic behaviour of steel frames with URM brick infill panels with large window openings. The test results indicate: (1) infill strength and stiffness deterioration seemed to be independent of the pier width and wall thickness; (2) ultimate shear strength of the infills ranged between 0.82 and 1.08 *MPa* at drifts of between 0.75% and 1.5%; and (3) stiffness deterioration for each infill was almost identical (see Figure 3). By 0.2% drift each infill had lost more than 70% of its original effective stiffness and all gone by 2% drift. Finally, narrow piers tended to be more ductile than wide piers, resulting in less observable damage than wide piers. In contrast, two-brick thick piers tended to be more ductile than single-brick thick piers.

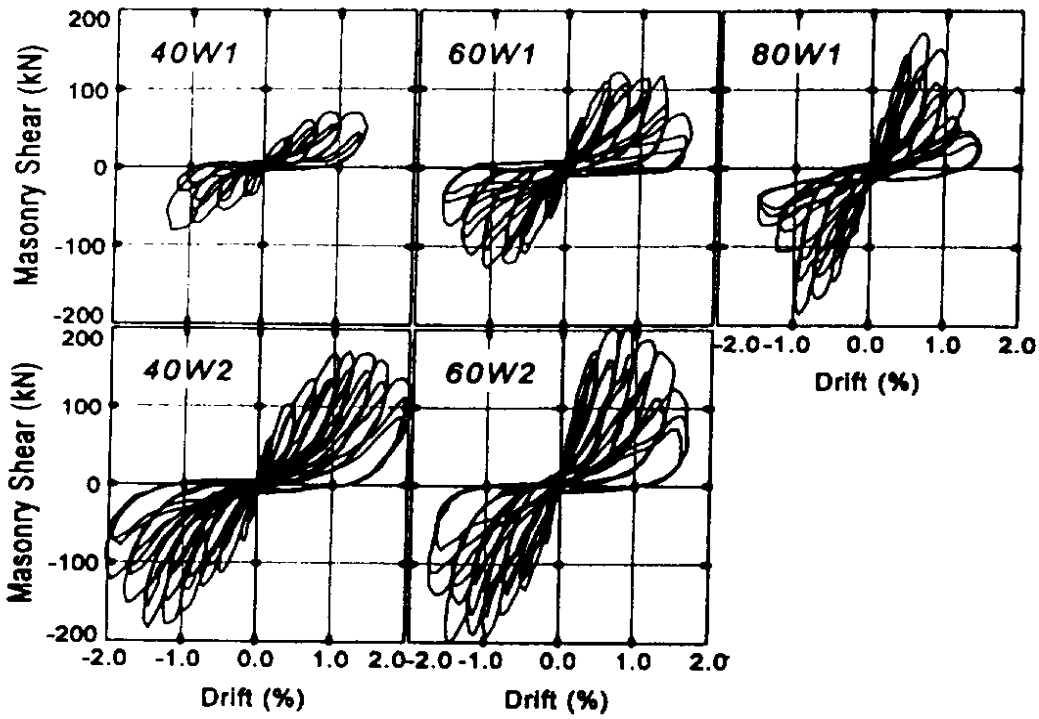

Figure 3 – Horizontal shear force versus lateral drift test results.
(after Schneider et al, 1998)

Govindan et al (1986) conducted tests on a 7-storey concrete frame with brick infill walls in order to study its strength, ductility and energy absorption characteristics as compared to those for the same frame without infill walls. It was observed that the infill frame strength was double that of the bare frame and that its maximum strength was reached at a lateral drift of 3.7%. The bare frame reached its maximum strength at a drift of 1%.

A large amount of effort has been put into the development of mathematical models capable of representing the seismic behaviour of concrete frames with masonry infill. The approaches use, for the most part, either a finite element approach or an "equivalent diagonal strut"

approach to represent masonry infill walls. For example, Shing et al (1992) and Combescure (1996) have developed finite element models based on both a smeared crack and discrete crack approach to evaluate the shear resistance of masonry walls and masonry infilled RC frames. While such models were shown to be capable of reproducing experimental results with a high degree of accuracy, it was observed that the numerical results were highly dependent on the finite element idealisation. Furthermore, the inelastic behaviour of an infilled frame is very sensitive to the shear strength of the mortar.

Valiasis et al (1993) presented a hysteresis model for analysis of weak URM infilled RC frames where there is no positive connection between the infill and surrounding frame elements. Analytical results are compared to prior experimental results where the URM reached its ultimate strength and cracked at a drift of 0.2 to 0.3%.

Zarnic (1995) developed a mathematical model, based on results of experiments on 34 1-bay, 1-storey frame plus infill models, which is suitable for dynamic analysis of infilled frame structures. The masonry infill is modelled as pairs of compressive longitudinal springs and the model has been incorporated into the DRAIN-2D computer program. Formulae for stiffness of masonry panels reach their ultimate strength at 0.3% drift and maintain this strength level until 2% drift at which point the strength goes to zero.

Michailidis et al (1995) developed an analytical model for masonry infilled RC frames subject to seismic loads. The model is capable of representing strength and stiffness degradation, hysteretic pinching and slippage (see also Kappos et al, 1998). Similar work has been conducted by Via et al (1995) using “equivalent strut” models and Papadopoulos and Karayiannis (1995) who developed a simple method for non-linear dynamic analysis of RC frames with structural and infill walls.

The benefits of using masonry infill as a structural element to improve the seismic behaviour of building frames were summarised by Zarnic and Gostic (1997). They noted, however, that incorrect use could actually cause unexpected damage. An “equivalent strut” model was developed, based on extensive experimental testing, in order to investigate some key parameters for design. In this model, the ultimate strength of the frame was reached at 1% drift and the ultimate strength of the masonry infill at 0.2% drift. Tests conducted at the ELSA reaction wall testing facility showed that the infill strength degraded to 60% of its maximum at about 1% drift.

Comparative analyses of some RC frame buildings with URM brick infill panels using first diagonal strut elements and then isoparametric shear-only plate elements to represent the infill panel effects was performed by Kappos et al (1998). Both techniques were seen to give almost identical results and agreed quite well with results of prior experiments by Valiasis & Stylianiais (1989) in Greece. In those experiments the infill was observed to fail at about 0.3% drift but without any significant loss of strength until about 3% drift where the strength was about 80% of the maximum. The analytical results clearly indicated the benefit of the infill on the seismic behaviour of the building.

Finally, Fardis and Calvi (1995) presented an overview of the 1st year of progress of the “Infilled Frames” part of the PREC8 project. This research focussed on masonry infilled concrete frame structures and included experiments on full-scale infilled frames. Shaking table tests were used for out-of-plane behaviour investigation and pseudo-dynamic testing for in-plane investigation. Modelling and numerical simulations for parametric analyses were

also carried out. They suggested that the ultimate shear strength of infill could be estimated using the formula $V_m = 1 - 1.3$ times $A_m f_{c,diagonal}$ where A_m is the horizontal area of the infill and $f_{c,diagonal}$ is the cracking strength of the masonry in diagonal compression. The stiffness can be estimated using the formula $k_m = \frac{G_m A_m}{h_m}$ where G_m is the shear modulus for the masonry obtained from diagonal compression tests and h_m is the height of the masonry infill. The maximum infill strength is assumed to occur at a drift of approximately 10% of the maximum drift for the bare frame (see Figure 4).

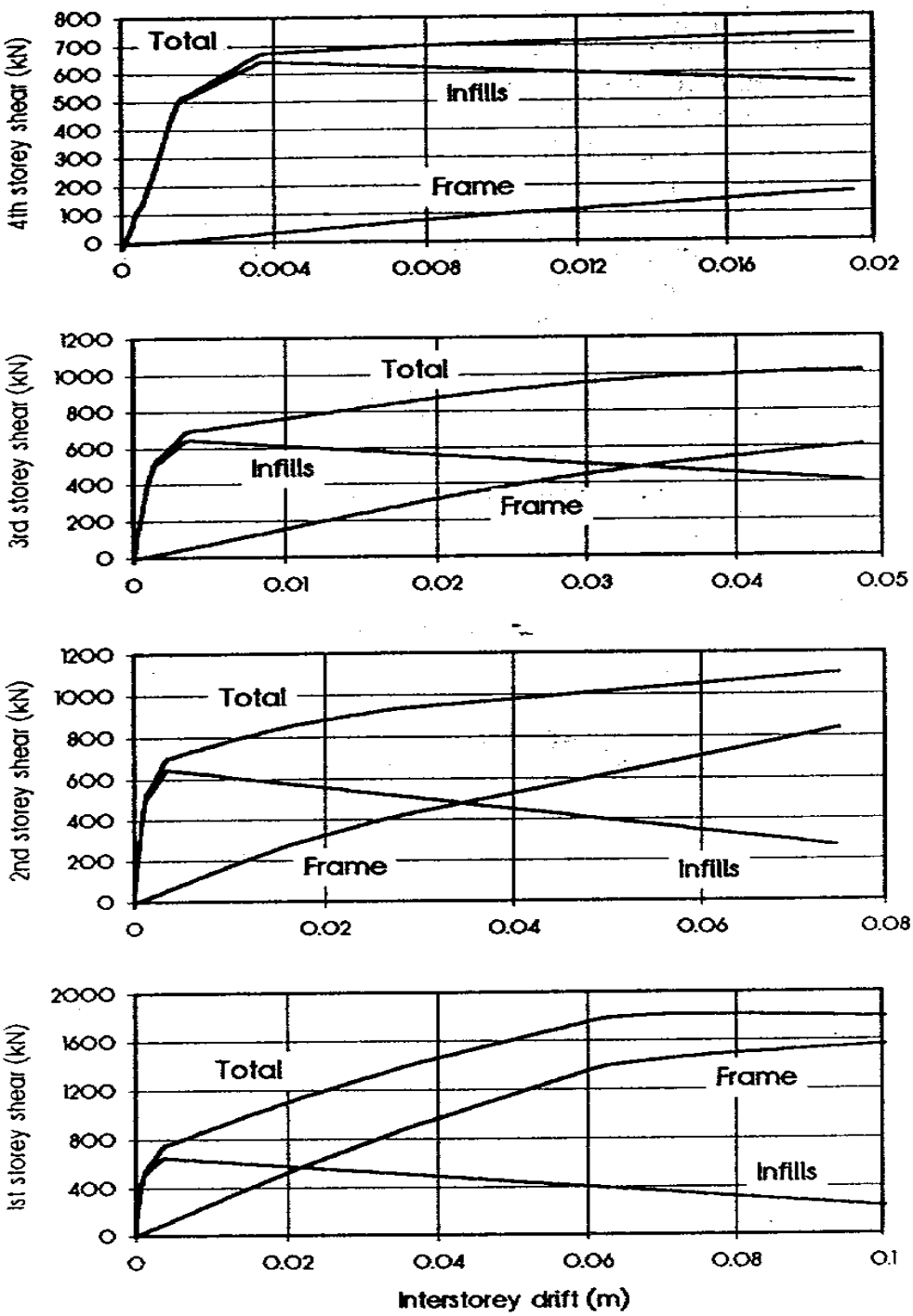

Figure 4 – Interstorey shear versus drift relations for infilled frame.
(after Fardis and Calvi, 1995).

As mentioned at the outset of this section, there have been many experimental investigations into the seismic behaviour of masonry infilled frames. The results of just some of these studies have been discussed here. In order to highlight the information most pertinent to the present study from a "deformation" perspective, these results are summarised in Table 1 below where the lateral storey drifts for first cracking of URM infill or where the URM, infilled frame or bare frame reached their respective maximum strengths.

Table 1 – Summary of infilled frame test results.

Reference	δ_{cr} (%)	$\delta_{\max}$ (%)			Other Notes
	URM	URM	Frame + Infill	Bare Frame	
Mosalam et al (1998)	0.3	<0.8			
Negro & Verzeletti (1996)	<0.3		1.1	2.4	$V_{\max} = 0.4W$ for bare frame, $V_{\max} = 0.62W$ for infilled frame
Fardis and Calvi (1995)					$\delta_{\max}$ for URM is 0.1 $\delta_{\max}$ for frame
Schneider et al (1998)	0.1	1			$k < 0.3k_i$ by 0.2% drift
Kappos et al (1998)	0.07	0.2-0.4	0.4	0.7	Comparative drift values for structure subject to same input. $\tau_u \approx 0.35MPa$
Valiasis & Stylianidis (1989)			0.3		$V = 0.8V_{\max}$ at 3% drift
Zarnic (1995)	0.1	0.3	0.6	2	
Manos et al (1995)	0.15		0.3	1	$\tau_u \approx 0.3MPa$ $V = 0.8V_{\max}$ at 2% drift for infill frame
Michailidis et al (1995)	0.1	0.25-0.35			$\tau_{cr} \approx 0.25MPa$, $\tau_u \approx 0.32MPa$
Pires & Carvalho (1992)			0.5		$\tau_u \approx 0.27 - 0.51MPa$
Pires et al (1995)			0.3	2	$V = 0.8V_{\max}$ at 6% drift
Zarnic & Tomazevic (1984)	0.2		1	3	
Zarnic & Gostic (1997)		0.2	1	>1	
Valiasis & Stylianidis (1989)			0.6	1	$\tau_u \approx 0.25 - 0.3MPa$
Mehrabi et al (1996)	0.3		0.6	3.1	$\tau_u \approx 0.5MPa$, $V = 0.8V_{\max}$ at 1.5% drift for infilled frame, 6.8% for bare frame
Shing et al (1992)					$\tau_u \approx 0.34MPa$
Carydis et al (1992)					Good system behaviour up to 0.14% drift; steel frame with infill
Govindan et al (1986)			3	1.5	
Zarnic (1998)			0.3		$V = V_{\max}$ for $0.3\% < \delta < 2\%$ used in mathematical model of URM infill

1. δ_{cr} = is the lateral storey drift at which the masonry infill cracks.
2. $\delta_{\max}$ = the lateral storey drift at which the maximum force $V_{\max}$ is attained for the URM infill, infilled frame or bare frame, respectively.
3. W = weight of structure.
4. k = lateral in-plane stiffness of the URM, k_i = the initial stiffness.
5. τ_u = maximum shear strength of the URM, τ_{cr} = cracking strength of the URM.

2.3.3 Out-of-plane strength of URM Infill

The out-of-plane strength of unreinforced masonry is well-recognised as being one of the major weaknesses of URM buildings with regard to seismic actions. However, as noted in Section 2.1, masonry infill walls must not fail in the out-of-plane direction if they are to maintain their in-plane load capacity. To that end, Abrams et al (1996) proposed some simple procedures for estimating the out-of-plane strength of URM infill panels. The procedures are based on experimental results. Results of PSD tests on a full-scale 4 storey RC building have already been reported in Section 2.3.1 (Negro and Verzeletti, 1996). However, in companion shake-table tests of a scale-model of the same 4-storey RC building, it was observed that that out-of-plane loading was not a problem for the concrete frame with infill (even without mechanical connections) if the gap between the infill and frame was less than 10*mm* (Negro and Taylor, 1996).

2.3.4 Frame plus Infill Retrofit Case Studies

A large number of seismic retrofit projects have been documented and provide an excellent database for practising engineers. The retrofit methods used naturally reflect the state-of-the art at the time. A selection of case studies encompassing a wide range of retrofit options is presented in this section.

Johnson and Smietana (1990) conducted a case study of a phased seismic retrofit program. The existing structure had masonry walls and concrete frames with and without masonry infill. Scott and Deneff (1993) presented the results of a case study into the seismic retrofit of a 5-storey non-ductile concrete flat slab building with URM infill located in western Kentucky. Performance criteria for this project required the building and its contents to be operable during and immediately following a major New Madrid fault zone earthquake. The seismic retrofit of a building of similar construction, the Monte Cristo Hotel in Everett Washington, was reported by Lundeen and Perbix (1994).

Murray and Parker (1994) reported on the seismic retrofit of a group of low-rise non-ductile concrete frame buildings with URM infill walls in western Tennessee. Interestingly, a post-earthquake operability criterion was one of the design requirements for the retrofit scheme. The addition of infill walls or steel bracing were considered by Pincheira and Jirsa (1994) as part of a comparative analytical study of different seismic retrofit schemes for a non-ductile RC frame building. In contrast, Miller and Gould (1996) presented a comparative study of the use of base isolation and the addition of shear walls for the seismic retrofit of a RC frame with masonry infill walls.

Jokerst (1995) presented a case study of the seismic retrofit of four PG&E office buildings in downtown San Francisco. The 1989 Loma Prieta earthquake caused significant disruption and some damage to all four of these buildings. Two of the buildings were 17-storey steel frame buildings with concrete slabs and URM infill bearing on a timber pile foundation system. The other two buildings were 14-storey and 7-storey steel frames with concrete slabs and reinforced concrete infill walls.

2.4 Beam-Column Joints and Connections

2.4.1 Overview

Reconnaissance reports from recent earthquakes indicate that connection failure is a common failure mode (Park et al, 1995; Holmes and Somers, 1996; Benuska, 1990). In unreinforced masonry buildings, this typically occurs between gable-end walls and the roof or between the

wall and floor system. In both instances, the wall collapses often leading to local or total collapse of the building (Bruneau, 1994). In reinforced concrete buildings, connection failure usually occurs in the region of beam-column joints or column splice joints. For example, in many older concrete frame buildings (El-Attar et al, 1997) the bottom beam steel is not adequately anchored into the joint (or continued through the joint) so that the bottom beam steel can not develop its full tensile capacity under reversed loading. Similarly, columns in concrete frame buildings often contain splice joints immediately above the floor slab level. In older buildings, the splice lengths are often inadequate for the column steel to reach its full tensile capacity. Thus, under seismic loading, the columns may not be able to achieve their full moment capacity due to the inadequate splice joint let alone balance the full moment capacity of the beams connected to the joint. Consequently, many older concrete buildings collapse as a result of joint failure or as a soft-storey collapse mechanism.

The performance of older concrete frames with brick infill walls in recent earthquakes suggests that the most likely failure modes are brick infill wall failure followed immediately by column shear failure or beam-column joint failure. Significantly better behaviour has been observed in frames where the brickwork completely fills the bay. Many collapses and near-collapses of concrete frames with partial-height brick infill have been observed in relatively recent earthquakes (Park et al, 1995; Holmes and Somers, 1996). Consequently, there has been a significant amount of research activity in the broad area of connections that is of particular relevance to this project.

2.4.2 Dowel shear/tension connections

In the late 1980s and early 1990s, a number of researchers studied the seismic behaviour of "dowel-type" connections which could be used to connect "post-cast" and "pre-cast" concrete and masonry infill walls to surrounding concrete frame members. For example, Bass et al (1989) tested 33 full-scale push-off specimens in "direct shear" mode to study the interface shear capacities between new concrete cast against an existing concrete surface. Test variables were surface preparation, amount and anchorage depth of interface reinforcement, reinforcement details, compressive strength of new and existing concrete. Reversing cyclic loading was used. Peak strength, degradation of strength with repeated load cycles and increasing deflections were monitored. Eligehausen & Vintzeleou (1989) conducted tests of metallic anchors under both monotonic and cyclic shear loading to study (1) the effect of cracks which crossed the anchor, (2) the distance of the anchor from the free edges of the section, (3) the presence of transverse reinforcement and (4) the loading history. These anchors are typical of those used to connect infill shear panels to concrete frames. Jirsa (1989) conducted tests on reinforcing bars and bolts, epoxy grouted into concrete elements. The results were relevant for designers using epoxy connectors in seismic retrofit work. Shimizu (1989) conducted shear tests on 48 expansion anchors and pull-out tests on 407 specimens. Improved expansion anchors are described where no thread comes to the surface. Ductility is compared to that previously reported for other connections.

In 1989, Wyllie presented a paper that gave design guidelines for the use of epoxy-grouted dowels to transmit tension or shear from new concrete members to existing concrete. Several years later, Hosokawa (1992) reported on the performance of post-installed anchors during experimental tests at the University of Tokyo. It was observed during tests that: (1) the reliability of the metal expansion anchors in their pull-out stiffness and strength was improved by applying preloading during installation; (2) cleaning of the installation holes improved strength and stiffness; and (3) the installation technique substantially increased the lateral resistance of frames retrofitted with a post-cast concrete shear wall.

McVay et al (1996) presented the results of experimental and analytical investigations into the behaviour of chemically bonded anchors for use in seismic retrofit work. They concluded that a simplistic, uniform bond stress applied over the whole anchor did an excellent job of predicting pullout capacity. Further work in this area was conducted by Nielsen, et al (1998a,b). They reported on the experimental tests of a flexible connection for the seismic retrofit of precast panels used as cladding. Monotonic and cyclic tests were conducted in both in-plane and out-of-plane directions (i.e., shear and tension pullout tests). Results indicate that the use of epoxy cements may be limited due to their thermal sensitivity and that Portland cement grouts were preferred.

2.4.3 Beam-column joint tests

Experiments at the University of Texas at Austin (Leon & Jirsa, 1986; Bastos et al, 1986) involved the testing of 14 full-scale biaxially loaded RC beam-column joints to study the effects of: load history, beam reinforcement size, beam geometry, and floor slabs on joint behaviour under cyclic bi-directional reversible loading. From these tests it was observed that internal beam-column joints reached their ultimate strength at drifts of about 3.75%. By a drift of 5% the strength had dropped to about 65% of their maximum. Exterior joints performed similarly. Furthermore, Bastos et al compared these results to results from full-scale building tests in Japan and small-scale joint tests performed at Stanford. They found that the small-scale test results gave much "fatter" hysteresis loops than did either the Texas or full-scale Japanese tests.

Owada, (1992) also reported on experimental cyclic tests of seven 1/5-scale beam-column joints. In these tests, it was observed that varying axial load adversely affected joint behaviour and joints with transverse beams behaved better than those without transverse beams did (e.g., exterior joints). The maximum joint shear stresses were found to range between $1.33\sqrt{f_c'}$ and $1.70\sqrt{f_c'}$. At the time, research was being conducted at Cornell (Beres et al, 1992a) into the seismic behaviour of gravity designed concrete frames in parallel with experiments on over 30 full-scale interior and exterior beam-column joint regions. The joint details typically included few ties in the joint region, discontinuous bottom beam steel extending only about 150*mm* into the column, column lap splice joints immediately above each joint and columns with low amounts of longitudinal steel and very few lateral ties. The joints were subject to quasi-static cyclic loading. These results were later confirmed in experiments by Bracci et al (1995a). In essence, lightly reinforced concrete frames are highly flexible and may suffer large "P-delta" effects during moderate earthquake loading. Even though non-seismic detailing was not found to be critical in the failure process of the 1/3-scale model test structure, the floor slab contributions to the beam capacity lead to the frame exhibiting a soft storey column side-sway mechanism. The peak shear stresses were between $\sqrt{f_c'}$ and $1.2\sqrt{f_c'}$, noticeably smaller values than those recorded by Owada (1992). Exterior joint failures occurred at lateral storey drifts of between 1.5% and 2%. Interior joints failed at drifts of between 2% and 2.5%.

The results of investigations into the seismic behaviour of reinforced concrete frames designed principally for gravity loads (Bracci et al, 1995a,b) have highlighted that such frames often are dominated by "weak-column strong-beam" behaviour and that strong ground motions are likely to cause seismic strength demands in excess of capacity. Even where the seismic demand is less than capacity, excessive lateral drift due to the weak-column character of the frame will tend to cause severe damage to non-structural elements and other secondary systems. Hence, these "weak-column" structures appear to have a smaller margin of safety

against collapse in the event of strong ground motions than comparable "strong-column weak-beam" frames and should be considered for retrofit. In tests they observed that a gravity-load designed concrete frame responded elastically with a maximum base shear reaction of 0.065W (W = weight of structure) and peak lateral drift of 0.5% when subjected to a 0.05g (small magnitude) earthquake motion. For 0.2g earthquake input, the base shear reaction was 0.15W and the storey drift was 1%. For 0.3g earthquake input, the response was highly inelastic with a base shear of 0.153W and a drift of nearly 2%. In essence, the maximum base shear of $V_{\max} = 0.15W$ was reached at about 1% drift and maintained to approximately 2% drift.

Special RC beam-column connections were tested under simulated earthquake loading by Raffaelle et al (1992). The connections included wide beam-column connections and eccentric beam-column connections. The experimental program investigated the effect of varying the beam width and depth and the amount of longitudinal reinforcement. From the results, an equation for estimating the effective joint width of eccentric connections was proposed. Good joint behaviour was observed in these tests up to 4% drift.

A summary of these test results is given below in Table 2.

Table 2 - Summary of column and beam-column joint test results.

Reference	$\delta_{\max}$ (%)	Comments
Leon and Jirsa (1986) Bastos et al (1986)	3.75	$V = 0.65V_{\max}$ at 5% drift. Beam-column joint tests (interior and exterior joints essentially the same)
Owada (1992)		Maximum joint shear stress $\tau_{\max} \approx 1.3 \text{ to } 1.7\sqrt{f_c'}$
Bracci et al (1995a,b)	≈ 2	Maximum joint shear stress $\tau_{\max} \approx 1 \text{ to } 1.2\sqrt{f_c'}$ $V_{\max} = 0.065W$ at 0.5% global drift for $0.05g$ input MRF: $V_{\max} = 0.15W$ at 1% global drift for $0.20g$ input $V_{\max} = 0.153W$ at 2% global drift for $0.3g$ input
Lynn et al (1996)	0.6	$V = 0.6V_{\max}$ at 1.5% drift. Test results of pre-1970s columns. Premature shear failure of columns (M only $\approx 0.7M_u$)
Rodriguez and Park (1994)	1.5	$V < 0.75V_{\max}$ at 2 – 2.5% drift (pre-1970 columns).
Priestley et al (1994a,b)	1	$V \approx 0.6V_{\max}$ at 1.5 – 2% drift (column tests).
Aboutaha et al (1996a)	1	$V \approx 0.5V_{\max}$ at 1.5% drift (non-ductile column tests).
Gomes and Appleton (1996)	1	$V \approx 0.8V_{\max}$ at 5% drift (column tests).
Tanaka et al (1985)	1.1	$V \approx 0.8V_{\max}$ at 2% drift (column tests). Stirrups with 90° bends opened at $\approx 2 - 2.5\%$ drift.
Bett et al (1988)	≈ 1	$V \approx 0.7V_{\max}$ at 1.5 – 2% drift (column tests).
Chai et al (1994)	≈ 1	$V \approx 0.5V_{\max}$ at 2% drift (column tests).
Saadatmanesh et al (1997)	≈ 2	$V \approx 0.5V_{\max}$ at 3.5% drift (column tests).
Beres et al (1992)	2 (int.) 1 (ext.)	$V \approx 0.8V_{\max}$ at 3% drift for internal beam-column joints. $V \approx 0.8V_{\max}$ at 2% drift for external beam-column joints.
Raffaelle et al (1992)	3	$V \approx 0.8V_{\max}$ at 5% drift for beam-column joints.
Alcocer (1992)	2	$V \approx 0.8V_{\max}$ at 4% drift for beam-column joints

Note: $\delta_{\max}$ = the lateral storey drift at which the maximum column or beam-column force $V_{\max}$ is attained.

2.4.4 Strengthened beam-column joints

Repair and strengthening techniques for beam-column joints are discussed in this section. The methods vary from simple repair of joints to their "original" condition to major strengthening techniques designed to increase the joint's strength and ductility. At the "repair" end of the spectrum, Chung & Lui (1978) conducted dynamic shear tests on beam-column joints that had been severely damaged and then repaired by epoxy grout injection. Results were encouraging, showing that the repaired joint strength was similar to the strength of the "virgin" test specimen.

In experiments by Chronopoulos et al (1995), 2 large scale beam-column RC joints were tested with cyclic reversing loads up to displacement ductility indexes of 2, 4 and 6. The joint designs were typical of "older" concrete frame construction (few stirrups in the joint) and "newer" construction (with substantial joint reinforcement). Each joint was tested to the 3 levels of displacement ductility and then repaired by epoxy injection of cracks and replacement of cover concrete and welding of reinforcing steel where broken. The repaired joints were then re-tested and found to be half as strong as originally (for the "new" joint detail) and equally as strong for the "old" joint detail. However, the repaired joints tended to suffer more rapid strength degradation than the original joint specimens and were seen to be only good for displacement ductilities of approximately 2. The original strength of the "new" joint detail exhibited clearly superior behaviour to the "old" joint detail.

The behaviour of concrete frame slab-beam-column connections was investigated experimentally by Alcocer (1992). Joints both before and after seismic retrofit intervention were studied. The retrofit consisted of reinforced concrete jacketing the columns or both the columns and the beams adjacent to the joint region. Only interior joint regions were investigated. The rehabilitated specimens were designed such that the flexural strength of the retrofit columns was greater than the flexural strength of the beams. The joint regions not confined by the adjacent columns and beams were strengthened with a cage of steel angles and flat steel bars. Slab participation was monitored during the tests and compared to code design guidelines. From these tests, it was concluded that jacketing of frame elements can improved strength, stiffness and energy dissipation characteristics as well as change the collapse/failure mechanism of a frame to that dominated by strong-column weak-beam behaviour. Slab participation was estimated to be approximately 20% of the transverse span for the damaged (pre-tested) specimens and half the transverse spans for undamaged specimens. All rehabilitated specimens reached their maximum strength at lateral drifts of approximately 2% and maintained this strength up to 4% lateral drift.

A method of jacketing RC beam-column joints with corrugated steel sheeting was presented by Ghobarah et al (1996a,b). This form of jacketing was shown to not suffer from the outward bulging problems of steel jacketing with flat steel sheets reported by Priestley (1994a). The corrugations act to stiffen the jacket in the needed "hoop", or tie, direction and so maintain good confinement of the concrete. Four concrete test specimens were tested, one which represented existing structures, one represented current seismic design standards and two rehabilitated connections. Results indicated good cyclic performance at high load levels and significant increases in shear capacity ($V_u = 400kN \rightarrow 450kN$) and displacement ductility ($\mu_\Delta = 2 \rightarrow 4$).

2.5 Column Strengthening

2.5.1 Overview

The state-of-the-art in strengthening of concrete columns appears to be by "jacketing" – i.e., the use of reinforced concrete, steel plates or fibre (carbon or glass) reinforced plastic (FRP) sheets wrapped around the column. This method using steel or FRP is particularly well suited to addressing inadequate column splice joint details since steel or FRP column jackets are efficient in tension and can provide an alternative load path for the column tension forces in the splice joint region. The method is also well suited to correcting problems of inadequate column confinement due to column ligatures which are spaced too far apart to prevent, on their own, the buckling of the longitudinal column steel once the concrete cover spalls off. Fundamental research was first conducted on beams under monotonically increasing static loading (e.g., Saadatmanesh and Ehsani, 1991; Plevris et al, 1995; Ritchie et al, 1991). The benefit of confinement on the stress-strain behaviour of concrete was highlighted previously by Mander et al (1988) so that the application of jacketing to concrete columns, in retrospect, is obvious. A number of recent papers give experimental data on the effectiveness of this technique (Gomes and Appleton, 1998, 1996) and design equations (Monti and Nistico, 1998). Of course, concrete columns can also be strengthened with concrete jackets. Bush et al (1990) and Monti and Nistico (1998) give comparative analyses of various retrofit strategies that incorporate some form of concrete jacketing for columns and beam-column joints. Rodriguez and Park (1991) provide an excellent review of the literature on the repair and strengthening of RC buildings with particular emphasis on columns.

2.5.2 Column Behaviour

The seismic behaviour of concrete columns has been studied for some time. For example, Tanaka et al (1985) tested four square RC columns under axial compressive load and cyclic flexure to simulate severe seismic loading. The main variable in this study was the type of anchorage used for the hoops and cross ties. Around this time, Mander et al (1988) presented the results of experiments that attempted to quantify the improved stress-strain characteristics due to concrete confinement. The results highlight the benefits of lateral reinforcement (confinement) and hence the benefits of jacketing. These numerical models are now in many non-linear dynamic analysis packages.

Recently, Lynn et al (1996) reported on their experimental and analytical study into the seismic evaluation of existing RC building columns. The lateral and vertical load-resisting behaviour of concrete columns typical of pre-1970s construction (90° hooks in stirrups) was considered. Eight full-scale columns were tested with constant axial load and increasing cyclic lateral displacements until failure. Test data was compared to results from a variety of evaluation methods. Results indicate that splice development lengths of $20d_b$ (where d_b is the diameter of the longitudinal reinforcement) are adequate to develop yield strain in longitudinal reinforcement. However, with repeated yielding cycles the column rapidly loses strength. The ultimate strength of the specimens was reached at about 0.6% drift. By 1.5% drift, the specimen strengths had dropped to less than 60% of their maximum. Shear failure occurred in most specimens at loads of about 70% of that needed to create M_u in the columns.

2.5.3 Grout Injection

Grout injection is still commonly used for the repair of concrete elements. It has been reported elsewhere that it typically can be relied upon to return concrete members to their original strength. Along these lines, Tasai (1992) conducted a study into the use of epoxy

resin grout for the repair of bond failure in RC members. Tests were conducted on short and long column specimens to illustrate the use of (1) epoxy mortar or pre-packed concrete mixed resin to repair regions of spalled cover concrete and (2) low viscous epoxy resin to inject into bond splitting cracks. In contrast to the results reported earlier (Chung and Lui, 1978; Chronopoulos et al, 1995), the performance of the repaired specimens was actually better than that of the original specimens. For smooth, round bars, the specimens were slightly stronger after grouting than they were originally. Grouting doubled the strength of specimens with deformed bars over their original strength.

2.5.4 Concrete Jacketing

Concrete jacketing was probably the first of the jacketing methods to be employed in practice. Early efforts at this were reported by Hayashi et al (1980) where they describe an experimental investigation of two strengthening methods for RC buildings. These were (1) adding new shear walls inside the frame and (2) strengthening existing columns with welded wire fabric and mortar. More recent work in this area was conducted by Bett et al (1988). They tested three concrete columns to study the effect of column strengthening and jacketing. The basic unstrengthened column was 12”x12” (300x300*mm*) with 8 No.6 bars and 6*mm* ties at 8” (200*mm*) spacing. This specimen was tested to 2% drift and then repaired by removing all loose cover, adding 4 No.3 corner bars and 4 No.6 mid-face bars, 6*mm* ties at 2.5” (63.5*mm*), epoxy bonded No.3 cross-ties at 9” (229*mm*) and a 2.5” (63.5*mm*) shotcrete shell. The second specimen had the same basic concrete core but was jacketed with 4 No.3 corner bars, 6*mm* ties at 2.5” (63.5*mm*) and a 2.5” (63.5*mm*) shotcrete shell. The third specimen was strengthened and jacketed in the same way as the first specimen. Specimens 2 and 3 were in “as-new” condition when strengthened and jacketed. All three specimens were tested to 2.5% drift in their jacketed condition. Results indicate that columns strengthened by jacketing, both with and without supplementary cross-ties, were much stiffer and stronger (about 90 kips (400*kN*)) laterally than the original, unstrengthened column (about 45 kips (200*kN*)). The column repaired by jacketing was also much stiffer and stronger (about 80 kips (355*kN*)) laterally than the original column and performed almost as well as the strengthened columns.

Work at the University of Texas at Austin was reported by Bush et al (1990) who conducted tests on a RC frame with deep spandrel beams (weak-column structure) which was retrofit by cast-in-place piers (i.e., concrete encased columns) to change the mechanism to that of a strong-column weak-beam. Epoxy-grouted dowels were inserted into the original columns to make the jacket work integrally with the original column. The original frame base shear strength was 30 kips (130*kN*) and the retrofit frame strength was 300 kips (1300*kN*). Also at Austin, Alcocer and Jirsa (1990) tested four full-scale interior concrete frame beam-column joints that were repaired with varying amounts of jacketing. Two joints had only columns steel-jacketed. The other two specimens had both columns and beams jacketed. Bi-directional cyclic loading was used in the tests.

Reinforced concrete jacketing was widely used in Mexico City after the 1985 earthquake. Teran and Ruiz (1992) reviewed the seismic retrofit practices used in Mexico City. In their paper, they give quantitative detailing recommendations as well as general guidelines and emphasise the need for good connections between the original and new structural material. They note that jacketing only mildly effects strength and stiffness but substantially increases ductility (see also Bett et al, 1988).

The use of vertical column post-tensioned concrete jackets was studied by Reinhorn et al (1993). They reported on the experimental tests of a retrofitted 1/3-scale RC frame that had

been tested previously without any strengthening. Some joint strengthening was also carried out on the damaged structure before testing.

In New Zealand, Rodriguez and Park (1994) performed tests on RC columns strengthened by RC jacketing. Four columns were tested. The results showed the effectiveness of the technique but the authors noted that the method was labour intensive. The unretrofitted column reached its maximum strength at 1% drift but lost most of it by 2% drift. The retrofitted column strengths were typically 3-4 times greater and performed well to 3% drift.

A variety of concrete jacketing techniques were investigated by Bracci et al (1995a,b). In this work they studied the use of (1) pre-stressed concrete jacketing, (2) masonry block jackets, and (3) masonry pier infill to retrofit RC frame buildings designed primarily for gravity loads. The pre-stressed concrete jacketing method was selected from the three options as being the most suitable, based on preliminary analyses. Its effectiveness was evaluated experimentally with shaking table tests of a 3-bay, 3-storey RC frame at SUNY at Buffalo. Results indicated that the retrofit scheme was successful in changing the yield mechanism for the building from a "weak-column strong-beam" to a "strong-column weak-beam" mechanism. The original frame had an ultimate base shear strength of 0.15*W* and acceptable behaviour up to 2% storey drift although exhibiting a weak-column sidesway mechanism. The sum of the column moment capacities at the first storey was only 75% of the sum of the girder moment capacities. After retrofit, the frame had a base shear strength of 0.30*W*, ultimate storey drift of 1% and column to girder moment capacity ratio for all levels of between 1.5 and 2.

Around the same time, Ciampoli (1995) in Italy was conducting an extensive series of comparative analyses of a RC bridge girder to study the effectiveness of seismic upgrading using either concrete jacketing or isolation/energy dissipators. In Taiwan, meanwhile, Sheu and Chang (1995) had proposed hysteresis rules for short RC columns based on the analysis and testing of 16 different columns. Three groups of columns were tested. Group 1 was a collection of conventional reinforced concrete columns. Group 2 columns were strengthened with wire mesh and additional concrete. Group 3 columns were strengthened with steel hoop plates and concrete. The strength of group 2 and 3 columns was equal to or slightly (20%) greater than the group 1 columns. Displacement drifts for the 2nd and 3rd group of columns, however, had increased to 8% from between 3% and 6%.

Quite recently, Bracci et al (1997) presented a static pushover analysis procedure for evaluating the seismic performance and retrofit options for low-to-medium rise RC buildings. The technique is based on the capacity spectrum method and was illustrated by application to the 1/3-scale 3-storey RC frame model that had been previously tested on the shaking table at Buffalo. Three retrofit examples were considered. These were (1) prestressed concrete jacketing of internal columns, (2) RC fillets around beam-column joints, and (3) post-tensioning of additional column longitudinal reinforcement. Retrofit increased the frame's base shear strength by 66% (from 0.15*W* to 0.25*W*) and the maximum drift from 1% to 2%.

Finally, in Portugal Gomes and Appleton (1998, 1996) tested 9 RC columns with reversing cyclic loading in order to assess the efficiency of column jacketing. Concrete jacketing was used in this project which effectively increased the column dimensions from 200x200*mm* to 260x260*mm*. Additional longitudinal steel was added to keep the reinforcement ratio at approximately 1%. Jacketing increased the strength by 250% and greatly improved the hysteretic behaviour.

2.5.5 Steel Jacketing

Steel jacketing seems to have first appeared widely in the literature in the early 1990s, possibly as a consequence of the 1989 Loma Prieta earthquake that caused a number of highly publicised collapses of concrete bridge and elevated freeway structures. Along these lines, Ersoy et al (1993) conducted two series of tests to study the seismic behaviour of jacketed columns. The first test series consisted of uniaxially loaded specimens. The second series consisted of combined axial and flexural loading. At the same time, Valluvan et al (1993) was studying the behaviour of retrofitted column splice joints. In this work, twelve 2/3-scale specimens with columns splice details were strengthened either by (1) welding the splice bars, (2) confining the splice region with steel angles and straps, or (3) providing internal or external steel reinforcing ties in the splice region.

In San Diego, Priestley et al (1994a,b) conducted a comprehensive analytical and experimental study of the shear behaviour of reinforced concrete bridge columns designed before 1971 (90° stirrups). They were interested, in particular, in the effectiveness of full-height steel jackets for enhancing the seismic shear strength of concrete columns. The first paper presents new equations for predicting the enhanced shear strength due to steel jackets for circular and rectangular columns. The second paper presents the results of cyclic tests of 8 circular columns and 6 rectangular columns. Half of the columns were tested in the "as-built" condition and the other half were retrofitted with full-height steel jackets. All of the "as-built" columns failed in shear in a brittle fashion or at low flexural ductility levels. The jacketed columns performed extremely well, with stable hysteresis loops being achieved to displacement ductility levels of 8. The retrofit columns increased the drift ratio from those observed for the "as-built" columns from approximately 1% to 4 – 5%. Similarly, the jacketed columns had stiffness and strength increases over the "as-built" columns of approximately 30-60% and 50%, respectively. Based on this work, Chai et al (1994) developed an analytical model for predicting the first-yield Limit State and the ultimate Limit State of flexurally dominated steel-jacketed circular bridge columns. The model for the ultimate Limit State was governed by low-cycle fatigue fracture of the longitudinal steel that was assessed using an energy-based damage model. The model was checked against earlier experimental results where the steel jacketing was seen to improve strength by approximately 50% and the ultimate lateral drift ratio (from 1% to nearly 5%).

Refinements in the technique were investigated by Sanders et al (1995). They tested two ½-scale beam-column joint specimens to investigate the benefit of shifting the hinge down into the column and away from the bent cap in bridge structures. In order to do this, the portion of the column above the new desired hinge location was reinforced with steel jackets to prevent failure. A gap was provided in the steel jacket to prevent the hinge capacity from being too large. The retrofit column achieved a displacement ductility of 6 with the maximum base shear occurring at a drift of nearly 1%. Another variation was reported by Frangou and Pilakoutas (1995). They developed a method of externally confining concrete members which involved post-tensioning metal strips by using conventional strapping equipment used in the packaging industry. Beam tests were used to illustrate a 25% increase in strength and a 50% increase in displacement ductility.

At the same time, a variation of the above technique (Frangou and Pilakoutas, 1995) was developed by Georgopoulos and Dritsos (1995). They reported their results for tests on concrete columns which had been "jacketed" by pre-tensioned steel cages. The steel cages essentially consisted of steel angles running vertically at each corner of the rectangular column and held together by pre-tensioned horizontal steel (hoop) tie sheets. The tie sheets

were pre-stressed either by special wrenches or by pre-heating prior to welding. The level of pre-tension and the spacing of the tie sheets were two factors that most influenced the results, although the amount of pre-tension was seen to be much more significant than the spacing.

The application of steel jackets to concrete frame buildings was investigated by Aboutaha et al (1996a). In their work, they tested non-ductile concrete frame columns retrofit using rectangular steel jackets. Eleven columns were tested. Solid steel jackets with and without anchor bolts were used. Test results indicate that a thin rectangular jacket with adhesive anchor bolts can be highly effective for concrete columns with inadequate lap splice joints. Unretrofitted columns reached their ultimate strength at about 1% drift and were at less than 50% of their ultimate strength by 1.5% drift. On the other hand, the retrofit columns were 1 to 1.5 times stronger and maintained strength greater than 80% of their maximum up to 4% drift.

Elsewhere, Ersoy (1996) gave an overview of the current state of seismic assessment and rehabilitation. Experimental research at the Middle East Technical University in Ankara, Turkey on jacketed columns and infilled frames was also summarised. In Texas, Aboutaha and Jirsa (1996b) conducted an experimental study on the use of rectangular steel jackets for seismic strengthening of RC columns. Four large-scale columns with inadequate lap splices and four large-scale columns with inadequate shear strength were tested. The basic unretrofitted columns exhibited non-ductile cyclic behaviour. Retrofitted columns exhibited ductile response, higher strength, and improved ductility and energy dissipation.

Parallel developments into the use of composite materials for jacketing of concrete columns were well underway by this time. Monti and Nistico (1998) conducted a parametric study of the effect of steel and FRP jacketing on concrete bridge piers. The work resulted in some design equations for circular bridge piers.

2.5.6 Composite Jacketing

The use of fibre composite materials in seismic retrofit applications has grown steadily in the 1990s. Early experiments considered static loading of beams strengthened with composite sheets epoxy bonded onto their tension face. Examples of such research include that by Saadatmanesh and Ehsani (1991) who conducted 4-point static bending tests of RC beams strengthened with epoxy bonded glass fibre reinforced plastic (GFRP) plates.

Ritchie et al (1991) conducted tests on 16 under-reinforced concrete beams to study the effect of flexural strengthening by epoxy bonding FRP plates onto the tension face. Static, monotonic 4-point loading was used. Results showed an increase in stiffness over the working load range from 17% to 99% and increases in strength (ultimate) from 40% to 97%. Ultimate failure of the beams normally did not occur in the region of maximum moment but rather by local shear failure (tension/shear peeling) at the end of the FRP plates. Analytical models were developed by An et al (1991) to predict stresses and deformations in concrete beams strengthened with fibre composite plates epoxy-bonded to their tension faces. Triantafillou et al (1992) conducted experimental and analytical studies of the use of pre-stressed FRP sheets to strengthen concrete members. In this project, beams had pre-stressed FRP sheets bonded onto their tension zones. The beams were tested with monotonically increasing static 3-point bending tests. While all these tests only involved monotonically increasing static loading of beams, the results encouraged research into seismic applications.

The use of carbon fibre reinforced plastics (CFRP) in structural applications was investigated by Plevris et al (1995). They performed a study of the reliability of statically loaded RC beams strengthened with CFRP plates epoxy-bonded onto the tension face. The authors derived a value for the general strength reduction factor of 0.80.

More recently, studies have been conducted to investigate the feasibility of FRP sheets to increase the shear strength of concrete beams and columns. For example, Triantafillou (1998) tested eleven concrete beams strengthened in shear with carbon FRP fabric. Various area fractions and fibre configurations were tested. The effectiveness of the technique was seen to increase almost linearly with the FRP axial rigidity to a maximum beyond which there was not further enhancement. In these tests, the shear strength was seen to increase by between 65% and 95% although the failure mechanism of CFRP sheet debonding was still brittle.

Norris (1997) tested nineteen beams that had been retrofitted with CFRP sheets. The sheets were epoxy bonded to the tension face and web of the concrete beams to enhance their flexural and shear strength. The effect of the CRFP sheets on the strength and stiffness of the beams were studied. Only monotonic static 4-point loading beam tests were conducted. Substantial increases in strength were observed. For beams where the fibre orientation was perpendicular to the cracks, the strength increases were the largest but the failure mode was quite brittle with little ductility. When the fibres were oriented obliquely to the cracks in the beam, smaller increases in strength and stiffness were observed, however, the mode of failure was more ductile and preceded by warning signs such as snapping sounds or peeling of the CFRP. The results of this work indicate that it may be possible to retrofit concrete structures with CFRP sheets and still achieve ductile overload behaviour.

GangaRao and Vijay (1998) tested 24 concrete beams with static, monotonic 4-point loading. The beams had carbon fabric wrapped around them and were studied only from a flexural point of view. As above, significant increases in strength (57% - 100%) were observed. The maximum carbon fibre strain was in the range of 1% to 1.5%. Beams that were tested without carbon fabric wraps and then retrofit with the carbon material exhibited similar performance to the “undamaged” wrapped beams. The ultimate strength of the wrapped beams was successfully predicted using conventional concrete beam theory by properly accounting for the tension forces in the carbon layers.

The behaviour of FRP jacketed concrete (bridge and building) columns has also been studied. For example, Yamamoto (1992) conducted an experimental study to develop a strengthening method for RC columns using FRP. Two kinds of tests were performed. Uniaxial tests showed that the uniaxial strength of the columns increased in proportion to the FRP strengthening ratio. Shear-flexure tests were then used to study the effect that FRP had on combined shear and flexural strength. Ductility was increased by about 250% while strength and stiffness remained about the same. From this data, design equations for FRP jacketed columns subject to combined axial, shear and flexure were developed.

Extensive testing of FRP jacketed columns has also been conducted at San Diego. Based on this experience, Seible et al (1995) describes (1) jacket design aspects, (2) mechanical performance of carbon jacket retrofits, (3) jacket installation and (4) full-scale field tests. The program used filament winding of prepreg carbon fibre tows around columns to form a carbon jacket. Xiao et al (1995) reported on tests of three large-scale bridge column tests which had been retrofit using a prefabricated composite wrapping system. Finally, Seible (1995)

provides an excellent overview of the use of advanced composites in the seismic retrofit of concrete structures.

Saadatmanesh et al (1996) tested 5 different bridge columns. Two columns served as control specimens, one having a splice joint and the other having continuous longitudinal reinforcement. The other 3 specimens had FRP straps wrapped around the hinge regions. One of the jacketed columns had grout pressure injected to create an initial prestress in the straps. The other two jackets were ungrouted. It was seen that whereas the control specimens were only able to reach displacement ductilities of about 3 (4.5% drift) for the spliced control specimen and 4 (6% drift) for the continuous control specimen, the jacketed specimens exhibited very stable hysteretic behaviour at displacement ductilities in excess of 6 (9% drift). Increases in strength of over 30% were also apparent. The use of continuous reinforcement gave better performance than the spliced bridge column. There was little apparent difference between the performance of the grouted and ungrouted-jacketed columns.

Saadatmanesh et al (1997) also tested 4 building columns to failure under reversed inelastic cyclic loading and then repaired the columns with prefabricated FRP wraps and retested them. The columns were repaired only in the critically stressed regions near the column footing joint. The results showed that the unretrofit columns reached their ultimate strengths of approximately 13*kN* at about 1.7% drift which quickly dropped off by more than 30% at a drift of 3%. In contrast, the retrofitted columns exhibited good hysteretic behaviour and did not reach their ultimate strength of about 16*kN* until a drift of about 4% which was essentially maintained for drifts up to about 6.5%.

2.6 Bracing

2.6.1 Overview

At the recent European Conference on Earthquake Engineering, there were a number of papers which discussed the use of bracing for strengthening concrete frames (e.g., Badaloukas et al, 1998; Shahin et al, 1998) and masonry walls (Taghdi et al, 1998). Papers along these lines have also appeared in technical journals (Galano and Gusella, 1998; Badoux and Jirsa, 1990). A number of papers have also appeared which discuss the merits of additional damping being incorporated into the bracing system (Rai and Wallace, 1998; Tena-Colunga and Vergara, 1997, 1996c; Dorka et al, 1998; Cahis et al, 1998). The key issues in the use of bracing appear to be that both the stiffness and strength requirements of the structure must be addressed. Implicit in this is the need for the force-deformation behaviour of the bracing system to be in harmony with the structure being retrofit. Hence, while bracing may increase both strength and stiffness, its post-yield behaviour may not be compatible with the building's load-deformation characteristics. Damping devices may be incorporated into the bracing system, however, these normally require sizeable structural deformations in order to work efficiently. Hence, the most common of these (friction-based and viscoelastic-based devices) may not be compatible with the small deformations that cause cracking in unreinforced brick masonry infill walls. Of course, if one is prepared to let the masonry infill fail, then there is no "compatibility" problem. However, for situations where this is not desirable, the use of a yielding "shear-link" device that is capable of yielding at relatively small strains may be suitable (Rai and Wallace, 1998).

2.6.2 Steel Bracing

The seismic retrofit of the 8-storey RC frame building on the Tohoku Institute of Technology in Sendai that was damaged in the 1978 Miyaki-ken-oki earthquake is discussed by

Kawamata and Ohnuma (1980). The building was retrofit in the longitudinal direction using eccentric steel cross bracing that was installed in both facades from the exterior of the building. Experimental results of the cross-bracing behaviour and the strength of the frame-to-brace connections are presented which illustrate the system's effectiveness.

Many analytical studies have been conducted into the use of steel bracing as a seismic retrofit technique for frame buildings. For example, Badoux and Jirsa (1990) studied the use of steel bracing for the seismic retrofit of RC frame structures. Frames with short, weak columns were studied in particular. It was found that bracing can be used to specifically target strength and stiffness deficiencies but inelastic buckling of braces should be avoided. A combination of bracing plus beam weakening was found to significantly improve the inelastic frame behaviour. Nevertheless, even with bracing and beam weakening, the retrofit frame structure was limited to less than 1% lateral drift. In this application, the frame stiffness was approximately the same as the bracing stiffness, the strength and stiffness were doubled and the system ductility was improved from 2 to 5. Other examples of similar work include that by Yamamoto and Umemura (1992), Tagawa et al (1992), Nateghi (1995), Badaloukas et al (1998), and Shahin et al (1998).

An experimental study at Michigan by Masri and Goel (1996) reported on the use of ductile steel bracing for strengthening seismically weak RC slab-column buildings. Concentric bracing plus steel cage jacketing of columns was tested experimentally. 2.75% drift was achieved experimentally in the 1st storey (and 2% drift overall) for the retrofit frame compared with less than 2% for the bare frame. The braced-frame strength was nearly four times greater than that of the bare frame. This aspect is one that may render this retrofit technique unsuitable if the structural foundations require substantial strengthening as a consequence of over-strengthening of the super-structure.

Also at Michigan, Rai and Goel (1996) conducted tests on URM wall piers that were strengthened with steel elements. The steel elements consisted of horizontal and vertical steel straps (bolted to the brickwork) and diagonal steel bracing connected to the surrounding frame only. The strengthened piers were stronger than originally and began to rock at a lateral drift of 0.6%. A similar approach was investigated in Canada by Taghdi et al (1998) who reported the results of an experimental investigation of the effect of seismic retrofit of non-ductile concrete and masonry walls with steel-strip bracing. The bracing consisted of flat steel strips attached to the walls with through bolting. Results indicate that the strength increased by 300%, the ductility went from 0 to between 2 and 3, and the URM failed at 0.5% drift while the retrofit wall failed at a drift of 1%.

Finally, Galano and Gusella (1998) proposed a design criterion for seismic retrofit of masonry walls by steel cross bracing. The effectiveness of the technique was investigated analytically. A design parameter is given to achieve an optimal interaction between the URM wall and the bracing. This depends upon the relative stiffnesses of the wall and bracing. They also presented a formula for calculating the wall stiffness that is similar in form to the formula suggested by Fardis and Calvi (1995). Best results are obtained when the wall and bracing stiffnesses are similar.

2.6.3 Post-tensioned Steel Bracing

The use of post-tensioned steel bracing for seismic retrofit of frame structures is an important specialised form of steel bracing. Pincheira and Jirsa (1992) analysed 2 RC frame buildings to illustrate the benefits of post-tensioned steel bracing for seismic retrofit. An unretrofit 3-

storey structure was calculated to reach its maximum lateral strength at a drift of 0.75%. The retrofit version of the same structure was seven times stronger at a drift of 1%. The 12-storey original structure reached its maximum lateral strength at a drift of 0.5% whereas its retrofit version was 3 times stronger at 0.75% drift. Hence, post-tensioned systems were shown to be more effective for low-rise buildings on both firm and soft soils. The technique was seen to only be suitable for medium-rise buildings on firm soil. Similarly, Pincheira and Jirsa (1995) analysed a 3-storey, 7-storey and 12-storey RC frames which were retrofit (in turn) with (1) post-tensioned bracing, (2) steel bracing and (3) RC infill walls. In all cases, the retrofit buildings were much stronger than the original buildings (4 to 10 times stronger). However, the maximum lateral strength of all buildings was reached at a drift of approximately 1%.

Other examples of work in this area include that by Tena-Colunga (1996a) who performed an analytical study to compare the relative effectiveness of seismic retrofit of Mexican RC frame school buildings with full and partial-height brick infill walls. The two retrofit schemes considered were (1) post-tensioned steel bracing and (2) base isolation with lead-rubber bearings. The post-tensioned steel bracing consisted of high-slenderness steel strands, which were taken to be tension only, and stressed up to between 20% and 40% of yield. This system was used to retrofit a number of school buildings after the 1985 Michoacan earthquake and proved to be economical. The buildings were calculated to be between two and four times stronger after being retrofit. Teran-Gilmore et al (1996) reached similar conclusions in their study into the effectiveness of post-tensioned steel bracing for the seismic retrofit of RC frame buildings with URM infill walls. The retrofit strength was calculated to be 4 times that of the original building. Design issues and further research needs were also discussed.

2.6.4 Bracing plus Damping

Most of the research into seismic retrofit with steel bracing has involved the use of additional damping devices in order to minimise the increase in strength that the bracing would otherwise impart to the structure. Keeping in mind the adverse consequences of greatly increased base shear reaction forces, it is easy to see why so much research has gone into this area. Essentially, there are 3 categories of damping into which devices are generally classed, depending upon their mechanism for dissipating energy. These are either (1) friction-based, (2) viscoelastic-based or (3) hysteretic-based devices.

The uptake by the profession of this technology has been rather pleasing. For example, Martinez-Romero (1993) describes the retrofit of 3 different buildings in Mexico City using damping devices. The size and number of the devices are a function of the dynamic characteristics of the specific structure, the amount of previous damage, the anticipated earthquake motion and the design performance level intended. The techniques have been confirmed sufficiently by experimental and analytical work that preliminary design guidelines are now under development (Bozzo et al, 1996). A quick overview of some of the work that has gone into the development of typical damping devices is given below.

<u>Friction</u>

Aiken et al (1993) presented an overview of their experience of tests conducted at Berkeley on seven different passive energy dissipation devices tested between 1986 and 1991. Four of the systems were friction systems of which three are based on Coulomb friction (Sumitomo, Pall and Friction-Slip) while the fourth is Fluor-Daniel Energy Dissipating Restraint that provides self-centring friction resistance that is proportional to displacement. The three other systems all have different energy dissipating mechanisms. ADAS elements utilise the yielding of mild-steel X-plates. Viscoelastic shear dampers using a 3M acrylic copolymer as

the dissipative element and Nickel-Titanium alloy shape-memory devices that take advantage of reversible, stress-induced phase changes in the alloy to dissipate energy. The effectiveness of the various systems was evaluated by comparing the response of test structures with and without each of the energy dissipators.

ViscoElastic

As mentioned above, several viscoelastic damping devices were tested in the late 1980s at Berkeley. Other examples of work in this area is that by Zhang et al (1989) who conducted a feasibility study of the use of VE damped bracing to mitigate seismic response of steel frame structures. A procedure whereby the damping effect is incorporated into the modal damping ratios is proposed. Computer simulation indicates that the system is capable of significant reduction in floor displacements. Oh et al (1992) summarise an experimental and analytical study on the application of viscoelastic dampers as energy dissipation devices that can be incorporated into structural bracing. Experiments were conducted by Chang et al (1992) who tested a 2/5-scale steel frame with added VE damped bracing to study the seismic performance of the frame. Lateral drifts during these tests were recorded in the range of 0.2% to 0.5%.

Hysteretic

A large amount of work has gone into the development of hysteretic damping devices that can be incorporated into steel bracing. Examples of this work include that by Whittaker et al (1991) who conducted extensive experiments to establish the seismic performance of steel plate *added damping and stiffness* (ADAS) elements. The devices are typically incorporated into steel bracing and were shown to have stable hysteresis for a large number of yielding cycles. The devices were shown to be suitable for the upgrade of moment resisting frames and concentrically braced frames to achieve a moderately stiff building with extremely good energy dissipation characteristics. For example, the ADAS elements dissipated about 74% of the total input energy during the earthquake simulator tests. The test frame with the devices installed was also 66% stronger and experienced drifts that were one-quarter of those recorded for the frame without the ADAS elements. A number of buildings in Mexico City have subsequently been retrofit using these devices (Tena-Colunga, 1996a; Tena-Colunga and Vergara, 1996c).

Tena-Colunga et al (1996b) analysed the use of various seismic retrofit schemes for an existing 9-storey RC frame building. The different retrofit schemes considered were (a) weight reduction; (b) column and waffle slab jacketing; (c) addition of energy dissipation devices; (d) removal of top floors; (e) replacement of diagonal bracing with newer bracing; and (f) various combinations of the above. It was noted that the masonry infill behaviour was adequate at drifts up to approximately 0.5%. Further, Tena-Colunga and Vergara (1996, 1997) conducted a comparative study of an existing retrofit for a mid-rise steel building using additional stiff steel bracing against an alternate retrofit using ADAS (additional damping and stiffness) devices. The results suggest that retrofit using the ADAS devices would have yielded a better dynamic performance. However, the steel bracing retrofit was used since it provided more strength and its initial cost was less.

The recent development of specialised hysteretic damping devices that are capable of dissipating sizeable amounts of energy at comparatively small inter-storey drifts is also in progress. For example, Dorka et al (1998) conducted a parametric analysis to determine the optimum force level for a proprietary hysteretic device for use in retrofit of large panel buildings. The device is used to dissipate energy using shear links with hysteretic devices.

The devices can be either friction or yield devices with large stiffness and low yield displacement so that they will give full elasto-plastic hysteresis under small deformations. This gives a fairly accurate limit on the maximum force that can be transmitted and so protects the rest of the structure. Because the devices can be designed to work with relatively small deformations, they can be effective in protecting structures with panels of limited deformation capacity.

Work on devices with similar characteristics has also been reported by Cahis et al (1998). They presented the results of experimental and analytical studies of various energy dissipation devices suitable for earthquake protection. The devices were developed for the protection of masonry infill walls. The devices proposed by the authors are placed between infill and the frame to protect the infill. The devices have yield strengths less than the cracking strength of the infill. The devices are "shear" links between wall and frame and do not require large deformations to work.

Rai and Wallace (1998) report on the use of aluminium shear links in frame bracing. The shear links can be designed for stiffness, strength and deformation. Analytical comparisons are made for a steel moment resisting frame with (1) ordinary concentric bracing and (2) concentric bracing with the new shear link. Results indicate that the shear link bracing gives more uniform storey drifts, reduced the base shear and has larger energy dissipation capacity per unit drift than the conventional bracing.

2.7 Masonry Strengthening

2.7.1 Overview

With the imminent introduction of EC8 and the recognition of the benefit of masonry walls in concrete frames there has been a large amount of research into the seismic behaviour of masonry walls and various strengthening techniques. At least 9 papers were published on this topic at the recent European Conference on Earthquake Engineering (Yuksel et al, 1998; Wasti et al, 1998; Sofronie and Popa, 1998; Pires et al, 1998a; Juhasova et al, 1998; Irimies et al, 1998; Ehsani et al, 1998; Carydis et al, 1998; Braga et al, 1998). In summary, two basic techniques for retrofitting masonry walls were discussed: (1) better tying of the walls into the surrounding frame and/or (2) wall jacketing. The first of these techniques can be accomplished by the use of dowel connections, "keying" the brickwork into the concrete column, "tie columns" or the like. The second technique can involve application of steel or wire mesh and concrete, fibre-reinforced mortar/render which is troweled onto the brickwork, polymer grids/sheets bonded onto one or both sides of the brick walls, sandwiching the brickwork between additional concrete. Most of the jacketing techniques were considered both with and without bolts through the wall thickness to study the effect of deformation compatibility between the brickwork and the outer strengthening material. The effectiveness of many of these techniques have also been tested experimentally (Benedetti et al, 1998; Tomazevic and Klemenc, 1998; Juhasova et al, 1998; Pires et al, 1998a,b).

2.7.2 Reinforcement for Strengthening

Plecnik et al (1986) performed experiments aimed at developing a method of seismic strengthening for URM buildings whereby cored vertical holes in brick walls are filled with steel reinforcement and then filled with grout. The experiments studied various grouts and filler materials, size of core diameter and flow and strength characteristics of the filler

materials. Of particular interest to this project was the finding that the unstrengthened masonry reached its ultimate strength of $\tau_u \approx 0.2MPa$ at about 0.1% lateral drift.

Many other examples of reinforcing URM walls and buildings can be found in the literature. The majority of work has been focussed on improving the flexural strength of URM walls since that is its major weakness with regard to seismic loading. Examples of research into these areas are that by Mullins and O'Connor (1987), Carydis et al (1998), and Braga et al (1998). In the paper by Braga et al (1998) it was noted that retrofit interventions which are reversible are preferred over non-reversible interventions, especially for structures of historical significance.

2.7.3 Wall-to-Floor/Roof Connections

Another area that has received much attention is the seismic behaviour of wall connections in URM buildings. While most of this work has been focussed on improving the out-of-plane behaviour of URM walls, there has also been some research into the in-plane behaviour of walls with various degrees of connectivity between infill walls and the surrounding frame. For example, Tomazevic and Klemenc (1997) reported the results of experiments and analyses of confined masonry walls. Tie-columns were used to help confine the masonry walls. Buccino and Vitiello (1995) tested 3 common types of anchorage between steel bars and masonry walls. Out-of-plane cyclic loading was used to simulate seismic effects. Their experimental results were found to agree reasonably well with the current design rules. It was observed that: (1) strengths depended very much on anchorage type and quality of mortar; (b) strengths were reasonably high even at high levels of damage in the wall; (3) collapse was associated with large displacement; and (4) simple theoretical models gave a good estimation of strength.

While not of direct interest to this project, it should be noted that parallel studies of the out-of-plane behaviour of URM infill walls have also been conducted by Tomazevic et al (1995). For example, they performed a series of shaking table tests on models of historic and brick-masonry houses to study the effect of wall ties on the seismic behaviour. It was found that by tying the walls, the out-of-plane collapse of the walls of houses with wooden floors was prevented. Prestressing of the ties further improved the behaviour. More recently, Benedetti et al (1998) performed shaking table tests of 24 half-scale 2-storey masonry buildings to study the seismic behaviour and effectiveness of various retrofit techniques for masonry buildings. Some of the retrofit methods were: (1) cracking repaired by sealing and/or grout, (2) slab-wall connections improved with steel mesh and concrete topping and/or rendered steel mesh around building at each floor level and/or horizontal tendons (prestressed) at each storey level, (3) steel arches to reinforce door and window openings. Another example of work in this area is that by Antonucci and Giacchetti (1992) who presented the results of experiments on wall-to-floor connections typically used to connect masonry walls to floor slabs. These consisted of reinforcing bars inserted into the wall on one side and the floor slab on the other. The ability of the connection to adequately restrain the wall in the out-of-plane direction was investigated.

2.7.4 Jacketing and/or Grout Injection of URM Walls

Of particular interest to this project are the results of research into the behaviour of URM walls that have been repaired by grout injection or strengthened by some form of jacketing. An overview of Russian work in this area is given by Klyachko (1995). The methods covered include vertical post-tensioning of walls, RC wall jacketing, adding tie columns and use of tuned-mass dampers. Another excellent overview of this topic is provided by Tomazevic et al

(1993a,b) who discuss the strengthening of masonry buildings. In the first paper, test results are presented which illustrate that stone-masonry walls can be strengthened using "masonry-friendly" grouts and that the results are essentially independent of grout strength. The second paper presents test results showing that adequate diaphragm action can be obtained with wooden floor systems provided that the walls are "pre-stressed" to the floor system.

Shake table tests were performed on a brick URM monument building before and after retrofit (Juhasova et al, 1998). The retrofit scheme consisted of applying a fibre-cement plaster to all surfaces, adding steel arches over door and window openings, adding steel grid reinforcement and plaster to all ground floor walls and adding bolt connections between the first floor wood beams and walls. Ehsani et al (1998) tested seven ½-scale brick masonry walls to study the effectiveness of fibre composites for improving the out-of-plane behaviour of URM brick walls. Reversing cyclic tests, using air bags, were performed. Wall span drift in excess of 5% was measured. The flexural strength and ductility were significantly enhanced. The specimens were capable of supporting lateral pressures in excess of 30 times their own weight. The failure mode was controlled by delamination of the composite strips.

Of perhaps more relevance to this project is the improvement of the in-plane behaviour of grouted and/or jacketed URM walls. To that end, Maldonado and Olivencia (1992) reported the results of an experimental study into seismic retrofit techniques for masonry walls. Two of the methods investigated were simple grout injection of cracking for concrete and masonry elements and concrete jacketing of masonry walls. Only in-plane tests were conducted. The URM reached its ultimate strength at drifts of between 0.2% and 0.3%. The simple grout injection method was seen to return the URM walls to close to original condition. This result is consistent with that of Manzouri et al (1996). Based on extensive experimental testing, they reported that grout injection can fully restore the strength and stiffness of damaged URM walls and noted that the URM wall ultimate strength was reached at 0.3% drift.

Moghaddam and Mahmoodi (1995) also conducted tests of concrete frames with brick infills which were strengthened by (1) replacement of the brickwork in the corners with reinforced concrete and (2) adding a 25*mm* thick concrete render to the brickwork.

Schwegler (1995) performed tests on brick masonry shear walls strengthened with epoxy bonded composite fibre sheets. Increases in strength of over 30% were obtained. Failure of the strengthened specimens occurred at in-plane displacements of about 6*mm* (0.3% drift) for the polyester fabric material, in excess of 15*mm* (0.75% drift) for the carbon fibre sheets, and about 3*mm* (0.15% drift) for the unstrengthened masonry wall. In addition, significant "yield plateau" like behaviour was noted for the specimen retrofit with the carbon fibre sheets. Similar work was performed by Ehsani and Saadatmanesh (1996) who also presented some laboratory and field test results for buildings repaired with the technique following the 1994 Northridge earthquake.

In Lisbon recently, Pires et al (1998a) conducted tests on a bare RC frame and two brick masonry infilled RC frames. In the first phase of testing, the models were subjected to horizontal cyclic actions that caused severe damage to the infill and also some damage to the RC frames. In the second phase of testing, the models were repaired and subjected again to the horizontal actions. Different repair techniques were used for the two infill frames. Both infilled frames had mortar-repaired cracks and 25*mm* thick render applied to all external surfaces. The second infill frame, however, also had polymer grid reinforcement in the 25*mm* thick render. Good results were obtained for both repair techniques. The bare frame strength

was a maximum of 60*kN* at 3% drift and performed acceptably to drifts of 5%. The frame plus infill system attained its maximum strength of 180*kN* at a drift of 0.3%. The infill was completely "lost" at a drift of 2%.

Also recently, Yuksel et al (1998) in Turkey conducted tests of 1-storey, 1-bay RC frames with and without masonry infill walls. The influence of shear connectors through walls and steel mesh, steel mesh on walls, and shotcrete plastering were investigated. The degree of interlocking between the infill and frame was also studied. The bare frame plus infill without interlocking was twice as stiff and 25% stronger than the bare frame. The bare frame plus infill with shear-key interlocking of the brickwork was 50% stronger and at least twice as stiff as the bare frame specimen. The frame plus infill with interlocking plus retrofit with reinforced render with through bolting was 20% stronger and 75% stiffer than the frame plus infill with interlocking.

The benefit of grout injection of cracks and RC tie columns has been investigated in Romania by Irimies et al (1998). They conducted tests on URM brick walls that were strengthened with RC tie columns after having cracks repaired by grout injection. Of particular interest to this project is the result that the URM infill walls typically failed at lateral drifts of 0.6%.

2.7.5 Case Studies

The effectiveness of seismic retrofit techniques used in California to strengthen URM buildings has been studied through case studies following the Whittier Narrows, Loma Prieta and Northridge earthquakes. For example, Deppe (1988) presented the results of a survey of the performance of URM buildings during the Whittier Narrows earthquake. Damage patterns in buildings that had been strengthened prior to the earthquake were compared to those for buildings that were unstrengthened. From that analysis, the most effective ways for improving the seismic performance of URM buildings was identified. Bonneville and Cocke (1992) reported on the performance of URM buildings during the 1989 Loma Prieta earthquake that had been subjected to a minimal level of upgrade prior to that event. It was concluded that many buildings with a minimal upgrade (for example, parapet bracing and wall-to-diaphragm anchorage) were able to withstand the earthquake satisfactorily but would have suffered partial or full collapse had they not been upgraded. The effectiveness of the current minimal strengthening methods for moderate seismic loads was therefore demonstrated. Kehoe (1996) reported on the effectiveness of seismic retrofit techniques for URM buildings as evidenced by the damage observed in 40 previously retrofitted URM buildings during the Northridge earthquake. One problem that was identified was that the current URM retrofit and ABK rocking block theory does not correctly address corner, upper storey cracking and collapse failures. Otherwise, most of the techniques performed satisfactorily.

The assessment and seismic repair technique used to strengthen 42 of 152 buildings damaged in the 1995 Dinar, Turkey earthquake is described by Wasti et al (1998). Some laboratory test results for material properties are also given. The retrofit scheme consisted of: (1) wire/steel mesh fixed to exterior walls and rendered with 50mm thick layer of high-strength cement mortar, (2) RC columns added at corners to help "tie" the walls together, (3) stiffened roof diaphragms, and (4) window/door openings rebuilt or strengthened as necessary.

There has been much work in Europe on the topic of seismic retrofit of historical monuments and buildings. For example, Zarnic and Tomazevic (1986b) describe an historical 17th century 3-storey urban masonry building which was seismically retrofit by cement grouting of

the walls, foundation strengthening, replacement of timber floor with concrete slab and anchorage of walls to floors with steel ties. Briseghella and Negro (1986) analysed two different cathedrals in order to evaluate the potential effectiveness of wall strengthening and wall connections on the seismic resistance and Zingone, (1993) details the seismic restoration of the 800 year old Zisa Palace in Palermo, Italy. Other examples and case studies of seismic retrofit in Europe are given in Gavrilovic and Sendova (1995), Antonucci (1995) and La Mendola et al (1995).

2.8 Concrete Walls

2.8.1 Overview

The addition of concrete walls to existing concrete frame buildings is a common retrofit technique. This technique is able to provide substantial increases in strength and stiffness for a building. However, it must also be recognised that the seismic forces will tend to be concentrated in the stiffest elements. The foundations may need to be strengthened accordingly and this is not always easily or inexpensively done. Knoll (1983) presented 3 case studies where concrete shear walls were added as part of seismic retrofit works to 3 different buildings in Montreal, Canada. A more recent experimental study by Altin et al (1992) tested fourteen 2-storey by 2-bay concrete frames that were strengthened with cast-in-place concrete infill walls. The effectiveness of various degrees of inter-connection between the infill wall reinforcement and the surrounding concrete frame were assessed and all results were compared to the hysteretic behaviour of the bare concrete frame. It was observed that while the peak strength (reached at about 0.4% drift) was not sensitive to the degree of frame-wall connectivity, the hysteretic behaviour was best for the most integrally connected infill walls (maximum displacements corresponding to drifts of approximately 1 to 1.5%). Finally, in the recent European Conference on Earthquake Engineering, 3 papers were presented on this topic. The first of these presented the results of analyses on the seismic response of concrete frame school buildings in Taiwan retrofitted with concrete infill walls (Sheu et al, 1998). The paper by Pop et al (1998) presented the experimental results of tests on bonded anchors for use between concrete infill walls, which were added to pre-existing concrete frames. It was determined that an embedment length of 8 bar diameters into the concrete frame was required to achieve optimal force transfer and interaction. The third paper (Ozcebe et al, 1998) presented the results of an experimental investigation into the effectiveness of cast-in-place concrete walls as a seismic retrofit strategy for concrete frame buildings damaged in the 1995 Dinar, Turkey earthquake. The tests showed that the once the walls had been added the frame had little apparent effect on the strength of the rehabilitated structure. However, lap splices in the columns prevented the infill from achieving full effect. Steel plate jackets were recommended for the column splice zones to solve this problem.

2.8.2 Masonry Shear Walls for Seismic Retrofit

One of the earliest techniques used for seismic retrofit of concrete buildings was the addition of masonry shear walls. Examples of some of the research on this topic are detailed in the present section.

Much of this work was carried out in Japan. For example, the use of sidewalls and precast concrete panels to strengthen RC columns was studied by Higashi et al (1977). They also conducted experiments to verify the effectiveness. Higashi et al (1980) tested thirteen 1/3-scale 1-bay, 1-storey RC frames with poor column details that were strengthened by adding various shear walls. The frame base shear strength was seen to increase by between 131% to 430% for the variety of wall configurations considered. Around the same time, Sugano and

Fujimura (1980) tested ten 1-storey, 1-bay 1/3-scale RC frames that were strengthened by a variety of infill walls and bracing techniques. Design guidelines were developed based on their test results.

At the University of Texas at Austin, Valluvan et al (1992) conducted tests on strengthened column splices in columns of frames with RC infill walls. The retrofit consisted of adding RC infill walls to the frame and strengthening the column splice joints. This was done either by welding the splice bars and adding a stirrup or by increasing the confinement in the splice region. Three methods of confinement were considered. They were (1) adding corner angle steel with steel straps, (2) extra stirrups around the existing concrete, and (3) removal of cover concrete and addition of stirrups. The original wall strength of 380*kN* was increased by between 130% and 180%.

In Mexico, Ramirez-de-Alba et al (1992) reported on: (1) direct shear tests of beam-wall connections; (2) 2D tests of frame-wall components from a "weak-column strong-beam" frame retrofit with masonry infill walls; and (3) 3D tests of floor slab and waffle slab structures. The frame-wall system had an ultimate strength that was nearly 15 times greater than the strength of the bare frame. The frame-wall system reached its ultimate strength at a lateral drift of 1.5% after which the strength dropped off quickly. On the other hand, the bare frame reached its maximum strength at a drift of 2% and maintained that strength until reaching a drift of 3.6%. Canales et al (1992) reported on several seismic retrofit techniques that have been used to upgrade telephone buildings in Mexico City. The retrofit schemes employed varied in terms of material and depended largely on whether the existing building was a concrete or steel frame construction. Strengthening elements used were additional RC walls, additional steel bracing and in some cases additional steel moment resisting frames.

An experiment study by Bhende and Ovadia (1994) tested fully grouted RC masonry wall panels that were retrofitted with external steel plates attached to each face with through-bolts. Quasi-static out-of-plane cyclic loading was used. Lateral drifts of 2% to 3% were achieved. The use of epoxy bonded composite sheets (rather than steel plates) was investigated by Ehsani and Saadatmanesh (1997a,b). In their paper they present a method of epoxy bonding FRP sheets to damaged or under-strength masonry walls as a seismic retrofit technique. Key design issues are outlined and discussed.

Frosch et al (1996a) discuss the key issues involved with the use of pre-cast concrete panels for the retrofit of non-ductile RC frames. The precast concrete panels are used to infill selected bays in a frame building. This method involves the use of (1) shear keys into the frame elements and (2) grouted reinforced "closure strips" between the panels and between the panels and the frame. The grouted, infill walls reached their maximum strength at drifts of only 0.4%. Frosch et al (1996b) also tested a 2/3-scale model of a non-ductile RC frame that was strengthened with precast concrete infill panels. The main deficiencies of the original structure were (1) inadequate column shear strength, (2) poor column splice joint details and (3) poor anchorage details. The infill panels were designed to convert the lateral building response from that of a frame to that of a shear-wall structure.

2.9 Seismic Isolation

2.9.1 Overview

Of course, seismic isolation has been mentioned frequently as a possible retrofit technique for non-ductile buildings (e.g., Kelly, 1983). There has also been a number of very significant

retrofit projects that have employed the seismic isolation technique (Poole and Clendon, 1992; Nasseh, 1995; Mokha et al, 1996; Mayes et al, 1992; Elsesser et al, 1995). However, most of these projects have one thing in common – there were over-riding architectural/aesthetic or post-earthquake functionality factors which precluded the use of most if not all other conventional retrofit techniques. Because seismic isolation can be employed at the sub-foundation level, there is less intervention in the actual building. The costs associated with the implementation of seismic isolators under an existing building generally make it one of the more expensive retrofit options and so it is normally only used as a last resort. A number of recent papers have been published on this topic (e.g. Calderoni et al, 1998) but since it is not a likely candidate for the structure being considered in this project, only limited discussion (mainly case studies) is given to the topic here.

2.9.2 Case Studies

One of the earlier papers published on this topic is that by Korenev and Poliakov (1978) who analysed some of the problems in determining the efficiency of vibration absorbers employed to increase the seismic resistance of structures. The economic feasibility of seismic rehabilitation using base isolation was demonstrated by Kelly (1983) through a design study of a non-ductile building in San Francisco. The retrofit costs for the isolation scheme and alternative conventional retrofit schemes were compared, showing the isolation option to be the most economic for this particular case. Similar conclusions were reached by Delfosse and Delfosse (1992) when they conducted a feasibility study and found that rehabilitation by means of rubber base isolation bearings is safer and 64% cheaper than conventional retrofit of the superstructure.

Shaking table tests were conducted at Berkeley by Griffith et al (1990) on five different base isolation bearing systems under a 6-storey reinforced concrete frame plus shear wall building to study the effectiveness of seismic isolation of medium-rise buildings which are subject to column uplift. The effectiveness of positioning isolation devices at a number of different levels in a building was studied analytically by Keshtkar and Hanson (1992). A six-storey building with a soft first-storey was analysed with base isolation devices (bearings) and dampers positioned at various storey levels within the structure. This approach was seen to result in superior seismic response during strong earthquake input.

A number of large seismic retrofit projects employing base isolation have also been published in the last decade. Among these, Nasseh (1995) reviewed the architectural and structural characteristics of the San Francisco City Hall building before retrofit, the damage it sustained in the 1989 Loma Prieta earthquake, the repair and retrofit schemes studied, and the selected solution. A base isolation scheme combined with superstructure strengthening with concrete shear walls was adopted. Mokha et al (1996) have reported on the seismic isolation retrofit of the US Court of Appeals building in San Francisco. The procedure adopted for seismic evaluation of the existing building, establishment of the design approach, selection of the optimum isolator system and distribution. Elsesser (1993) reported on the seismic retrofit of the 18-storey Oakland City Hall tower that was damaged in the 1989 Loma Prieta earthquake. A combination of base-isolation, supplemental concrete shear walls and concentric steel tower bracing was used to retrofit the 1913 steel frame plus masonry infill tower.

Finally, in Italy Calderoni et al (1998) discussed the use of base isolation for retrofit of RC buildings. An example of a RC frame with brick infill was used to illustrate the effectiveness of isolation. It was noted that the additional stiffness of the infill wall panels greatly improved the isolation system's effectiveness. Furthermore, by decreasing the seismic

demands with isolation, the infill response was kept within "elastic" limits. Of particular interest to this project is the fact that the URM was modelled as having an ultimate strength of $\tau_u = 0.2MPa$, a "yield" shear strain of 0.0003 and a maximum shear strain of 0.0006.

2.10 Summary

In summary, the typical structural weaknesses that must be addressed by seismic retrofit strategies are reported in the literature to be:

- vertical and/or plan irregularities;
- columns weak in shear and/or flexure;
- beam-column joints weak in shear;
- inadequate splice joint lengths;
- inadequate development length;
- inadequate confinement steel; and
- unrestrained and/or under-strength masonry.

The building under consideration in this project will be seen (in Section 3) to possess most of the weakness in the above list. The various seismic retrofit techniques encountered in the literature which address, to varying degrees, the structural weaknesses listed above include:

- jacketing of existing structural elements and joints to improve their strength, stiffness, and/or ductility;
- addition of structural walls (precast or in situ concrete or masonry);
- tying of existing masonry infill walls to a surrounding frame with dowel connections, tie columns or tie beams;
- addition of structural bracing (with or without special damping devices); and
- seismic isolation of the entire structure.

In order to assist with the assessment of the seismic strength of the existing building in Section 3 of this report, an attempt at a quantitative summary of the most relevant results of the literature review (presented in Tables 1 and 2) is given 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) that the maximum shear stress occurs. Nevertheless, based on the literature it appears that this maximum stress may be assumed to occur at a lateral drift angle of approximately 0.3% (see for example Pires and Carvalho, 1992; Valiasis et al, 1993; Zarnic, 1995; Pires et al, 1995; Fardis and Calvi, 1995; Zarnic and Gostic, 1997; Schneider et al, 1998). Of course, the effect of the door and window openings and the degree of confinement of the infill by the frame will determine to a large extent how well the URM infill will behave with better confinement giving better hysteretic behaviour and less rapid strength degradation.

3. DESCRIPTION OF THE EXISTING BUILDING

3.1 Introduction

As stated in Section 1, the main objective of this study was to investigate possible seismic retrofit schemes for use in the seismic upgrade of a reinforced concrete frame. The building is a 4-storey, 3-bay reinforced concrete frame with unreinforced brick masonry infill walls. The concrete frame was designed essentially only 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 remaining sections of this chapter, the building will be described in particular and an attempt will be made to assess the likely seismic behaviour of the existing building. It is hoped that this exercise will assist in the identification of the structural components requiring seismic retrofit.

3.2 Frame Geometry, Section Details and Material Properties

The dimensions of the building and section details are shown in Figures 5 – 7. It can be seen in the elevation and plan drawings (Figure 5) that the storey heights are 2.7*m* and there are two 5*m* span bays and one 2.5*m* span bay. Brick masonry infill (200*mm* thick) is contained within each bay. The left-hand bay infill contains a window (1.2 x 1.1*m*) at each of the 4 levels. The central bay contains a doorway (2.0 x 1.9*m*) at ground level and window openings (2.0 x 1.1*m*) in each of the upper 3 levels of the building. The right-hand (2.5*m* span) bay contains solid infill (i.e., without openings). The beam reinforcement details are shown in Figure 6. It should be noted that the longitudinal reinforcing steel was smooth round bars, not the deformed steel bars used for reinforcement today. All beams in the direction of loading are 250*mm* wide and 500*mm* deep. The transverse beams are 200*mm* wide and 500*mm* deep. The concrete slab thickness is 150*mm*. The column reinforcement details are shown in Figure 7. The column splice joint detail and the column stirrup detail should be noted in particular. Their likely poor seismic performance will be discussed later in Section 3.7.

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 3 have been used. Estimates of the member strengths, likely structural behaviour under severe seismic overload conditions, relative strength and stiffness of the masonry infill and concrete frame and concrete frame details (beam-column joints, column splice joints, shear stirrups) are described in Sections 3.3 – 3.7.

Table 3 – Material properties.

Material	Relevant Properties (*mean values*)	
Steel	Yield strength:	$f_{sy} = 235MPa$
	Young's Modulus:	$E_{st} = 200x10^3 MPa$
Concrete	Ultimate stress & strain:	$f_c' = 24MPa, \varepsilon_{cu} = 0.003$
	Young's Modulus:	$E_c = 20x10^3 MPa$
Brick Masonry	Ultimate shear stress	$\tau_u = 0.4MPa$
	Ultimate shear strain:	$\gamma_u = 0.003$

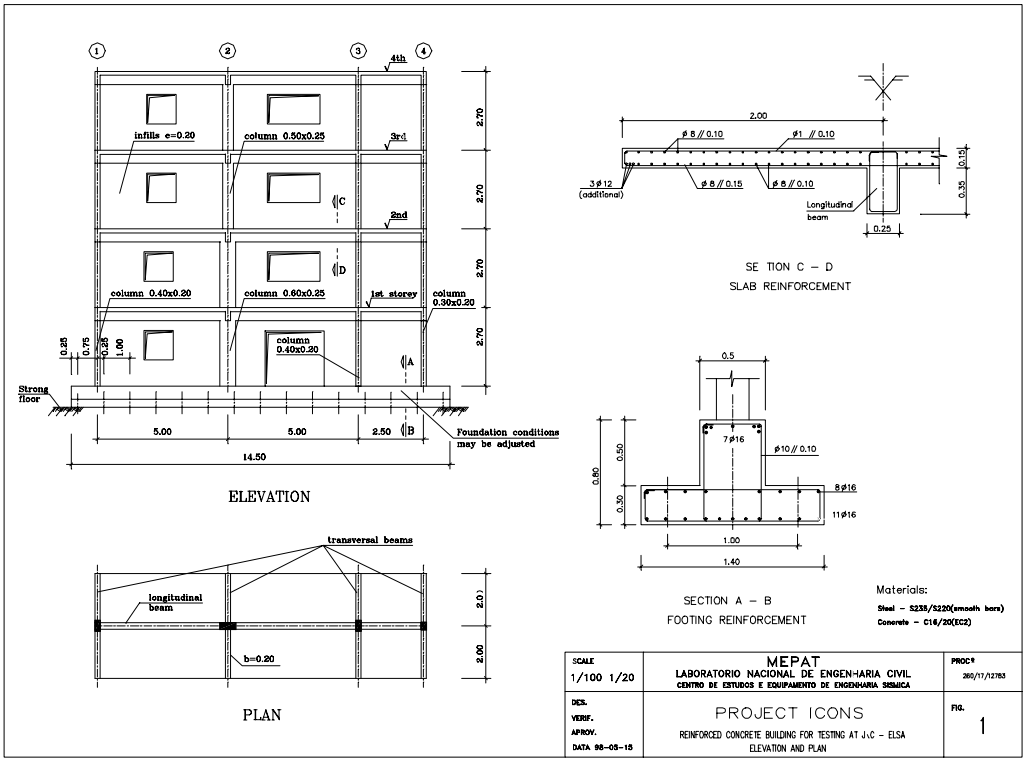

Figure 5 – Plan and elevation views of concrete frame plus masonry infill building.

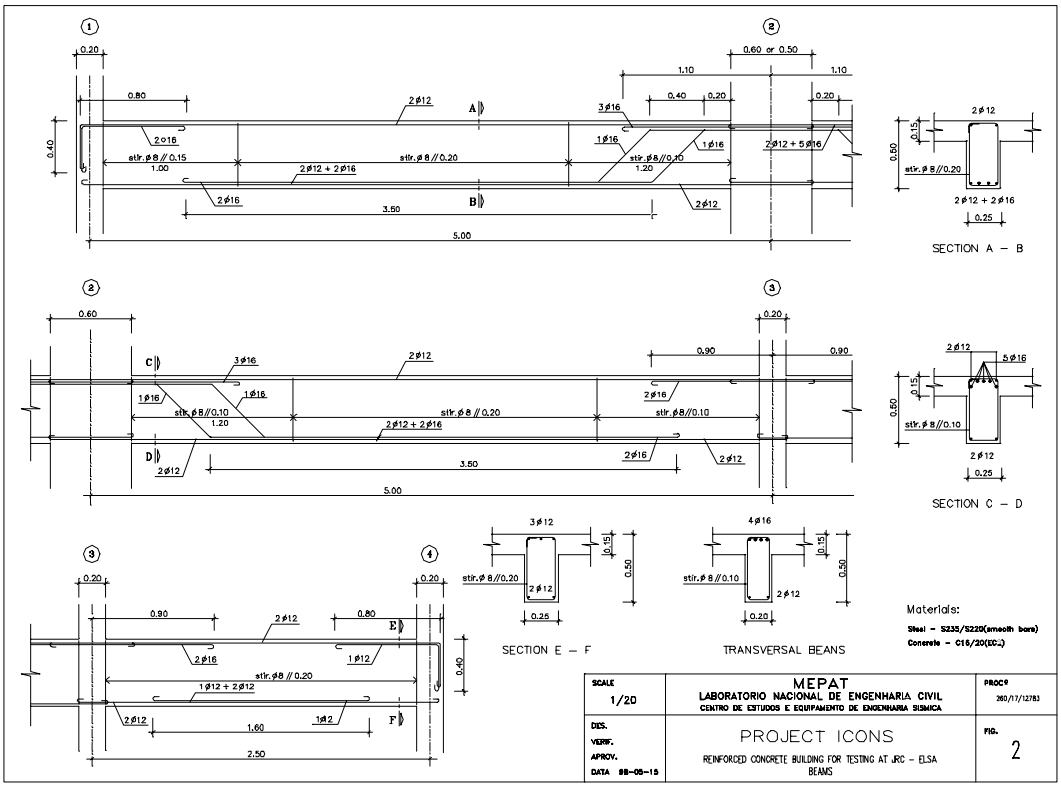

Figure 6 – Beam reinforcement details.

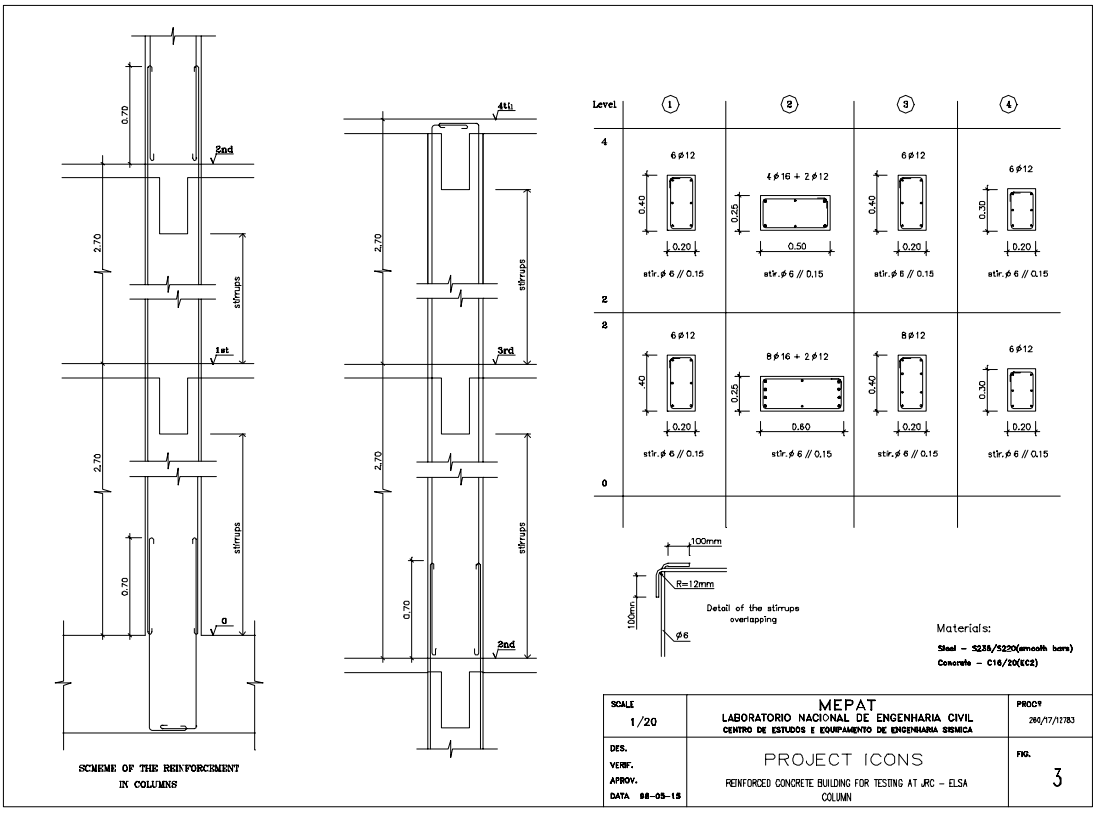

Figure 7 – Column reinforcement details.

3.3 Beam and Column Strengths

First, the ultimate moment capacity, M_u, was 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 3. The results of these calculations are listed in Table 4. The beam moment capacities are also indicated in Figure 8 below where the strong and weak direction bending moment capacities of the beams at the face of each column and at each beam midspan are shown. It should be noted that the beam cross-sections are labelled as A-B, C-D and E-F to be consistent with the nomenclature used in Figure 6.

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).

$$\frac{\sum M_{u,columns}}{\sum M_{u,beams}} \tag{1}$$

If the value given by equation (1) was greater than one, then no column-sidesway would be expected. (Note: in practice, the value given by equation (1) should be markedly greater than 1, say 1.4 for example (CEN, 1994), to ensure that a column sidesway mechanism is not

likely to occur.) The results of these calculations are presented in Figures 9 and 10 for sway to the right and to the left, respectively. Because the structure is not symmetric, it is slightly more prone to collapse when swaying to the left than to the right. In either case, it can be seen from the values in Figures 9 and 10 that the structure is highly susceptible to column sidesway collapse in either direction during seismic overload. Only at the ground storey is the sum of the column and beam moment capacities approximately equal. However, column sidesway is likely even at this level since the slab effects were not included in the calculation of the beam moment capacities.

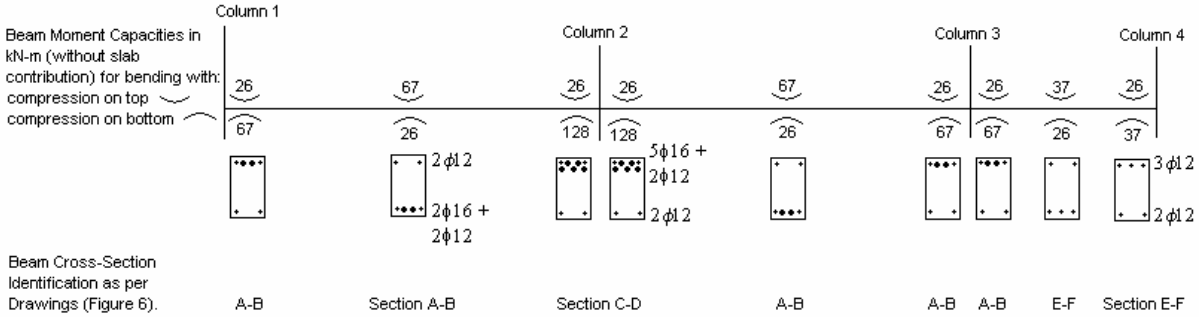

Figure 8 – Beam moment capacities.

Table 4 – Moment and shear capacity of beam and column cross-sections.

Cross-Section	M_u (kNm)	V_u (kN)	$V_p = 2M_u/h$ (kN)	V_u/V_p
Beam A-B	-26.2, +67.0	97.2	NA	NA
Beam C-D	-26.3, +127.9	156.7	NA	NA
Beam E-F	-26.0, +37.4	91.1	NA	NA
Column 1:				
Level 0-2	±14.6	48.5	13.3	3.7
Level 2-4	±14.6	48.5	13.3	3.7
Column 2:				
Level 0-2	±117.9	117.2	107.2	1.1
Level 2-4	±55.6	88.6	50.5	1.8
Column 3:				
Level 0-2	±18.9	52.4	17.2	3.1
Level 2-4	±14.6	48.5	13.3	3.7
Column 4:				
Level 0-2	±14.2	42.9	12.9	3.3
Level 2-4	±14.2	42.9	12.9	3.3

Note: (i) "-" bending moment capacity refers to weak direction curvature, "+" bending moment is flexural strength for opposite sense of curvature.
(ii) "NA" refers to fact that formula for V_p in column(4) is not applicable for beams.

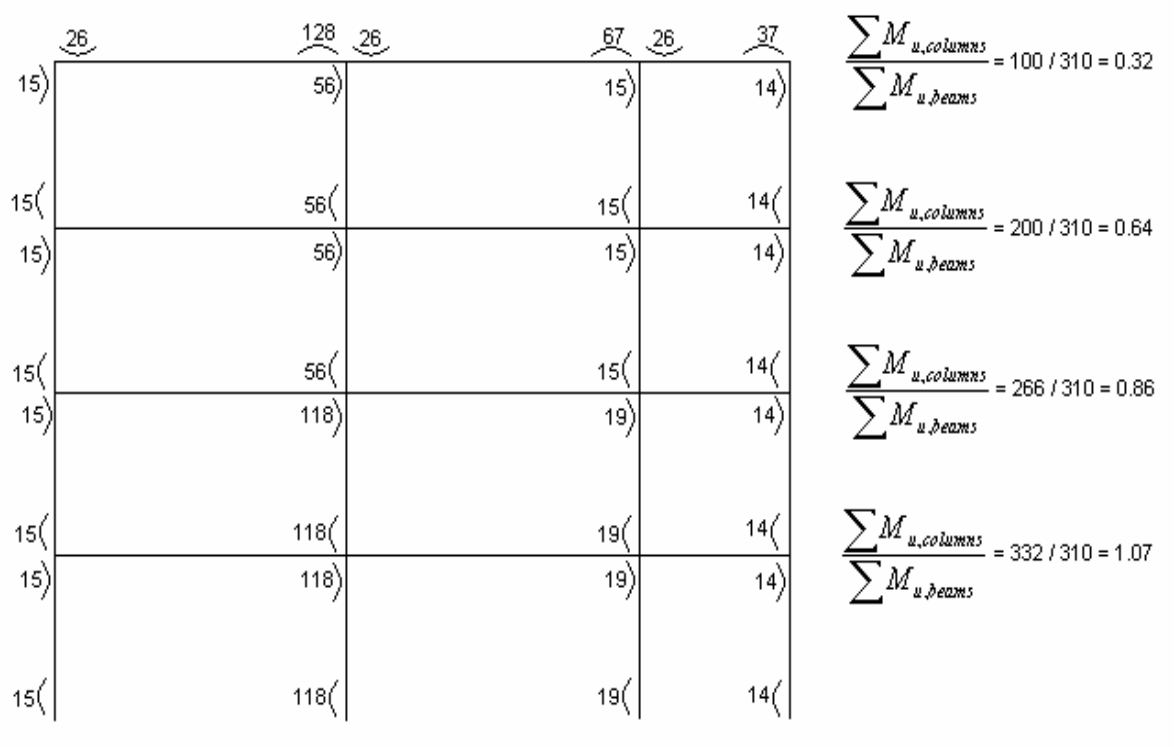

Figure 9 – Results of calculations to assess column sidesway vulnerability to the right. (Note: Figure indicates curvature and moment capacity (*kNm*) of beams and columns adjacent to the joints. Beam capacities are given only at the top since they are the same for each level.)

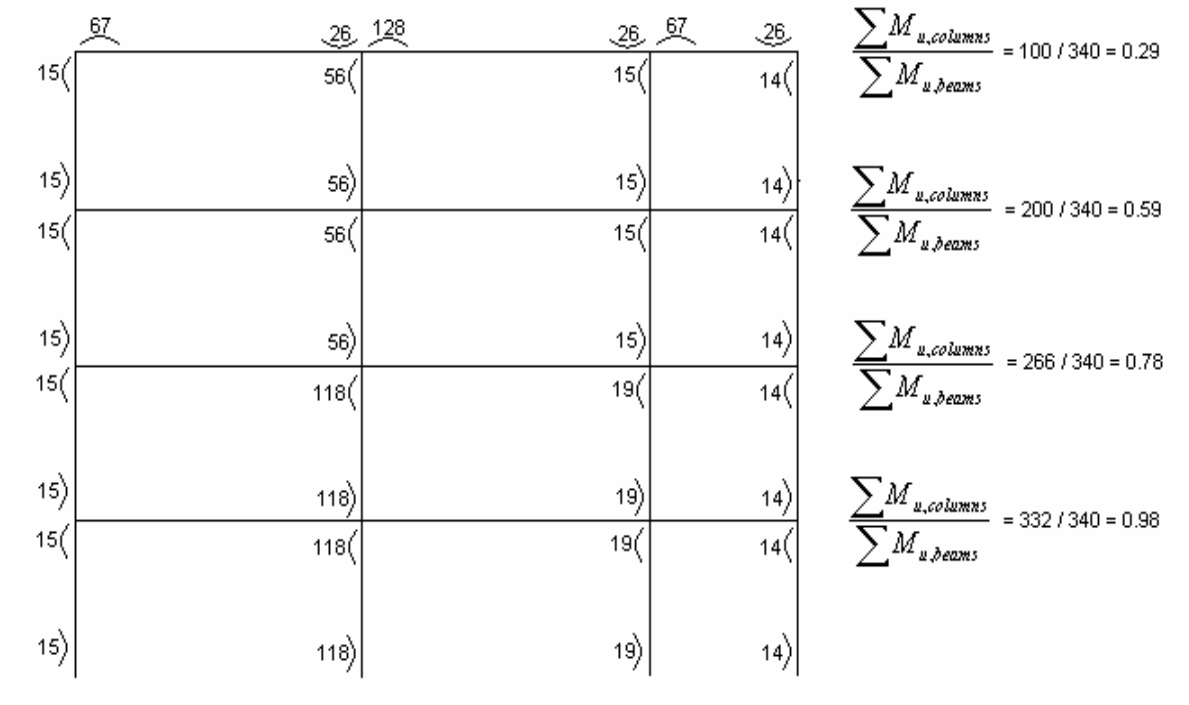

Figure 10 – Results of calculations to assess column sidesway vulnerability to the left. (Note: Figure indicates curvature and moment capacity (*kNm*) of beams and columns adjacent to the joints. Beam capacities are given only at the top since they are the same for each level.)

3.4 Column Shear Strength

The shear capacity of the columns, V_u, was estimated (using the ACI formula, see Appendix A for details) to determine whether the columns were likely to suffer shear failure before reaching their maximum flexural strength. The results of these calculations are given in column 3 of Table 4. Furthermore, the estimated shear capacity, V_u, was compared to the plastic column shear, V_p, which would exist if each column were subjected to its ultimate moment capacity, M_u, at both ends. The ratio of V_u/V_p is given in column 5 of Table 4. Note, $V_p = 2M_p/h$ where h is the free storey height of the column. A value greater than 1 suggests that premature shear failure will not occur. As can be seen from the values in column 5 of Table 4, there appears to be adequate shear capacity in all of the columns at all floor levels to prevent premature shear failure. Only in the bottom 2 storeys of column 2 where V_u/V_p = 1.1 is there a suggestion that shear may be critical.

3.5 Masonry Infill and Concrete Frame Shear Strengths

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. Based on experimental test results reported in the literature (Section 2), the in-plane shear strength of the masonry infill wall panels is likely to fall in the range of 0.2*MPa* to 0.6*MPa*. Taking the average of this range, 0.4*MPa* (as indicated in Table 3) and multiplying by the minimum effective cross-sectional area of masonry at each level of the building, $A_{e,m}$, the in-plane shear capacity of the masonry was estimated for each storey of the building. The critical sections for each storey were a horizontal plane through the window and door openings. The values are given below in Table 5. The maximum storey shear capacity for each level of the bare frame structure was estimated by simply summing up the plastic column shear values (column 4, Table 5) for each storey. The storey shear capacities estimated in this way for the bare frame are given below in Table 5. If one compares the frame and masonry infill shear strengths on a storey by storey basis, it is clear that if the masonry infill fails at some stage during seismic loading, then the total lateral shear strength of the structure will be greatly diminished. The estimated maximum base shear reaction for the bare frame is only 20% of the total combined base shear. Of course, the column storey shears will not achieve their maximum at the same lateral drift as will the masonry. Nevertheless, there is a large difference in the lateral strengths of the masonry and the concrete frame. Any retrofit scheme therefore needs to be capable of accommodating this difference in strength. It should also be noted that there is a substantial change in the shear strength of the concrete frame at level 2. The shear strength of the concrete frame above level 2 is only 60% of the frame's shear strength below level 2. As long as the masonry infill retains its load-carrying capacity, this will not be a problem. However, once the infill loses its strength, this difference in strength at level 2 of the frame is likely to become critical.

Table 5 – Storey shear strengths for masonry, bare frame and combined total.

Levels	Effective Shear Area of URM, $A_{e,m}$ ($x10^6 mm^2$)	$V_{u,masonry}$ ($= A_{e,m} \cdot 0.4MPa$) (kN)	$\sum V_p$ (kN)	$V_{u,total} = V_{u,masonry} + \sum V_p$ (kN)
2-4	1.68	672	90 ($0.12V_{u,total}$)	762
0-2	1.66	664	151 ($0.19V_{u,total}$)	815

3.6 Masonry Infill and Concrete Frame Shear Stiffnesses

The relative stiffness of the concrete frame and the brick masonry infill is another issue that must be addressed by the seismic retrofit scheme. Typical in-plane stiffness values for brick masonry infill are estimated by noting that most masonry infill was reported to reach its ultimate strength at lateral drifts of between 0.2% and 0.4% of the storey height. Using the ultimate shear strengths (column 3, Table 6) and assuming that the maximum strength occurs at $\Delta = 0.003h = 6.6mm$, values for the horizontal stiffness of the masonry infill were estimated (see column 2, Table 6). The lateral stiffness of the bare concrete frame was estimated by assuming the columns deflect in reverse curvature (as indicated in Figures 9 and 10) and using the formula

$$k_h = \frac{12EI}{h^3} \tag{2}$$

where E is Young's modulus for the concrete, I is the second moment of area for the column cross-section and h is the free storey height of the column. Here, $E = 20x10^3 MPa$, I was estimated as being 25% of the moment of inertia for the gross concrete cross-section and $h = 2200mm$. The values for the lateral storey frame stiffnesses estimated in this way are listed in column 3 of Table 6. The total storey shear stiffness for each level of the building is given in column 4 and it is clear that the frame contributes only modestly to the structure's lateral stiffness until the masonry infill fails. Again, once the masonry infill loses its load-carrying capacity the 35% decrease in the lateral stiffness of the concrete frame at level 2 will be critical. Even though the storey shear forces are likely to be smaller in the upper levels, the decrease in stiffness will amplify the upper storey drifts and potentially lead to an upper storey column sidesway collapse as was widely observed in Kobe, Japan (Park et al, 1995).

Table 6 – Lateral storey shear stiffness for the masonry, frame and total.

Level	Masonry Infill $k_m = V_{u,m} / 6.6mm$ (kN/mm)	Concrete Frame $k_c = \sum k_h$ (kN/mm)	Frame + Infill $k_t = k_m + k_c$ (kN/mm)
3-4	102	19 $(= 0.16k_t)$	121
2-3	102	19 $(= 0.16k_t)$	121
1-2	101	29 $(= 0.22k_t)$	130
0-1	101	29 $(= 0.22k_t)$	130

(Note: An alternative method for estimating the effective lateral stiffness for each storey of the bare concrete frame could have been used. This approach would simply have been to assume that the maximum frame shear strength is reached at each storey at a lateral storey

drift of 1%. Hence, by dividing the maximum storey shear strengths for the frame in Table 5, column 4 by the storey drift of 0.01*h* = 22*mm*, secant stiffness values can be obtained which are approximately one third of the values shown in column 3 of Table 6.)

3.7 Section and Joint Details

There are two aspects of the beam-column joint details shown in Figure 6 that are of concern. The first is related simply to the shear strength of the joint in view of the lack of additional joint reinforcement. In order to assess the potential seriousness of this concern, the average shear stress in the columns corresponding to the maximum moment capacity of the columns was estimated by dividing V_p by the gross column cross-section area, A_g. The range of shear stress values was between 0.17*MPa* and 0.79*MPa*. In comparison, the maximum joint shear stresses recorded by Beres et al (1992) were between $\sqrt{f_c'}$ and $1.2\sqrt{f_c'}$ in beam-column joints with a similar level of detailing to those in the building under consideration here. Thus, using 24*MPa* for f_c', the beam-column joints in the building under consideration should be able to withstand shear stresses of between 5*MPa* and 6*MPa*. Assuming that the joint shear stresses are not more than 5 to 10 times greater than the maximum column shear stress, then the joints should have adequate shear capacity without requiring additional reinforcement.

The second aspect concerns 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 these 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 might be expected that the joints in the building of interest should perform adequately, at least up to drifts of 2% to 2.5%.

Another detail which is of concern is the use of the 90° stirrup "overlapping details" (Figure 7) in the curtailment of the shear reinforcement for the beams and, more particularly, for the columns. Experience in recent earthquakes 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 150*mm* in all columns ($s = 10d_b$ or $12.5d_b$) and either 100*mm* or 200*mm* in the beams ($s = 8.3d_b$ or $16.7d_b$). These are remarkably close spacings in view of the age of the structure. Nevertheless, the "overlapping stirrup" detail shown in Figure 7 is not likely to withstand repeated cycles of lateral loading once concrete crushing has occurred and the concrete cover has been lost. In order to estimate the lateral drift at which this might occur, the section curvature, ϕ_u, in each column section at its corresponding ultimate strength M_u was first calculated. It was then assumed that the ultimate strength of the frame would correspond to a column sidesway mechanism as indicated in Figures 9 and 10. Each column was assumed to have "plastic hinges" at each end with lengths, L_p, equal to the effective depth, *d*, of the column cross-section. The drift angle at maximum curvature was then estimated by multiplying the maximum curvature ϕ_u by the plastic hinge length L_p. The results are shown below in Table 7. From the results it can be seen that concrete crushing of the column sections in the regions of maximum moment may be expected at lateral drifts of approximately 2% for column cross-sections 3 and 4, 2.5% for cross-section type 1 and 4% for cross-section type 2. Hence, it

might practically be expected that the concrete frame strength will degrade rather rapidly after it reaches a lateral drift of about 2%.

Table 7 – Lateral drift calculations for columns at maximum bending moment.

Column Cross-Section Type	Curvature ϕ_u at maximum M_u (1/*mm*)	Plastic hinge length, L_p (*mm*)	Lateral drift angle $\theta = L_p \cdot \phi_u$ (radians)	Lateral drift as % of storey height *h*
Column 1:				
Level 0-2:	0.00016	165	0.026	2.6%
Level 2-4	0.00016	165	0.026	2.6%
Column 2:				
Level 0-2	0.00008	565	0.045	4.5%
Level 2-4	0.00009	465	0.042	4.2%
Column 3:				
Level 0-2	0.00012	165	0.020	2.0%
Level 2-4	0.00016	165	0.026	2.6%
Column 4:				
Level 0-2	0.00012	165	0.020	2.0%
Level 2-4	0.00012	165	0.020	2.0%

Finally, smooth round reinforcement was used for the building in this project and 180° bends have been specified at the end of column splice joints and/or transition zones in beams. In the case of the column splice joints (Figure 7), the joint "development" length is 700*mm* (either $44d_b$ or $58d_b$, depending on the diameter d_b of the longitudinal reinforcement). This length in combination with the 180° bends is much greater than the $20d_b$ length required to develop yield strains in the longitudinal bars reported by Lynn et al (1996). Hence, no column splice joint failures are expected. With regard to the transition zone details (Figure 6), the main problems will occur in the beam-column joint regions where, under sidesway, the 180° bends may be positioned in a "tension" zone. The behaviour of this type of anchorage is not clear since little guidance was found in the literature.

3.8 Summary

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

(a) 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 660*kN* at a storey deformation of 6*mm* (approximately 0.3% drift). On the other hand, the bottom storey of the frame is estimated to reach its ultimate strength of 150*kN* at a drift of between 1% and 2%.

(b) 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.

(c) The seismic behaviour of the bare concrete frame is likely to be that of a "weak-column, strong-beam" mechanism under ultimate deformation conditions (Figures 9 – 10).

(d) The stirrup curtailment detail (Figure 7) which use 90° hooks is unlikely to withstand repeated cycles of large deformation (say ±2% drift).

4. RETROFIT STRATEGIES: OPTIONS

In this section of the report a number of seismic retrofit strategies will be discussed. Each has its relative merits. The decision as to which scheme is the most suitable will depend upon additional information not available at the time of writing. 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 are presented in the following sections.

4.1 Option 1: Replacement of URM infill with damped K-bracing

In this option, the URM infill would be replaced with K-bracing in the 2.5*m* 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 an appropriate force amplitude. 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.

To illustrate how this might be done, a sample damped bracing design was performed for the concrete frame building. These calculations are attached in Appendix B. The hysteretic devices were designed to "yield" at inter-storey drifts of 0.5% and reach a maximum drift of 1% in the DBE. The "elastic" stiffness of the devices was estimated using the design methodology outlined by Taucer (1998) which is based on deformation-based design principles. From this analysis, it was concluded that the optimum stiffness for the devices was 50% greater than the storey stiffness of the concrete frame at each level. The retrofit option just outlined is highly attractive because it increases the building's ductility without overly increasing its strength. Furthermore, the damped bracing is sufficiently stiff and damped so that it limits the storey drift during the DBE to approximately 1%, thereby keeping the columns within their recognised range of acceptable behaviour.

4.2 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.

4.3 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% in ‘Section 3.7. 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 must be minimised in this option. This could be done simply by concrete jacketing column number 2 above level 2. However, it should be noted that the estimated ultimate shear capacity of column 2 was essentially equal to the shear force required to balance the presence of the ultimate (plastic) moment capacity of the column at each end. Hence, under ultimate strength limit-state loading, column number 2 will experience both bending and shear forces nearly equal to its ultimate capacity. Composite jacketing could be applied to the full-height of column 2 in order to also increase its shear strength and prevent premature shear failure.

4.4 Option 4: Seismic Isolation

If major works to the building foundations are required regardless of which seismic retrofit option is selected, then seismic (base) isolation should also be considered. The main reason it is not more commonly used is because of the large costs associated with modifying the foundations of an existing building in order to accommodate the isolation devices. On the other hand, isolation systems are considered to be very efficient and reliable. Rather than increasing the resistance of the existing building, the isolation system works by reducing the seismic demands exacted on the building by the DBE. There are a large number of recent examples where this technique has been used to retrofit existing buildings. However, for the reasons described above, the applications have largely been on buildings with special historical, architectural or other aesthetic attributes that precluded the use of more “conventional” techniques.

4.5 Summary

In summary, four retrofit options have been briefly discussed in this section. Each of the options has its merits and all can be made to “work”. The “best” scheme can be decided only after considering the costs and benefits of each scheme. To do this, more refined analyses of the seismic behaviour of the “as-is” building and the building with the various retrofit options are required. In other words, more detailed “seismic assessment” (see Section 2.2) must be performed.

5. SUMMARY

In summary, a fairly comprehensive literature review (Section 2) has been 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 in Mediterranean European countries more than 40 years old 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 Section 3, the specific details of a 4-storey, 3-bay reinforced concrete frame building with unreinforced brick masonry (URM) infill walls were described. In particular, preliminary estimates of its likely weaknesses with regard to seismic loading were outlined. Based on this work, the concrete frame was found 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% 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, a number of retrofit options have been outlined in Section 4 of this report which require much closer scrutiny, perhaps using some of the seismic assessment methods described in Section 2.2 of this report. The retrofit options described consisted of:

- replacing the URM with damped bracing to create a ductile braced frame structure;
- strengthening the masonry and columns by jacketing to create a ductile shear wall structure;
- removing the URM infill and jacketing the columns and joints to create a ductile moment frame structure; and
- seismically isolating the superstructure to reduce the seismic demands to an acceptable level and allow the masonry infill to remain elastic.

The choice of retrofit scheme may, in the end, not be one of the above but rather some other combination of techniques. In any case, the final retrofit scheme will almost certainly need to address the concerns highlighted in Section 3 and that have been summarised here.

APPENDIX A – SHEAR STRENGTH CALCULATIONS

The shear strength of the beams and columns shown in Figures 6 and 7 were estimated using the ACI formulae outlined by Priestley et al (1994). The shear strength of a concrete member, V_u is assumed to be the sum of the strength due to concrete shear resistance, V_c, and strength due to steel shear reinforcement, V_s as indicated by equation A1.

$$V_u = V_c + V_s \tag{A1}$$

Simple strut and tie modelling suggests that the respective contributions involve the transverse reinforcement contributing transverse tie forces to a 45° truss in conjunction with diagonal concrete compression struts. Thus, the steel contribution to the shear strength is given by equation A2

$$V_s = \frac{A_v f_{sy} d}{s} \tag{A2}$$

where A_v = the total transverse reinforcement area per layer; s = the spacing along the member axis; f_{sy} = the steel yield strength; and d = the effective depth of the member, normally taken as 80% of the total depth D for rectangular beams and columns.

There are two alternate methods for evaluating V_c. The "simple" method was used in the calculations described in Section 3 of this report and is given by equation A3 as

$$V_c = v_b \left(1 + \frac{3P}{f_c' A_g}\right) b_w d \tag{A3}$$

where P = the axial load; b_w = the web width (taken as b for rectangular beams and columns); f_c' = the compressive cylinder strength of the concrete; A_g = the gross column area; and the basic shear stress is given by

$$v_b = \left(0.067 + 10\rho_w\right)\sqrt{f_c'} \le 0.2\sqrt{f_c'} \quad \text{(in } MPa\text{)} \tag{A4}$$

where ρ_w = the ratio of longitudinal tension reinforcement, taken as $0.5\rho_t$ for columns ($\rho_t = A_{st}/A_g$ = the gross longitudinal reinforcement ratio).

APPENDIX B – DESIGN CALCULATIONS FOR RETROFIT OPTION 1

The following set of design calculations is intended only to serve as an example of how damped bracing might be suitably designed for such a building. It was also intended to verify that the stiffness of the energy dissipation devices was within the realm of practicality. The steps below follow the methodology outlined by Taucer (1998) (see also REEDS, 1998) which is based on "deformation-based" design principles. Of course, more detailed analysis is required to verify the suitability of the bracing system design.

Step 1: Choose target drift.

A target drift of 1% was selected in order to keep the columns and joints within their expected range of acceptable behaviour. That is, the exterior joints are expected to begin rapid strength degradation at 1.5% drift and columns to lose their axial load capacity at 2% drift. In addition, the sway index for the building suggests that a soft-ground storey collapse mechanism is likely. Hence, the 1% drift value was nominated for this retrofit design.

Step 2: Calculate the damping expected of the hysteretic device.

Now, $\tan\delta = \frac{A_{diss}}{A_{rect}}$ depends on the yield deformation Δ_y. For this example, I have chosen Δ_y to correspond to a storey drift of 0.5% (ie, 13.5*mm*). Thus, it can be shown that $\tan\delta = 0.5$ for an elastic-plastic hysteresis loop which has its yield displacement equal to half of the ultimate displacement.

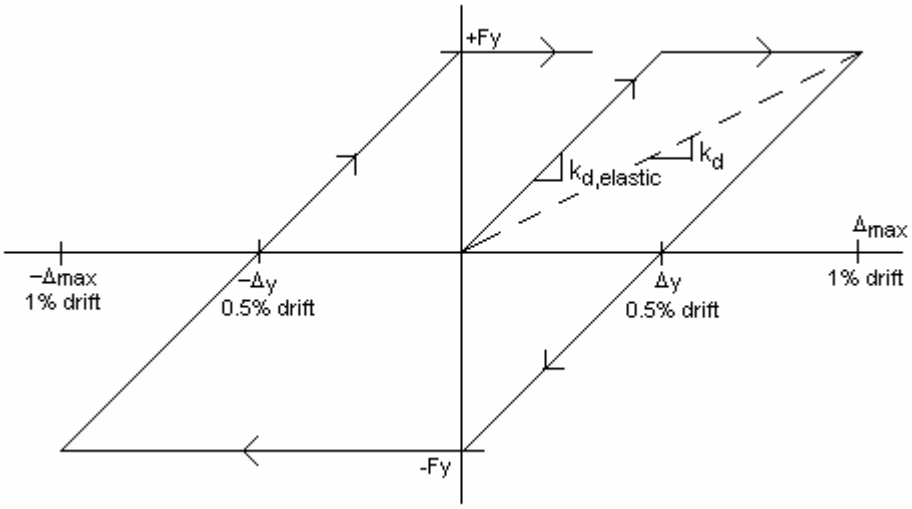

Figure 1 – Idealised force-displacement hysteresis loop for hysteretic damping device.

Step 3: Calculate the fundamental period for the equivalent SDOF system.

The weight of the structure, *W*, is estimated by assuming that the *DL* + 0.4*LL* = 5*kPa* and multiplying 5*kPa* by the effective floor area for 1 frame of $12.5x4 = 50m^2$ to get 250*kN* per floor. There are 4 storeys so the total weight of the structure was estimated to be $W = 1700kN$.

The effective "global" stiffness for the SDOF system, $K_{S,eff}$ was calculated by assuming that the structural mass for the SDOF system was located at 2/3rd the total height of the building

and that the peak base shear strength of the building would be reached at a lateral drift of 1%. Thus,

$$K_{S,eff} = \frac{V_b}{0.01x(0.67h_t)} = \frac{150kN}{0.01x7200mm} = 2.08kN/mm.$$

Finally, the natural frequency and period of the SDOF system were found to be

$$\omega_s = \sqrt{\frac{2.08kN/mm \cdot 9810mm/s^2}{1700kN}} = 3.46rad/\sec \text{ and } T_s = 1.81\sec.$$

<u>Step 4: Determine the optimum value for the design parameter *f*.</u>

Note, the equivalent viscous damping provided by the hysteretic device can be estimated from the formula $\xi_d = \frac{\tan\delta}{2} f$ where $f = \left(\frac{K_d}{K_d + K_s}\right)$. Other relationships used in the following table are: $K_d = \left(\frac{f}{1-f}\right)K_s$ and $\omega_d^2 = \left(\frac{1}{1-f}\right)\omega_s^2$. The table below was used to find a suitable value for *f* that yielded a drift displacement for the damped braced system that corresponded to a drift of 1%. The displacements were estimated using the damped period T_d and value of total damping, $\xi_t = \xi_d + 0.05$ and the NZS 4203:1992 displacement spectra for intermediate soils.

f	$(\omega_d/\omega_s)^2$	ξ_d	ξ_t	T_d (sec.)	Δ_d (mm)
0	1	0	0.05	1.81	220
0.2	1.25	0.05	0.10	1.62	150
0.3	1.43	0.075	0.125	1.52	130
0.4	1.67	0.10	0.15	1.40	110
0.5	2	0.125	0.175	1.28	100
0.6	2.5	0.15	0.20	1.15	75
0.7	3.3	0.175	0.225	1.00	60
0.8	5	0.20	0.25	0.81	40
1	∞	0.25	0.3	0	0

For a target drift of 1% (72*mm* at the height for the "equivalent" SDOF system), it appears that a value for *f* of about 0.6 should be used. The value of $f = 0.6$ is used in Step 5 below.

<u>Step 5: Calculate the stiffness of the hysteretic devices at the "local" level.</u>

Using the expression $K_d = \left(\frac{f}{1-f}\right)K_s$ and noting that for $f = 0.6$, $K_d = 1.5K_s$. Hence, the local stiffness for each hysteretic device was to be set equal to the storey stiffness. The storey stiffnesses at each level were estimated by dividing the storey shear capacity by the target storey drift of 1% (i.e., 27*mm*) to give

$$k_d = 1.5k_s = \begin{cases} 1.5 \cdot 90kN/27mm = 5.00\,kN/mm \text{ for levels } 2\text{ - }4 \\ 1.5 \cdot 150kN/27mm = 8.33\,kN/mm \text{ for levels } 0\text{ - }2 \end{cases}$$

However, it should be noted that these values are the "secant" stiffness values for the devices at 1% drift. The elastic stiffness values are just double the secant stiffness values as indicated below and illustrated earlier in Figure 1.

$$k_{d,elastic} = \begin{cases} 10.0 kN/mm \text{ for levels } 2\text{-}4 \\ 16.7 kN/mm \text{ for levels } 0\text{-}2 \end{cases}$$

In Summary:

For levels 2-4: $k_{d,elastic} = 10.0\, kN/mm$, $F_y = 135 kN$ **and** $\Delta_y = 13.5 mm$.

For levels 0-2: $k_{d,elastic} = 16.7\, kN/mm$, $F_y = 225 kN$ **and** $\Delta_y = 13.5 mm$.

Further refinements in this design may be made through more detailed analysis. In addition, it may be desirable to use the stiffer bracing at all four levels of the building in order to reduce the change in strength and stiffness which exists in the bare concrete frame structure as discussed in Sections 3.5 and 3.6 of this report.

APPENDIX C – BIBLIOGRAPHY

ABK, 1984. "Methodology for mitigation of seismic hazards in existing unreinforced buildings: The methodology," Agbabian Assoc., Report ABK-TR-08, El Segundo, CA.

Aboutaha, R.S., Engelhardt, M.D., Jirsa, J.O. and Kreger, M.E., 1996(a). "Retrofit of concrete columns with inadequate lap splices by the use of rectangular steel jackets," Earthquake Spectra, Vol. 12(4), pp. 693-714.

Aboutaha, R.S. and Jirsa, J.O., 1996(b). "Steel jackets for seismic strengthening of concrete columns," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 1, Paper No. 518.

Abrams, D.P., Angel, R. and Uzarski, J., 1996. "Out-of-plane strength of unreinforced masonry infill panels," Earthquake Spectra, Vol. 12(4), pp. 825-844.

Abrams, D.P. (ed.), 1994. "Proceedings from the NCEER workshop on seismic response of masonry infills," National Center of Earthquake Engineering Research, Report No. NCEER-94-0004, State University of New York at Buffalo, 138p.

Abrams, D.P. and Calvi, G.M. (eds.), 1994. "Proceedings of the US – Italian workshop on guidelines for seismic evaluation and rehabilitation of unreinforced masonry buildings," National Center of Earthquake Engineering Research, Report No. NCEEER-92-0021, State University of New York at Buffalo, 276p.

ACI Special Publication No.160, 1996. "Seismic rehabilitation of concrete structures."

Aiken, I., Nims, D., Whittaker, A. and Kelly, J., 1993. "Testing of passive energy dissipation systems," Earthquake Spectra, Vol. 9(3), pp. 335-370.

Alcocer, S.M., 1992. "Rehabilitation of RC frame connections using jacketing," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5235-5240.

Alcocer, S.M. and Jirsa, J.O., 1990. "Assessment of the response of reinforced concrete frame connections redesigned by jacketing," Proceedings of the 4th US National Conference on Earthquake Engineering, EERC, El Cerrito, California, Vol. 3, pp. 295-304.

Al-Hussaini, T.M., Zayas, V.A. and Constantinou, M.C., 1994. "Seismic isolation of multi-story frame structures using spherical sliding isolation systems," National Center for Earthquake Engineering Research, Report No. NCEER-94-0007, State University of New York at Buffalo, 180p.

Altin, S., Ersoy, U. and Tankut, T., 1992. "Hysteretic response of RC infilled frames," ASCE Journal of Structural Engineering, Vol. 118(8), pp. 2133-2150.

An, W., Saadatmanesh, H. and Ehsani, M.R., 1991. "RC beams strengthened with FRP plates. II: analysis and parametric study," ASCE Journal of Structural Engineering, Vol. 117(11), pp. 3434-3455.

Anicic, D., 1995. "Repair and retrofit of building structures: state-of-the-art report," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2199-2208.

Antonucci, R., 1995. "Rehabilitation of Villa Favorita," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2221-2226.

Antonucci, R. and Giacchetti, R., 1992. "Experimental analysis of a discontinuous wall-to-floor connection," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5355-5359.

Applied Technology Council, 1998. "Guidelines for the seismic rehabilitation of buildings," ATC-33 Report, Redwood City, California.

Applied Technology Council, 1996. "Seismic evaluation and retrofit of concrete buildings," ATC-40 Report, Redwood City, California.

Applied Technology Council, 1992. "Evaluation of the performance of seismically retrofitted buildings," ATC-31 Report, Redwood City, California, 75p.

Badaloukas, G.N., Giaka, A.P., Papadopoulos, G.A. and Dadalouka, B.G., 1998. "A strengthening model for concrete frames using diagonal tendons," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 6p.

Badoux, M., 1998(a). "Comparison of seismic retrofitting strategies with the capacity spectrum method," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 8p.

Badoux, M., 1998(b). "Developments in seismic retrofitting research in the USA," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 13p.

Badoux, M. and Jirsa, J.O., 1990. "Steel bracing of RC frames for seismic retrofitting," ASCE Journal of Structural Engineering, Vol. 116(1), pp. 55-74.

Badoux, M. and Jirsa, J.O., 1987. "Retrofitting of reinforced concrete structures with steel bracing systems," Proceedings, IABSE Symposium, Paris.

Bass, R.A., Carrasquillo, R.L. and Jirsa, J.O., 1989. "Shear transfer across new and existing concrete interfaces," ACI Structural Journal, Vol. 86(4), pp. 383-393.

Bastos, J.N., Jirsa, J.O. and Klingner, R.E., 1986. "The role of subassemblage tests in modeling connections of full-scale structures," Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Vol. 4, pp. 7.4/65-72.

Benedetti, D., Carydis, P. and Pezzoli, P., 1998. "Shaking table tests on 24 simple masonry buildings," Earthquake Engineering and Structural Dynamics, Vol. 27(1), pp. 67-90.

Benedetti, D. and Vitiello, E., 1978. "Strengthening of masonry buildings," Proceedings, IABSE Seminar on Constructions in Seismic Zones, Bergamo.

Benuska, K.L. (editor), 1990. "Loma Prieta earthquake reconnaissance report," Earthquake Spectra, Supplement to Vol. 6, 448p.

Beres, A., White, R.N., Gergely, P., Pessiki, S.P. and El-Attar, A., 1992(a). "Behavior of existing non-seismically detailed reinforced concrete frames," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 6, pp. 3359-3363.

Beres, A., El-Borgi, S., White, R.N. and Gergely, P., 1992(b). "Experimental results of repaired and retrofitted beam-column joint tests in lightly reinforced concrete frame buildings," National Center for Earthquake Engineering Research, Report No. NCEER-92-0025, State University of New York at Buffalo, 84p.

Bertero, V.V., 1992. "Seismic upgrading of existing structures," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5101-5106.

Bertero, V.V. and Brokken, S., 1983. "Infills in seismic resistant building," ASCE Journal of Structural Engineering, Vol. 109(6), pp. 1337-1361.

Bett, B.J., Klingner, R.E. and Jirsa, J.O., 1988. "Lateral load response of strengthened and repaired reinforced concrete columns," ACI Structural Journal, Vol. 85(5), pp. 499-508.

Bhende, D. and Ovadia, D., 1994. "Out-of-plane strengthening scheme for reinforced masonry walls," Concrete International, Vol. 16(4), pp. 30-34.

Bolong, Z., Mingshun, W. and Deyuan, Z., 1988. "Shaking table study of a five-story unreinforced block masonry model building strengthened with reinforced concrete columns and tie bars," Proceedings of the US-PRC Joint Workshop on Seismic Resistance of Masonry Structures, Harbin, pp. 1-11.

Bolong, Z., Zhixian, J. and Mingshun, W., 1982. "A study on aseismic capacity of brick masonry buildings strengthened with reinforced concrete columns and tie bars," Proceedings, US-People's Republic of China Bilateral Workshop on Earthquake Engineering, Vol. 1.

Bonacci, J.F., 1994. "Design forces for drift and damage control: a second look at the substitute structure approach," Vol. 10(2), pp. 319-331.

Bonneville, D.R. and Cocke, D.W., 1992. "Observed effectiveness of minimal seismic strengthening of unreinforced masonry buildings," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5401-5405.

Bozzo, L.M., Foti, D. and Lopez-Almansa, F., 1996. "Design criteria for e/q resistant buildings with energy dissipators," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 4, Paper No. 1933.

Bracci, J.M., Kunnath, S.K. and Reihnorn, A.M., 1997. "Seismic performance and retrofit evaluation of reinforced concrete structures," ASCE Journal of Structural Engineering, Vol. 123(1), pp. 3-10.

Bracci, J.M., Reinhorn, A.M. and Mander, J.B., 1995(a). "Seismic resistance of reinforced concrete frame structures designed for gravity loads: performance of structural system," ACI Structural Journal, Vol. 92(5), pp. 597-609.

Bracci, J.M., Reinhorn, A.M. and Mander, J.B., 1995(b). "Seismic retrofit of reinforced concrete buildings designed for gravity loads: performance of structural model," ACI Structural Journal, Vol. 92(6), pp. 711-723.

Bracci, J.M., Reinhorn, A.M. and Mander, J.B., 1992. "Evaluation of seismic retrofit of reinforced concrete frame structures: Part 1 – experimental performance and analytical study of a retrofitted structural model," National Center for Earthquake Engineering Research, Report No. NCEER-92-0031, State University of New York at Buffalo, 204p.

Braga, F., Liberatore, D. and Spera, G., 1998. "Numerical investigation on the effectiveness of vertical prestressing in strengthening masonry walls," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 8p.

Brandow, G.E. and Hart, C.H., 1995. "Seismic strengthening of a historical concrete frame with brick infills using base isolation," Proceedings of the 3rd National Concrete and Masonry Engineering Conference, San Francisco, California.

Briseghella, L. and Negro, P., 1986. "The Venzone and Gemona cathedrals reconstruction after the 1976 Friuli earthquakes," Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Vol. 5, pp. 11.2/9-16.

Bruneau, M., 1994. "State-of-the-art report on seismic performance of urm buildings," ASCE Journal of Structural Engineering, Vol. 120(1), pp. 231-251.

Buccino, F. and Vitiello, E., 1995. "Experimental behaviour and design criteria for anchorage between slabs and brick walls," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2255-2260.

Buckle, I.G. (ed.), 1994. "The Northridge, California earthquake of January 17, 1994: Performance of highway bridges," National Center of Earthquake Engineering Research, Report No. NCEER-94-0008, State University of New York at Buffalo, 132p.

Buckle, I.G. (ed.), 1992. "Proceedings from the first US-Japan workshop on earthquake protective systems for bridges," National Center for Earthquake Engineering Research, Report No. NCEER-92-0004, State University of New York at Buffalo, 614p.

Buckle, I. and Friedland, I. (eds.), 1994. "Proceedings from the third US-Japan workshop on earthquake protective systems for bridges," National Center for Earthquake Engineering Research, Report No. NCEER-94-009, State University of New York at Buffalo, 616p.

Bush, T.D. Jr., Talton, C.R. and Jirsa, J.O., 1990. "Behavior of a structure strengthened using reinforced concrete piers," ACI Structural Journal, Vol. 87(5), pp. 557-563.

Bush, T., Roach, C., Jones, E. and Jirsa, J., 1986. "Behavior of a strengthened reinforced concrete frame," Proceedings of the 2nd US National Conference on Earthquake Engineering, Charleston, Vol. 2, pp. 1203-1214.

Cagley, J.R., 1978. "Seismic hardening of unreinforced masonry walls through a surface treatment," NSF Workshop Proceedings.

Calderoni, B., Ghersi, A. and Rinaldi, D., 1998. "Base isolation as a retrofitting technique for existing R.C. buildings," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 12p.

Cahis, X., Bozzo, L. and Torresk, L., 1998. "Experimental studies of various innovative energy dissipation devices," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 9p.

Canales, M.D. and Briseno de la Vega, R., 1992. "Retrofitting techniques used in telephone buildings in Mexico," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5143-5147.

Carvalho, E.C., 1998. Personal communication to ICONS – Topic 2 participants.

Carydis, P., Mouzakis, H., Benedetti, D. and Pezzoli, P., 1998. "Repair and strengthening of seismically damaged traditional masonry buildings," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 10p.

Carydis, P.G., Mouzakis, H.P., Taflambas, J.M. and Vougioukas, E.A., 1992. "Response of infilled frames with brickwalls to earthquake motions," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 5, pp. 2829-2834.

CEB, 1997. "Fastenings for seismic retrofitting," Comite Euro-International du Beton, State-of-the-art report, London, Thomas Telford.

CEN, 1994. "Eurocode 8 – Design provisions for earthquake resistance of structures," Comite Europeen de Normalisation, Burssels.

CEN, 1998. "Eurocode 8, Part 1-4: Strengthening and repair of buildings," Draft, prENV 1998 – 1-4, Comite Europeen de Normalisation, Brussels.

Chai, Y.H., Priestley, M.J.N. and Seible, F., 1994. "Analytical model for steel-jacketed RC circular bridge columns," ASCE Journal of Structural Engineering, Vol. 120(8), pp. 2358-2376.

Chang, K.C., Soong, T.T., Oh, S.-T. and Lai, M.L., 1992. "Seismic response of steel-frame structures with added viscoelastic dampers," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5169-5173.

Choudhuri, D., Mander, J.B. and Reihnorn, A.M., 1992. "Evaluation of seismic retrofit of reinforced concrete frame structures: Part 1 – experimental performance of retrofitted sub-assemblages," National Center for Earthquake Engineering Research, Report No. NCEER-92-0030, State University of New York at Buffalo, 136p.

Chronopoulos, M., Scarpas, A. and Tassios, T.P., 1995. "Response of original and repaired R/C joints under cyclic imposed deformations," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2261-2267.

Chrysostomou, C.Z., Gergely, P. and Abel, J.F., 1992. "Nonlinear seismic response of infilled steel frames," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 8, pp. 4435-4437.

Chung, H.W. and Lui, L.M., 1978. "Epoxy-repaired concrete joints under dynamic loads," Journal of the American Concrete Institute, Vol. 75(7), pp. 313-316.

Ciampoli, M., 1995. "Upgrading R/C bridges for seismic risk reduction," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2209-2214.

Combescure, D., 1996. "Modelisation du comportement sous chargement sismique des structures de batiment comportant de murs de remplissage en maconnerie," PhD thesis, Ecole Centrale de Paris.

Committee on SPRC, 1993. "Application examples of retrofitting for existing R/C buildings in Shizuoka Prefecture," Tokyo, Japan Building Disaster Prevention Association (in Japanese).

Constantinou, M.C. and Symans, M.D., 1992. 'Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers," National Center for Earthquake Engineering Research, Report No. NCEER-92-0032, State University of New York at Buffalo, 211p.

Davis, H.A., 1978. "The center for educational development, a medium scale seismic upgrading," Proceedings, North American Masonry Conference.

Delfosse, G.C. and Delfoss, P.G., 1992. "Seismic rehabilitation of a shear wall building by means of base isolation," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5159-5162.

Del Valle-Calderon, E., Foutch, D.A., Hjelmstad, K.D., Figueroa-Gutierrez, E. and Tena-Colunga, A., 1989. "Seismic retrofit of a rc building: a case study," Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Vol. 7, pp. 451-456.

Deppe, K., 1988. "Evaluation of strengthened and unstrengthened unreinforced masonry buildings in Los Angeles city," Earthquake Spectra, Vol. 4(1), pp. 157-180.

Diaconu, D. and Vasilescu, D., 1997. "Studies on the bearing capacity of confined masonry members (short columns)," Report 3328, Iasi, INCERC.

Diaconu, D. and Vasilescu, D., 1996. "Studies on strengthening masonry buildings damaged by earthquakes," Report 695, Iasi, INCERC.

Dorka, U.E., Ji, A. and Flygare, E., 1998. "A hysteretic device system for earthquake retrofit of large panel buildings," Proceedings of the 11th European Conference on Earthquake Engineering, Paris.

Dovich, L. and Wight, J.K., 1996. "Lateral response of older flat slab frames and the economic effect of retrofit," Earthquake Spectra, Vol. 12(4), pp. 667-691.

Durrani, A.J. and Zerbe, H.E., 1987. "Seismic resistance of r/c exterior connections with a floor slab," ASCE Journal of Structural Engineering, Vol. 113(8), pp. 1850-1864.

Dyke, S.J. and Spencer, Jr. B.F., 1994. "Experimental verification of acceleration feedback control strategies for an active tendon system," National Center for Earthquake Engineering Research, Report No. NCEER-94-0024, State University of New York at Buffalo, 102p.

Dyngeland, T., 1998. "Retrofitting of bridges and building structures – a literature survey," European Laboratory for Structural Assessment, JRC-ISIS, Special Publication No. I.98.33, Ispra, 19p.

EERC General Report, 1995. "Repair and strengthening of moderately damaged rural structures in Dinar," Middle East Technical University, Ankara.

Ehsani, M.R., Saadatmanesh, H. and Velazquez-Dimas, J., 1998. "Retrofit of clay brick walls with fiber composites," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 7p.

Ehsani M. and Saadatmanesh, H., 1997(a). "Fabric composites: an economical alternative for retrofitting earthquake damaged precast walls," Earthquake Spectra, Vol. 13(2), pp. 225-241.

Ehsani, M.R., Saadatmanesh, H. and Al-Saidy, A., 1997(b). "Shear behavior of URM retrofitted with FRP overlays," Journal of Composites for Construction, Vol. 1(1), pp. 17-25.

Ehsani, M. and Saadatmanesh, H., 1996. "Seismic retrofitting of URM walls with fiber composites," The Masonry Society Journal, Vol. 14(2), pp. 63-72.

El-Attar, A.G., White, R.N. and Gergely, P., 1997. "Behaviour of gravity load designed reinforced concrete buildings subjected to earthquakes," ACI Structural Journal, Vol. 94(2), pp. 133-145.

Eligehausen, R. and Vintzeleou, E., 1989. "Behaviour of metallic anchors under generalized excitations," Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Vol. 7, pp. 505-510.

Elnashai, A.S. and Pinho, R., 1998. "Current status and future developments for repair and strengthening of RC structures in European seismic design practice," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 12p.

Elnashai, A.S. and Salama, A.I., 1992. "Selective repair and retrofitting techniques for RC structures in seismic regions," Engineering Seismology and Earthquake Engineering Section, Imperial College, London, Research ESEE/92-2.

Elsesser, E., Honeck, W. and Walters, M.T., 1995. "Seismic retrofit of the Oakland City Hall, Oakland, California," Structural Engineering International, Vol. 5(1), pp. 12-14.

Elsesser, E., 1993. "The seismic repair and rehabilitation of Oakland City Hall," Seismic Rehabilitation of Existing Buildings, EERI, Oakland, California, 34p.

Endo, T., Okifuji, A., Sugano, S., Hayashi, T., Shimizu, T., Takahara, K., Saito, H. and Yoneyama, Y., 1984. "Practices of seismic retrofit of existing concrete structures in Japan," Proceedings of the 8th World Conference on Earthquake Engineering, San Francisco, Prentice-Hall, New Jersey, Vol. 1, pp. 469-476.

Ersoy, U., 1996. "Seismic rehabilitation – application, research and current needs," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 4, Paper No. 2099.

Ersoy, U., Tankut, A.T. and Suleiman, R., 1993. "Behavior of jacketed columns," Journal of the American Concrete Institute, Vol. 90(3), pp. 288-293.

Ersoy, U., Altin, S. and Tankut, T., 1990. "Seismic strengthening of R/C frames with R/C infills," Middle East Technical University Report No. METU/SML-90/01, Ankara, Turkey.

Fardis, M.N., 1998. "Invited lecture: Seismic assessment and retrofit of r/c structures," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam.

Fardis, M.N., 1996. "Experimental and numerical investigations on the seismic response of RC infilled frames and recommendation for code provisions," Report No. 6: ECOEST/PREC8.

Fardis, M.N. and Calvi, M.C., 1995. "Effects of infills on the global response o reinforced concrete frames," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 4, pp. 2893-2898.

FEMA Report 308, 1998. "Repair of earthquake damaged concrete and masonry wall buildings."

FEMA Reports 273/274, 1997. "Guidelines and Commentary for the seismic rehabilitation of buildings."

FEMA 156, 1994. "Typical costs for seismic rehabilitation of existing buildings."

FEMA 172, 1992. "NEHRP handbook of techniques for the seismic rehabilitation of existing buildings."

Frangou, M. and Pilakoutas, K., 1995. "Confinement by external lateral tensioning," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2269-2274.

Frosch, R., Li, W., Jirsa, J.O. and Kreger, M.E., 1996(a). "Retrofit of non-ductile moment-resisting frames using precast infill wall panels," Earthquake Spectra, Vol. 12(4), pp. 741-760.

Frosch, R.J., Li, W., Kreger, M.E. and Jirsa, J.O., 1996(b). "Seismic strengthening of a nonductile RC frame using precast infill panels," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 3, Paper No. 1404.

Galano, L. and Gusella, V., 1998. "Reinforcement of masonry walls subjected to seismic loading using steel x-bracing," ASCE Journal of Structural Engineering, Vol. 124(8), pp. 886-895.

GangaRao, Hota V.S. and Vijay, P.V., 1998. "Bending behavior of concrete beams wrapped with carbon fabric," ASCE Journal of Structural Engineering, Vol. 124(1), pp. 3-10.

Gavrilovic, P. and Sendova, V., 1995. "Study for evaluation of a methodology for repair and strengthening of Byzantine churches," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2215-2220.

Georgopoulos, T. and Dritos, S.E., 1995. "A steel jacketing technique for strengthening RC columns," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2275-2280.

Ghobarah, A., Aziz, T.S. and Bidah, A., 1996(a). "Seismic rehabilitation of reinforced concrete beam-column connections," Earthquake Spectra, Vol. 12(4), pp. 761-780.

Ghobarah, A., Aziz, T.S. and Biddah, A., 1996(b). "Rehabilitation of reinforced concrete beam-column joints," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 3, Paper No. 1394.

Giuffre, A., 1989(a). "Principles for the philological restoration of historical buildings in seismic areas," Proceedings International Technical Conference On Structural Conservation of Stone Masonry, Athens.

Giuffre, A., 1989(b). "Philological restoration of historical monuments: the cathedral of Asnt'Angelo dei Lombardi in Irpinia," Proceedings International Technical Conference On Structural Conservation of Stone Masonry, Athens.

Giuffre, A., 1989(c). "Mechanics of historical masonry and strengthening criteria," Proceedings of XV Regional Seminar on Earthquake Engineering, Ravello, Italy.

Gomes, A.M. and Appleton, J., 1998. "Repair and strengthening of reinforced concrete elements under cyclic loading," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 12p.

Gomes, A. and Appleton, J., 1996. "Behaviour of repaired/strengthened concrete columns under alternated cyclic actions," European Earthquake Engineering, Vol. 10(1), pp. 21-28.

Govindan, P., Lakshmipath, M. and Santhakumar, A.R., 1986. "Ductility of infilled frames," Journal of the American Concrete Institute, Vol. 83(4), pp. 567-576.

Griffith, M.C. and Alaia, R., 1997. "Gap Effects on the Seismic Ductility of Brick Infilled Concrete Frames," Proceedings of the 15th Australasian Conference on the Mechanics of Structures and Materials, A.A. Balkema, Rotterdam.

Griffith, M.C., Aiken, I.D. and Kelly, J.M., 1990. "Comparison of earthquake simulator test results with the SEAONC tentative seismic isolation design requirements,' Earthquake Spectra, Vol. 6(2), pp. 403-417.

Hanson, R.D., 1977. "Repair, strengthening and rehabilitation of buildings," NSF Workshop Proceedings.

Hassan, A.F. and Sozen, M.A., 1997. "Seismic vulnerability assessment of low-rise buildings in regions with infrequent earthquakes," Journal of the American Concrete Institute, Vol. 94(1), pp. 31-39.

Hawkins, N.M. and Stanton, J.F., 1982. "Building rehabilitation strategies in Seattle, Washington," Proceedings, US-People's Republic of China Bilateral Workshop on Earthquake Engineering, Vol. 1.

Hayashi, T., Niwa, H. and Fukuhara, M., 1980. "Strengthening methods of the existing reinforced concrete buildings," Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul, Vol. 4, pp. 89-96.

Higashi, Y., Endo, T. and Shimizu, Y., 1984. "Experimental studies on retrofitting of reinforced concrete building frames," Proceedings of the 8th World Conference on Earthquake Engineering, San Francisco, Prentice-Hall, New Jersey, Vol. 1, pp. 477-484.

Higashi, Y. et al, 1980. "Experimental study of strengthening reinforced concrete structures by adding shear wall," Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul, Vol. 7, pp. 173-180.

Higashi, Y., Ohkubo, M. and Fugimata, K., 1977. "Behavior of reinforced concrete columns and frames strengthened by adding precast concrete walls," Proceedings of the 6th World Conference on Earthquake Engineering, New Delhi, Vol. 3, pp. 2505-2510.

Higashi, Y. and Kokusho, S., 1975. "The strengthening methods of existing reinforced concrete buildings," Proceedings of the US-Japan Cooperative Research Program.

Holmes, W.T. and Somers, P. (editors), 1996. "Northridge earthquake of January 17, 1994 reconnaissance report," Earthquake Spectra, Supplement C to Vol. 11, 278p.

Holmes, W.T., 1979. "The rehabilitation of history corner of the Stanford University main quad," Proceedings of the 2nd US National Conference on Earthquake Engineering, EERI, Berkeley, California, pp. 933-942.

Hoshikuma, J. and Unjoh, S., 1997. "Seismic retrofit of existing reinforced concrete columns by steel jacketing," Proceedings of the 2nd Italy-Japan Workshop on Seismic Design of Bridges, 27-28 Feb, Roma.

Hosokawa, Y., 1992. "Post-installed anchor bolts subjected to tension," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5241-5246.

Ifrin, M. and Stoica, N., 1995. "Strengthening of a monolithic reinforced concrete large floor after the earthquake of August 31, 1986, in Bucharest," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2281-2286.

Irimies, M., Cosma, M. and Tokes, A., 1998. "Experimental study on the behavior of strengthened unreinforced masonry shear walls by RC tie-columns," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 5p.

Japan Building Disaster Prevention Association, 1990. "Standard for Seismic Safety Evaluation and Guideline for Retrofitting of existing R/C buildings," Tokyo, Japan Building Disaster Prevention Association (in Japanese).

Jiangquian, X. and Quanshuan, H., 1987. "Model test of brick masonry building strengthened with RC tie-column," Proceedings of the 13th Regional Seminar on Earthquake Engineering, Istanbul, pp. 333-352.

Jirsa, J.O., 1994. "Divergent issues in rehabilitation of existing buildings," Earthquake Spectra, Vol. 10(1), pp. 95-112.

Jirsa, J.O. and Badoux, M., 1990. "Strategies for seismic redesign of buildings." Proceedings of the 4th US National Conference on Earthquake Engineering, EERI, El Cerrito, California, Vol. 3, pp. 343-351.

Jirsa, J.O., 1989. "Behaviour of epoxy-grouted dowels and bolts used for repair or strengthening of R.C. structures," Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Vol. 7, pp. 371-376.

Johnson, M.W. and Smietana, E.A., 1990. "Seismic strengthening of a large food processing facility," Proceedings of the 4th US National Conference on Earthquake Engineering, EERI, El Cerrito, California, Vol. 3, pp. 353-362.

Jokerst, M., 1995. "Retrofitting the PG&E 215/245 Market Street complex with concrete shear walls," Proceedings of the 3rd National Concrete and Masonry Engineering Conference, San Francisco, California, Vol. 2, pp. 797-823.

Juhasova, E., Pezzoli, P., Da Rin, E.M., Sofronie, R. and Zembaty, Z., 1998. "Resistance of brick building model with arches before and after retrofitting," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 12p.

Juhasova, E., 1996. "Slovakia Part of Final Report on PECO 93 Project 930060: Experimental evaluation of technical interventions to reduce seismic vulnerability of old existing buildings," Bratislava: ICA SAS.

Kabayama, K., Araki, H. and Shimazu, T., 1998. "Seismic safety evaluation of building structures and utilization of appropriate methods of retrofitting," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 12p.

Kahn, L.F., Hanson and Hanson, R.D., 1979. "Infilled walls for earthquake strengthening," ASCE Journal of Structural Engineering, Vol. 105(2), pp. 283-296.

Kappos, A.J., Stylianidis, K.C. and Michailidis, C.N., 1998. "Analytical models for brick masonry infilled r/c frames under lateral loading," Journal of Earthquake Engineering, Vol. 2(1), pp. 59-87.

Kariotis, Kesler and Allys, 1978. "Mitigation of seismic hazards in existing unreinforced masonry walls buildings: performance of undesigned and modified elements; evaluation of the modification methods," Agbabian Assoc., 28p.

Kawamata, S. and Ohnuma, M., 1980. "Strengthening effect of eccentric steel braces to existing rc frames," Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul, Vol. 4, pp. 513-520.

Kehoe, B.E., 1996. "Performance of retrofitted URM buildings," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 3, Paper 1417.

Kelly, J.M. and Way, D., 1982. "Seismic rehabilitation of existing buildings using natural rubber bearings," Proceedings, SEAOC Annual Seminar.

Keshtkar, H.E. and Hanson, R.D., 1992. "Rehabilitation of buildings using multiple levels of passive control," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5155-5158.

Klingner, R.E. and Bertero, V.V., 1978. "Earthquake resistance of infilled frames," ASCE Journal of Structural Engineering, Vol. 104(6), pp. 973-989.

Klyachko, M.A., 1995. "Preventative aseismic strengthening of the structures: From problems and approaches to implementation," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2287-2292.

Knoll, F., 1983. "Upgrading seismic resistance of buildings for moderate earthquake risk," Engineering Structures, Vol. 5(2), pp. 133-140.

Korenev, B.G. and Poliakov, V.S., 1978. "Reduction of seismic structural response using the vibration absorber," Proceedings of the 6th European Conference on Earthquake Engineering, Ljubljana, Vol. 2, pp. 159-166.

Koyluoglu, A.M., Yuva, Y., Askar, G.A., Verzeletti, G. and Pinto, A.V., 1995. "Seismic strengthening of RC joints using adhesively bonded steel plates," EC Joint Research Centre Report No. EUR 16276 EN, European Laboratory for Structural Assessment, Ispra, 60p.

Kratzig, W.B. and Meskouris, K., 1998. "Invited Lecture: Assessment of seismic structural vulnerability as a low-cycle fatigue process," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam.

Lagomarsino, S., 1998. "A new methodology for the post-earthquake investigation of ancient churches," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 12p.

La Mendola, L., Papia, M. and Zingone, G., 1995. "Effects of horizontal stiffeners on the seismic response of monumental masonry structures," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2227-2233.

Lemaire, R., 1993. "The engineer and the preservation of architectural heritage," Proceedings, IABSE Symposium, Rome.

Leon, R.T. and Jirsa, J.O., 1986. "Bi-directional loading of R.C. beam-column joints," Earthquake Spectra, Vol. 2(3), pp. 537-564.

Li, C. and Reinhorn, A.M., 1995. "Experimental and analytical investigation of seismic retrofit of structures with supplemental damping: Part II – friction devices," National Center of Earthquake Engineering Research, Report No. NCEER-95-0009, State University of New York at Buffalo, 240p.

Liauw, T.C. and Kwan, A.K.H., 1992. "Experimental study of shear wall and infilled frame on shake-table," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 5, pp. 2659-2663.

Lobo, R.F., Bracci, J.M., Shen, K.L. Reinhorn, A.M. and Soong, T.T., 1993. "Inelastic response of reinforced concrete structures with visco-elastic braces," National Center for Earthquake Engineering Research, Report No. NCEER-93-0006, State University of New York at Buffalo, 112p.

Lundeen, T.R. and Perbix, T.W., 1994. "Seismic retrofit of a historic five story concrete building," Proceedings of the 5th US National Conference on Earthquake Engineering, EERI, Oakland, California, Vol. 3, pp. 541-549.

Luzinda, B., Holmes, W.T., Longstreth, M. and Ken, A., 1997. "Development of procedures to enhance the performance of rehabilitated URM buildings," NIST GCR 07-724-1, US Department of Commerce.

Lybas, 1978. "Methods for seismic strengthening of buildings," Report M249 of the Construction Engineering Research Laboratory.

Lynn, A.C., Moehle, J.P., Mahin, S.A. and Holmes, W.T., 1996. "Seismic evaluation of existing reinforced concrete building columns," Earthquake Spectra, Vol. 12(4), pp. 715-739.

McVay, M., Cook, R.A. and Krishnamurthy, K., 1996. "Pullout simulation of post-installed chemically bonded anchors," ASCE Journal of Structural Engineering, Vol. 122(9), pp. 1016-1024.

Maissen, A., 1995. "Comparison of concrete beams prestressed with carbon fibre reinforced plastic and steel strands," Techtextil-Symposium, Vortrags-Nr.:543, Seite 1-10.

Maldonado, N.G. and Olivencia, L.A., 1992. "Techniques used to repair seismic-resistant masonry walls," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5389-5394.

Mander, J.B., Kim, J.H. and Ligozio, C.A., 1996(a). "Seismic performance of a model reinforced concrete bridge pier before and after retrofit," National Center for Earthquake Engineering Research, Report No. NCEER-96-0009, State University of New York at Buffalo, 208p.

Mander, J.B., Mahmoodzadegan, B., Bhadra, S. and Chen, S.S., 1996(b). "Seismic evaluation of a 30-year-old non-ductile bridge pier and its retrofit," National Center for Earthquake Engineering Research, Report No. NCEER-96-0008, State University of New York at Buffalo, 208p.

Mander, J.B., Nair, B., Wojtkowski, K. and Ma, J., 1993. "An experimental study on the seismic performance of brick-infilled steel frames with and without retrofit," National Center for Earthquake Engineering Research, Report No. NCEER-93-0001, State University of New York at Buffalo.

Mander, J.B., Priestley, M.J.N. and Park, R., 1988. "Theoretical stress-strain model for confined concrete," ASCE Journal of Structural Engineering, Vol. 114(8), pp. 1827-1849.

Manos, G.C., Triamataki, M. and Yasin, B., 1995. "Experimental and numerical simulation of the influence of masonry infills on the seismic response of reinforced concrete framed structures," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 1513-1518.

Manzouri, T., Schuller, M.P., Shing, P.B. and Amadei, B., 1996. "Repair and retrofit of unreinforced masonry structures," Earthquake Spectra, Vol. 12(4), pp. 903-922.

Martinez-Romero, E., 1993. "Experiences on the use of supplementary energy dissipators on building structures," Earthquake Spectra, Vol. 9(3), pp. 581-626.

Masri, A.C. and Goel, S.C., 1996. "Seismic design and testing of an RC slab-column frame strengthened by steel bracing," Earthquake Spectra, Vol. 12(4), pp. 645-666.

Mayes, R.L., Sveinsson, B.I. and Jones, L.R., 1992. "Seismic Isolation and analysis of the Salt Lake City and County Building," The Masonry Society Journal, Vol. 10(2), pp.101-106.

Mehrabi, A.B., Shing, P.B., Schuller, M.P. and Noland, J.L., 1996. "Experimental evaluation of masonry infilled RC frames," ASCE Journal of Structural Engineering, Vol. 122(3), pp. 228-237.

Michailidis, C.N., Stylianidis, K.C. and Kappos, A.J., 1995. "Analytical modelling of masonry infilled R/C frames subjected to seismic loading," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 1519-1524.

Miller, J.P. and Gould, N.C., 1996. "Seismic base isolation study for a Kentucky building," Proceedings of Structures Congress XIV, ASCE, New York, Vol. 1, pp. 389-396.

Moehle, J.P., Nicoletti, J.P. and Lehman, D.E., 1994. "Review of seismic research results on existing buildings: product 3.1 of the Proposition 122 Seismic Retrofit Practices Improvement Program," Report No. SSC 94-03, California Seismic Safety Commission, Sacramento.

Moghaddam, H.A. and Mahmoodi, A., 1995. "Seismic strengthening of masonry infilled structures," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2293-2297.

Mokha, A.S., Amin, N., Constantinou, M.C. and Zayas, V., 1996. "Seismic isolation retrofit of large historic building," ASCE Journal of Structural Engineering, Vol. 122(3), pp. 298-308.

Monti, G. and Nistico, N., 1998. "Design equations for FRP wrapping and steel jacketing of bridge piers," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 9p.

Monti, G. and Spoelstra, M.R., 1997. "Analysis of RC bridge piers externally reinforced with fiber-reinforced plastics," Proceedings of the 2nd Italy-Japan Workshop on Seismic Design of Bridges, 27-28 Feb, Roma.

Mosalam, K.M., White, R.N. and Ayala, G., 1998. "Response of infilled frames using pseudo-dynamic experimentation," Earthquake Engineering and Structural Dynamics, Vol. 27(6), pp. 589-608.

Mullins, P.J. and O'Connor, C., 1987. "Reinforcement of brick walls spanning vertically," ASCE Journal of Structural Engineering, Vol. 113(2), pp. 334-348.

Murray, P.A. and Parker, J.H., 1994. "Seismic retrofit of low-rise non-ductile concrete frame buildings in the central United States," Proceedings of the 5th US National Conference on Earthquake Engineering, EERI, Oakland, California, Vol. 3, pp. 607-616.

Naaseh, S., 1995. "Seismic retrofit of San Francisco City Hall: the role of masonry and concrete," Proceedings of the 3rd National Concrete and Masonry Engineering Conference, San Francisco, California, Vol. 2, pp. 769-795.

Nateghi, A.F., 1995. "Seismic strengthening of 8-story reinforced concrete building," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2235-2240.

Negro, P. and Verzeletti, G., 1996. "Effect of infills on the global behaviour of r/c frames: energy considerations from pseudodynamic tests," Earthquake Engineering and Structural Dynamics, Vol. 25(8), pp. 753-773.

Negro, P. and Taylor, C.A., 1996. "Effect of infills on the global seismic behaviour of R/C frames: results of pseudodynamic and shaking table tests," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 4, Paper No. 2045.

Nielsen, R.J., Schmeckpeper, E., Azari, A. and Crossler, R.W., 1998(a). "Seismic retrofit of precast concrete panels," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 10p.

Nielsen, R.J., Schmeckpeper, E.R. and Crossler, R.W., 1998(b). "Seismic retrofit of precast concrete connections," ATC-29-1 Seminar on Seismic Design, Retrofit, and Performance of Nonstructural Components, Applied Technology Council, Redwood City, California.

Niu, Z., 1982. "Calculating methods of strengthened and repaired brick masonry structures for earthquake resistance," Proceedings, US-People's Republic of China Bilateral Workshop on Earthquake Engineering, Vol. 1.

Norris, T., Saadatmanesh, H. and Ehsani, M.R., 1997. "Shear and flexural strengthening of R/C beams with carbon fiber sheets," ASCE Journal of Structural Engineering, Vol. 123(7), pp. 903-911.

Oh, S.T., Chang, K.C., Lai, M.L. and Nielsen, E.J., 1992. "Seismic response of viscoelastically damped structure under strong earthquake ground motions," Proceedings of the 10th World

Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5163-5168.

Okada, T., Murakami, M., Seki, M. and Ando, A., 1983. "Repair and strengthening of reinforced concrete buildings," Proceedings of the IABSE Symposium, Vol. 46, Vienna, 1983

Olaru, D., 1998. "RC and masonry structures – seismic risk evaluations and RC jacketed strengthening design procedures," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 11p.

Oliveira, C.S. and Ravara, A., 1978. "Methodology for strengthened existing structures built without the required earthquake resistance," Proceedings of the 6th European Conference on Earthquake Engineering, Ljubljana, Vol. 2, pp. 327-334.

Owada, Y., 1992. "Seismic behaviors of beam-column joint of reinforced concrete exterior frame under varying axial load," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 6, pp. 3181-3184.

Ozcebe, G., Sucuoglu, H., Ersoy, U. and Tankut, T., 1998. "Rehabilitation of moderately damaged RC buildings after 1995 Dinar earthquake," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 8p.

Papadopoulos, P.G. and Karayiannis, C.G., 1995. "Nonlinear analysis of infilled plane RC frames," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 2, pp. 1333-1338.

Park, R., 1997. "A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 30(3), pp. 213-226.

Park, R. et al, 1995. "The Hyogo-ken Nanbu earthquake (the great Hanshin earthquake) of 17 January 1995: Report of the NZNSEE reconnaissance team," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 28(1), pp. 1-98.

Peckan, G., Mander, J.B. and Chen, S.S., 1995. "Experimental performance and analytical study of a non-ductile reinforced concrete frame structure retrofitted with elastomeric spring dampers," National Center for Earthquake Engineering Research, Report No. NCEER-95-0010, State University of New York at Buffalo, 160p.

Peter, K. and Badoux, M., 1998. "Seismic evaluation of existing reinforced concrete buildings in Switzerland," Proceedings of the 11th European Conference on Earthquake Engineering, Paris.

Petrini, V., Casolo, S. and Doglioni, F., 1998. "Invited lecture: models for vulnerability analysis of monuments and strengthening criteria," Proceedings of the 11th European Conference on Earthquake Engineering, Paris.

Pilakoutas, K. and Dritsos, S., 1992. "Design of structural repair schemes for RC structures," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5183-5186.

Pincheira, J.A. and Jirsa, J.O., 1995. "Seismic response of RC frames retrofitted with steel braces or walls," ASCE Journal of Structural Engineering, Vol. 121(8), pp. 1225-1235.

Pincheira, J.A. and Jirsa, J.O., 1994. "Seismic response of a strengthened RC frame," Proceedings of the 5th US National Conference on Earthquake Engineering, EERI, Oakland, California, Vol. 3, pp. 749-758.

Pincheira, J.A. and Jirsa, J.O., 1992. "Post-tensioned bracing for seismic retrofit of RC frames," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5199-5204.

Pires, F., Rodrigues, J. and Campos-Costa, A., 1998(a). "Experimental study of the behavior under horizontal actions of repaired masonry infilled RC frames," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 13p.

Pires, F., Rodrigues, J., Bairrao, R. and Liu, X., 1998(b). "Study of the earthquake behavior of masonry infilled reinforced concrete frames. Test of models I1 – I9," Reports 15/98-C3ES to 29/98-C3ES, Lisbon, LNEC.

Pires, F., Campos-Costa, A. and Raposo, S., 1995. "Hysteretic behaviour of R/C frames infilled with brick masonry walls," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 1739-1744.

Pires, F. and Carvalho, E.C., 1992. "The behaviour of infilled reinforced concrete frames under horizontal cyclic loading," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 6, pp. 3419-3422.

Plecnik, J. Cousins, T. and O'Conner, E., 1986. "Strengthening of unreinforced masonry buildings," ASCE Journal of Structural Engineering, Vol. 112(5), pp. 1070-1087.

Plevris, N., Triantafillou, T.C. and Veneziano, D., 1995. "Reliability of RC members strengthened with CFRP," ASCE Journal of Structural Engineering, Vol. 121(7), pp. 1037-1044.

Poole, R.A. and Clendon, J.E., 1992. "N.Z. Parliament buildings: seismic protection by base isolation," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 25(3), pp. 147-160.

Pop, I., Terec, L., Kovacs, L., Mircea, D. and Filip, M., 1998. "Structures in reinforced frames strengthened with concrete panels," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 6p.

Popa, G. and Sofronie, R., 1996. "Romania Part of Final Report on PECO 93 Project 930060: Experimental evaluation of technical interventions to reduce seismic vulnerability of old existing buildings," Bucharest: TUB FLREE.

Prawal, S.P. and Reinhorn, A.M., 1985. "Seismic retrofit of structural masonry using a ferrocement overlay," Proceedings of the Third North American Masonry Conference, Arlington, Texas, pp. 59/2-59/19.

Priestley, M.J.N., 1997. "Displacement-based seismic assessment of reinforced concrete buildings," Journal of Earthquake Engineering, Vol. 1(1), pp. 157-192.

Priestley, M.J.N., Seible, F. and Calvi, G.M., 1996. "Seismic design and retrofit of bridges", John Wiley and Sons, 686p.

Priestley, M.J.N., Seible, F., Xiao, Y. and Verma, R., 1994(a). "Steel jacket retrofitting of reinforced concrete bridge columns for enhanced shear strength – Part 1: theoretical considerations and test design," ACI Structural Journal, Vol. 91(4), pp. 394-405.

Priestley, M.J.N., Seible, F., Xiao, Y. and Verma, R., 1994(b). "Steel jacket retrofitting of reinforced concrete bridge columns for enhanced shear strength – Part 2: test results and comparisons with theory," ACI Structural Journal, Vol. 91(5), pp. 537-551.

Priestley, M.J.N., Seible, F. and Fyfe, E., 1992. "Column seismic retrofit using fibreglass/epoxy jackets," Proceedings of the 1st International Conference on Advanced Composite Materials in Bridges and Structures, pp. 287-298.

PWRI, 1986. "Manual for repair methods of civil engineering structures damaged by earthquakes," Public Works Research Institute, Ministry of Construction, Japan (english version published by the National Center for Earthquake Engineering Research, Buffalo, New York).

Raffaelle, G.S., Gentry, T.R. and Wight, J.K., 1992. "Earthquake loading on R/C beam-columns connections," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 6, pp. 3185-3190.

Rai, D.C. and Wallace, B.J., 1998. "Aluminium shear-links for enhanced seismic resistance," Earthquake Engineering and Structural Dynamics, Vol. 27(4), pp. 315-342.

Rai, D.C. and Goel, S.C., 1996. "Seismic strengthening of unreinforced masonry piers with steel elements," Earthquake Spectra, Vol. 12(4), pp. 845-862.

Ramirez-de-Alba, H., Martinex-Rueda, E. and De la Colina-Martinez, J., 1992. "Behaviour of RC frame models strengthened with walls," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5257-5260.

REEDS, 1998. "Optimization of energy dissipation devices, rolling-systems and hydraulic couplers for reducing seismic risk to structures and industrial facilities: Task 10 – Numerical analysis of structures and components," Report of Task 10 working group, Joint Research Centre, Ispra, Italy.

Reinhorn, A.M., Bracci, J.M. and Mander, J.B., 1993. "Seismic retrofit of gravity load designed reinforced concrete buildings," Proceedings of the 3rd National Earthquake Conference, Central United States Consortium, Memphis, Vol. 2, pp. 245-254.

Ribeiro, 1995. "Repair materials of concrete structures – experimental modeling of an inorganic binder-based mortar," Master's Thesis, Lisbon: IST.

Ritchie, P.A., Thomas, D.A., Lu, L.-W. and Connelly, G.M., 1991. "External reinforcement of concrete beams using fiber reinforced plastics," ACI Structural Journal, Vol. 88(4), pp. 490-500.

Rodriguez, M. and Park, R., 1994. "Seismic load tests of reinforced concrete columns strengthened by jacketing," ACI Structural Journal, Vol. 91(2), pp. 150-159.

Rodriguez, M. and Park, R., 1991. "Repair and strengthening of reinforced concrete buildings for seismic resistance," Earthquake Spectra, Vol. 7(3), pp. 439-459.

Rojahn, C. and Comartin, C.D., 1998. "Criteria and guidance for the evaluation and repair of earthquake damaged concrete and masonry wall buildings," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 13p.

Rutherford and Chekene, 1994. "Provisional commentary for seismic retrofit: Product 1.1 of the Proposition 122 Seismic Retrofit Practices Improvement Program," Report SSC 94-02, California Seismic Safety Commission, Sacramento.

Saadatmanesh, H., Ehsani, M.R. and Jin, L., 1997. "Repair of earthquake-damaged RC columns with FRP wraps," ACI Structural Journal, Vol. 94(2), pp. 206-215.

Saadatmanesh, H., Ehsani, M.R. and Jin, L., 1996. "Seismic strengthening of circular bridge pier models with fibre composites," ACI Structural Journal, Vol. 93(6), pp. 639-647.

Saadatmanesh, H. and Ehsani, M., 1991. "RC beams strengthened with GFRP plates. I: experimental study," ASCE Journal of Structural Engineering, Vol. 117(11), pp. 3417-3433.

San Bernadino County Department of Economic and Community Development, 1987. "Seismic Strengthening, Final Report and Handbooks."

Sanders, D.H., Douglas, B.M. and Cornell, K., 1995. "Bridge seismic retrofitting of reinforced concrete beam-column joints by hinge shifting," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2241-2247.

Scalzi, J.B., 1983. "Strengthening of masonry buildings in the United States," Proceedings of the IABSE Symposium, Vienna, Vol. 46, 1983.

Schneider, S.P., Zagers, B.R. and Abrams, D.P., 1998. "Lateral strength of steel frames with masonry infills having large openings," ASCE Journal of Structural Engineering, Vol. 124(8), pp. 896-904.

Schwegler, G., 1995. "Masonry construction strengthened with fiber composites in seismically endangered zones," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2299-2303.

Scott, A.R. and Deneff, C.I., 1993. "Retrofit of a five-story non-ductile, flat slab concrete frame building with URM infill for operability after a major earthquake," Proceedings of the 3rd National Earthquake Conference, Central United States Consortium, Memphis, Vol. 2, p. 265-274.

Seible, F., Hegemier, G.A., Priestley, M.J.N. and Innamorato, D., 1995. "Developments in bridge column jacketing using advanced composites," Proceedings of the National Seismic Conference on Bridges and Highways, Dec 10-13, San Diego, 15p.

Seible, F., 1995. "Structural rehabilitation with advanced composites," Proceedings of IABSE Symposium, San Francisco, pp. 391-398.

Shahin, T., Osman, A. and Sadek, A.W., 1998. "Seismic upgrading of gravity-designed concrete frames using steel bracing," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 11p.

Sheu, M.S., Liu, P.M., Tu, Y.S. and Chang, Y.F., 1998. "Aseismic retrofitting of school buildings in Taiwan," Proceedings of the 11th European Conference on Earthquake Engineering, Paris.

Sheu, M.S. and Chang, S.F., 1995. "Hysteretic rules of short columns strengthened by wire meshes or hoop plates," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2305-2310.

Shimizu, Y., 1989. "Shear and tensile strength of expansion anchor," Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Vol. 7, pp. 511-516.

Shing, P.B., Lotfi, H.R., Barzegarmehrabi, A. and Brunner, J., 1992. "Finite element analysis of shear resistance of masonry wall panels with and without confining frames," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 5, pp. 2581-2586.

Sofronie, R.A. and Popa, G., 1998. "Confined structures of reinforced masonry," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 11p.

Sofronie, R., 1998. "Innovative method for repair masonry buildings," Proceedings of IABSE Colloquium, Berlin.

Sofronie, R., 1997. "Antiseismic reinforcement of masonry works," Proceedings of International Conference on New Technologies in Structural Engineering, Lisbon: LNEC, pp. 373-380.

Soong, T.T. and Dargush, G.F., 1997. "Passive energy dissipation systems in structural engineering," John Wiley and Sons, 356p.

Structural Engineers Association of Southern California, 1993. "Retrofit of existing tilt-up buildings and in-fill frames: Seminar," SEAOSC, Los Angeles.

Sucuoglu, H. and Erberik, M.A., 1997. "Performance evaluation of a 3-storey unreinforced masonry building during the 1992 Erzincan earthquake," Earthquake Engineering and Structural Dynamics, Vol. 26(3), pp. 319-336.

Sugano, S. and Endo, T., 1983. "Seismic strengthening of reinforced concrete buildings in Japan," Proceedings of Strengthening of Building Structures – Diagnosis and Theory, IABSE Symposium, Venice, pp. 371-378.

Sugano, S., 1982. "An overview of the state-of-the-art in seismic strengthening of existing reinforced concrete buildings in Japan," Proceedings of the Third Seminar of Repair and Retrofit of Structures, SEAOC.

Sugano, S., 1981(a). "Guideline for seismic retrofitting (strengthening, toughening and/or stiffening) design of existing reinforced concrete building," Proceedings of the Second Seminar of Repair and Retrofit of Structures, SEAOC.

Sugano, S., 1981(b). "Seismic strengthening of existing reinforced concrete buildings in Japan," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 14(4), pp. 209-222.

Sugano, S. and Fujimura, M., 1980. "Aseismic strengthening of existing reinforced concrete buildings," Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul, Vol. 4, pp. 449-456.

Sugano, S., 1980. "Aseismic strengthening of existing reinforced concrete buildings," Proceedings of the 1st Seminar on Repair and Retrofit of Structures, US/Japan Cooperative Research Program in Earthquake Engineering on Repair and Retrofit of Structures, Dept. of Civil Engineering, Univ. of Michigan, Ann Arbor, Vol. 1, pp. 13-40.

Syrmakezis, C.A., Giuffre, A., Carocci, C., Spence, R., D'Ayala, D. and Tzenov, L., 1998. "Report of the EAEE working group 7: Preservation of historical structures and historical centers," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 20p.

Syrmakezis, C.A., Giuffre, A. and Yorulmaz, M., 1995. "Seismic aspects of preservation of historical monuments," General Report of Working Group 7 of the European Association on Earthquake Engineering, Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 4, pp. 3099-3128.

Syrmakezis, C.A., Giuffre, A., Wenzel, F. and Yorulmaz, M., 1990. "Seismic aspects of preservation of historical monuments," General Report of Working Group 7 of the European Association on Earthquake Engineering, Proceedings of the 9th European Conference on Earthquake Engineering, Moscow, pp. 303-346.

Syrmakezis, C.A., 1986. "Seismic aspects of preservation of historical monuments, General Report" Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Vol. 7, pp. 157-167.

Tagawa, Y., Aoki, H., Huang, T. and Masuda, H., 1992. "Experimental study of new seismic strengthening method for existing RC structure," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5193-5198.

Taghdi, M., Bruneau, M. and Saatcioglu, M., 1998. "Seismic retrofit of non-ductile concrete and masonry walls by steel-strips bracing," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 11p.

Tanaka, H., Park, R. and McNamee, B., 1985. "Anchorage of transverse reinforcement in rectangular reinforced concrete columns in seismic design," Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 18(2), pp. 165-190.

Tasai, A., 1992. "Effective repair with resin for bond failure of RC members," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5211-5216.

Taucer, F., 1998. Personal communication, EC Joint Research Centre, Structural Mechanics Unit, ELSA, Ispra, Italy.

Tena-Colunga, A. and Vergara, A., 1997. "Comparative study on the seismic retrofit of a mid-rise steel building: steel bracing vs energy dissipation," Earthquake Engineering and Structural Dynamics, Vol. 26(6), pp. 637-655.

Tena-Colunga, A., 1996(a). "Some retrofit options for the seismic upgrading of old low-rise school buildings in Mexico, " Earthquake Spectra, Vol. 12(4), pp. 883-902.

Tena-Colunga, A., Del Valle, E. and Perez-Moreno, D., 1996(b). "Issues on the seismic retrofit of a building near resonant response and structural pounding," Earthquake Spectra, Vol. 12(3), pp. 567-597.

Tena-Colunga, A. and Vergara, A., 1996(c). "Seismic retrofit for a mid-rise building using steel bracing or ADAS energy dissipation devices: a comparative study," Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Pergamon, Elsevier Science Ltd., Disc 1, Paper No. 20.

Teran-Gilmore, A., Bertero, V.V. and Youssef, N.F.G., 1996. "Seismic rehabilitation of infilled non-ductile frame buildings using post-tensioned steel braces," Earthquake Spectra, Vol. 12(4), pp. 863-882.

Teran, A. and Ruiz, J., 1992. "Reinforced concrete jacketing of existing structures," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5107-5113.

Tomazevic, M. and Klemenc, I., 1997. "Seismic behaviour of confined masonry walls," Earthquake Engineering and Structural Dynamics, Vol. 26(10), pp. 1059-1071.

Tomazevic, M., Lutman, M. and Weiss, P., 1995. "The influence of tying the walls with steel ties on the seismic behaviour of historical stone and brick masonry urban buildings," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 2249-2254.

Tomazevic, M. and Apih, V., 1993(a). "The strengthening of stone-masonry walls by injecting the masonry-friendly grouts," European Earthquake Engineering, Vol. 7(1), pp. 10-20.

Tomazevic, M., Lutman, M. and Velechovsky, T., 1993(b). "Aseismic strengthening of old stone-masonry buildings: is the replacement of wooden floors with RC slabs always necessary?," European Earthquake Engineering, Vol. 7(2), pp. 34-46.

Triantafillou, T.C., 1998(a). "Strengthening of structures with advanced FRPs," Progress in Structural Engineering and Materials," Vol. 1(2), pp. 126-134.

Triantafillou, T.C., 1998(b). "Shear strengthening of reinforced concrete beams using epoxy-bonded FRP composites," ACI Structural Journal, Vol. 95(2), pp. 107-115.

Triantafillou, T.C., Deskovic, N. and Deuring, M., 1992. "Strengthening of concrete structures with prestressed fiber reinforced plastic sheets," ACI Structural Journal, Vol. 89(3), pp. 235-244.

UJCEERP, 1981. "Proceedings of the Second seminar on repair and retrofit of structures," US/Japan Cooperative Earthquake Engineering Research Program, Tsukuba, Japan.

UN Economic and Social Council, Committee on Housing, Building and Planing, 1982. "Redesign, repair and strengthening of buildings in seismic regions," Report ECE/HBP/43.

UNDP/UNIDO, 1984. "Volume 5: Repair and strengthening of reinforced concrete, stone and brick-masonry buildings," Building Construction under Seismic Conditions in the Balkan Region, Project Report RER/79/015.

Valiasis, T.N., Stylianidis, K.C. and Penelis, G.G., 1993. "Hysteresis model for weak brick masonry infills in r/c frames under lateral reversals," European Earthquake Engineering, Vol. 7(1), pp. 3-9.

Valiasis, T.N. and Stylianidis, K.C., 1989. "Masonry infilled r/c frames under horizontal loading – experimental results," European Earthquake Engineering, Vol. 3(3), pp. 10-20.

Valluvan, R., Kreger, M.E. and Jirsa, J.O., 1994. "Seismic performance of shear joints in R/C frames retrofitted using infill walls," Proceedings of the 5th US National Conference on Earthquake Engineering, EERI, Oakland, California, Vol. 3, pp. 655-664.

Valluvan, R., Kreger, M.E. and Jirsa, J.O., 1993. "Strengthening of column splices for seismic retrofit on nonductile reinforced concrete frames," Journal of the American Concrete Institute, Vol. 90(4), pp. 432-440.

Valluvan, R., Kreger, M.E. and Jirsa, J.O., 1992. "Strengthening of columns splices in infilled shear walls," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5121-5126.

Via, G., Decanini, L.D., Bertoldi, S.H. and Santini, S., 1995. "Analytical models of infilled frames," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 1533-1538.

Warner, J., 1980. "Repair and retrofit of buildings overview of US experiences – current practice and weaknesses," Proceedings of the First Seminar of Repair and Retrofit of Structures, SEAOC.

Wasti, S.T., Erberik, M.A., Sucuoglu, H. and Kaur, C., 1998. "Studies on strengthening of rural structures damaged in the 1995 Dinar earthquake," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 8p.

Weeks, J., Seible, F., Hegemier, G. and Priestley, M.J.N., 1994. "The US-TCCMAR full-scale 5-story masonry research building test, Part V – repair and retest," UCSD Report No SSRP-94/05.

Wenzel, F., 1989. "The civil engineer in monument preservation continuity and change of tasks," Proceedings International Technical Conference On Structural Conservation of Stone Masonry, Athens.

Whittaker, A.S., Bertero, V.V., Thompson, C.L. and Alonso, L.J., 1991. "Seismic testing of steel plate energy dissipation devices," Earthquake Spectra, Vol. 7(4), pp. 563-604.

Wyllie, L.A. Jr., 1989. "Guidelines for epoxy grouted dowels in seismic strengthening projects," Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Vol. 7, pp. 499-504.

Wyllie, R. and Loring, A., 1983. "Seismic strengthening procedures for existing structures," Proceedings of Strengthening of Building Structures – Diagnosis and Theory, IABSE Symposium, Vienna, Vol. 46.

Wyllie, J. and Loring, A., 1981. "Strengthening existing concrete and masonry buildings for seismic resistance," Proceedings of the Second Seminar of Repair and Retrofit of Structures, SEAOC.

Xiao, Y., Martin, G.R., Yin, Z. and Ma, R., 1995. "Retrofit design of existing reinforced concrete bridge columns using prefabricated composite jacketing," Proceedings National Seismic Conference on Bridges and Highways, Dec 10-13, San Diego, 14p.

Yamamoto, Y. and Umemura, H., 1992. "Analysis of reinforced concrete frames retrofitted with steel brace," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5187-5192.

Yamamoto, T., 1992. "FRP strengthening of RC columns for seismic retrofitting," Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, A.A. Balkema, Rotterdam, Vol. 9, pp. 5205-5210.

Yongsong, G., 1982. "Aseismic evaluation of brick structures," Proceedings of US-People's Republic of China Bilateral Workshop on Earthquake Engineering, Vol. 1.

Yuksel, E., Ilki, A. and Karadogan, H.F., 1998. "Strengthening of reinforced brittle masonry," Proceedings of the 11th European Conference on Earthquake Engineering, Paris, A.A. Balkema, Rotterdam, 8p.

Zarnic, R. and Gostic, S., 1997. "Masonry infilled frames as an effective structural sub-assemblage," Proceedings of the International Workshop on Seismic Design Methodologies for the next generation of codes, Bled, Slovenia, A.A. Balkema, Rotterdam, pp. 335-346.

Zarnic, R., 1995. "Modelling of response of masonry infilled frames," Proceedings of the 10th European Conference on Earthquake Engineering, A.A. Balkema, Rotterdam, Vol. 3, pp. 1481-1486.

Zarnic, R., Tomazevic, M. and Velechovsky, T., 1986(a). "Experimental study of methods for repair and strengthening of masonry infilled reinforced concrete frames," Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Vol. 5, pp. 11.1/41-48.

Zarnic, R. and Tomazevic, M., 1986(b). "Aseismic strengthening of an historical XVII-century urban building – a case study," Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, Vol. 5, pp. 11.2/1-8.

Zarnic, R. and Tomazevic, M., 1984. "The behaviour of masonry infilled reinforced concrete frames subjected to cyclic lateral loading," Proceedings of the 8th World Conference on Earthquake Engineering, San Francisco, Prentice-Hall, New Jersey, Vol. 6, pp. 863-870.

Zezhen, N. and Dinggen, L., 1989. "Seismic performance of strengthened buildings in 1985 Zigong Schuan earthquake," Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Vol. 7, pp. 433-438.

Zhang, R.-H., Soong, T.T. and Mahmoodi, P., 1989. "Seismic response of steel frame structures with added viscoelastic dampers," Earthquake Engineering and Structural Dynamics, Vol. 18(3), pp. 389-396.

Zingone, G., 1993. "Structural preservation of Italy's monuments: restoration of the Zisa Palace, Palermo," Structural Engineering International, Vol. 3(1), pp. 22-25.

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Abstract
The purpose of the present study was to investigate possible seismic retrofit options for use in the seismic upgrade of a reinforced concrete frame building with brick masonry infill walls. The building is typical of a Mediterranean European country (e.g., Greece, Italy, Portugal) and while designed according to the state-of-the-art over 40 years ago, it does not meet the present day seismic design requirements and contains a number of now “well-recognised” seismic design deficiencies and problems. The overall aim of this project was to identify the optimal combination of retrofit options that would enable the building to meet the present-day “life-safety” performance criteria for buildings subject to a design magnitude earthquake. As part of this study, a detailed review of the broader literature in the area of seismic rehabilitation was undertaken in conjunction with a preliminary assessment of the building’s seismic capacity.

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