REDUCING THE COST OF SEISMIC RETROFIT USING ADVANCED STRUCTURAL ASSESSMENT TECHNIQUES – CASE STUDY

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ABSTRACT

It has been more widely recognised by modern societies that a great number of currently operational buildings are at high seismic risk and that there is a growing need for a comprehensive campaign for structural seismic assessment and retrofit. However, since the seismic assessment of buildings has significant socio-economic implications, standard design tools are often not accurate and reliable enough to be confidently used and more refined procedures are needed. This is of critical importance for pre-code unreinforced masonry (URM) structures in seismic regions, which have a very complex non-linear response to lateral loads and are prone to brittle failure.

This paper describes a case-study where the latest assessment code recommendations, performance-based design concepts and advanced nonlinear numerical simulations were combined to perform a comprehensive seismic assessment of a three-storey pre-code URM structure, operating under tight security restrictions, and to propose a retrofit scheme. At first, the seismic assessment was performed with commonly used design procedures and, based on the results, a retrofit solution was proposed. Understanding the inherent conservatism of the standard design methods and the main structural deficiencies, the team advised the client to extend the assignment towards the use of performance-based techniques, together with advanced non-linear numerical simulations. In return, the capacity of the structure was fully explored and an improved cost-efficient retrofit solution was proposed, with a substantial reduction in materials, building works cost and downtime. This led not only to the reduction of the risk of life-loss but also to significant cost savings for the owner, while having a positive effect on the environment by reducing the carbon footprint of the designed intervention.

Keywords: Unreinforced masonry; Seismic assessment; Non-linear analysis; Seismic retrofit; Cost reduction

1. INTRODUCTION

Recent statistics show that the majority of structural collapses and the associated deaths in past earthquakes were due to the inadequate performance of unreinforced masonry (URM) structures. This, together with the national strategies for the preservation of the architectural heritage, focused the majority of the research on the seismic behaviour of masonry buildings on the seismic assessment of existing ones, rather than on innovative construction techniques for masonry structures. It also raised authorities’ and communities’ awareness of the need for realistic seismic risk assessment and efficient and applicable rehabilitation strategies.

A lot of research has been carried out on the seismic response of URM structures and numerous studies have been published on a variety of subjects in this field. From experimental research (Beyer & Dazio, 2012; Fehling, et al., 2007; Magenes, et al., 2008; Anthoine, et al., 1995; Benedetti, et al., 1998), through analytical and numerical modelling strategies (Priestley, et al., 2007; Cattari, 2014; Lagomarsino, et al., 2013; Lagomarsino & Cattari, 2014; Tomažević, 1978; Magenes & Della Fontana, 1998; Magenes,
2000; Roca, et al., 2005; Doherty, et al., 2002; Lam, et al., 2003) (D’Ayala & Speranza, 2003), to the development of retrofit techniques (Modena, et al., 2009; Manzouri, et al., 1996; Modena, 1994). In practice, however, with the time and budget constraints of a consultancy, the active codes and standards remain the basic source of guidelines and recommendations. Engineers have always relied upon them for simpler and easy-to-implement procedures, yet, with the tendency to give over conservative, often impractical, solutions.

1.1 State-of-practice for Seismic Assessment of URM

The codes that are commonly accepted to provide a rational framework for assessment of URM buildings are the documents by the Federal Emergency Management Agency (FEMA), by the American Society of Civil Engineering (ASCE) and the Eurocodes by the European Committee for Standardisation. For the case study described herein, a variety of codes were reviewed and those found most applicable were: FEMA 273 (ATC FEMA 273, 1997) and the corresponding commentary FEMA 274 (ATC FEMA 274, 1997); FEMA 306 (ATC FEMA 306, 1998) which guidelines were most helpful in the determination of the component failure types, global failure mechanisms and damage of infills; FEMA 307 (ATC FEMA 307, 1998) which describes with experimental results the hysteretic behaviour of URM walls subjected to in-plane demand, their force-displacement response and explains in comprehensible way the interconnection between different local failure modes; FEMA 356 (ASCE FEMA 356, 2000) based on which the damage limit states were defined; FEMA 547 (NIST FEMA 547, 2006) and FEMA 774 (ATC FEMA 774, 2009) for basic rehabilitation concepts and risk reduction; ASCE/SEI 41-13 (ASCE/SEI 41-13, 2014) for the definition of the seismic demand, the target building performance levels and the execution of Tier 1 Screening and Tier 2 Deficiency-based evaluation. The Eurocodes EN 1996-1-1 (cen EN 1996-1-1, 2005) and EN 1996-3 (cen EN 1996-3, 2006) were also reviewed for comparison and verification of the material properties and methods for the calculation of the component capacity. Although, the information in the FEMA documents overlaps to an extent, each of them provides some additional clarity, which helped to build a better understanding of the seismic response of URM structures, of critical aspects and the application of the overall methodology.

As for the out-of-plane failure of walls, contemporary seismic codes for design provide dimensioning and detailing guidelines to prevent this type of failure under severe seismic loads. A lot of the existing URM buildings, however, were constructed before the implementation of current seismic design requirements and it is expected that they lack such detailing restrictions, which makes them highly vulnerable to out-of-plane failure. In general, the question about the out-of-plane vibration response of masonry walls is still one of the most complex and cursorily addressed in the seismic assessment codes. Recent research has shown that out-of-plane response of masonry walls under seismic excitation is governed by the displacement response, rather than strength. This concept was originally proposed by Priestley (1985) and endorsed later on by Doherty et al. (2002) and Lam et al. (2003).

1.2 Non-linear analysis methods for URM structures

Non-linear methods of analysis have started to be more often implemented in practice and their advantages and necessity have been more frequently recognised by engineers. This is even more so when it comes to the performance-based assessment of masonry structures and their asymmetric response. At lateral displacements close to their ultimate capacity, strength distribution and not elastic stiffness is what matters, and this cannot be captured with linear elastic analysis procedures. If fully non-linear models are too challenging and sometimes even not appropriate for common engineering practice, the lumped-plasticity models and microelement discretisation methods need lower computational power and use concepts better understood by practitioners but with accuracy and integrity comparable to advanced FE models or experiments.

A lot of studies on the suitability of non-linear equivalent frame models for static and dynamic analyses in the assessment and design of URM were found in the technical literature (Magenes & Della Fontana, 1998; Magenes, et al., 2000; Magenes, 2000; Roca, et al., 2005; Lagomarsino, et al., 2013; Cattari, 2014; Lagomarsino & Cattari, 2014). It is important to note, that most of them consider only the in-plane
response of masonry walls (global failure mechanisms), by assuming that the out-of-plane collapse (local failure mechanism) is prevented.

This paper describes a case study of an URM building which seismic response and ultimate capacity was assessed through a successful combination of the force-displacement capacity and damage limits prescribed by the FEMA set of codes, the frame equivalent non-linear model with concentrated plasticity and static push-over analysis. It also demonstrates, what differences could be expected between an elastic model and a non-linear model used in the seismic assessment and how the non-linear model can lead to a significant optimisation of the retrofit solutions.

1.3 Overview of the project

For over 3 years now, our team has been involved in a large programme for reviewing the seismic vulnerability of structures with the aim of reducing the risk of life loss and the damaging impact on the communities. Part of this programme, is the Ankara project, which includes multiple buildings in the capital of Turkey.

Like others in the programme, it commenced with carrying out Rapid Visual Screenings (Tier 1) for all buildings based on FEMA154, the outcome of which was a numerical score for preliminary evaluation of the level of risk to the loss of life or injury when a damaging earthquake occurs and whether a more detailed structural assessment will be required. This first phase included not only on-site visits but also intrusive works, where found necessary. At the end of this phase, detailed reports were produced on the basic structural deficiencies identified as a precursor for the consequent detailed structural assessment.

The office building discussed in detail in this paper is one of these buildings. It is a pre-code three-storey URM structure constructed in mid-1940s. It is situated in a region with mid-to-high seismicity and still fully operational under tight security, providing services of high importance. Long-term disruption of its functionality due to structural damage could have a significant social impact. The Rapid Visual Screening scored the building much less than the cut-off score 2.0, which denoted a very high risk of structural failure in an earthquake. The challenges which our team faced where not only how to assess most realistically the structural capacity but also to propose a retrofit scheme which is not only cost-efficient but also quick to implement with minimum downtime and with regard to the local construction practice.

Due to the confidentiality requirements of the client, not all of the information will be explicitly written within this paper, and as such some information is not going to be available.

2. AS-BUILT STRUCTURE

The as-built structure is a pre-code three-storey URM building with reinforced concrete (RC) floor diaphragms. Vertical and plan irregularities are the most obvious structural deficiencies. Each storey is with a different storey height, the first, above the ground level, being higher than the others by a metre. The thickness of the masonry shear walls also varies with height. In plan, the building is L-shaped.

Figure 1 shows the layout of the storeys obtained from archive architectural drawing and on-site visits. The plan view also shows that there are a lot of opening for windows and doors. Internal partitions are generally constructed of brick masonry of less than 20cm thickness, with the exception of some stud partition walls. The slabs are lightly reinforced concrete diaphragms and the connection between them and the masonry walls below them could not be confirmed. The roof comprises a pitched timber structure. The stairs are RC with no movement joint to allow for inter-storey movement. The foundation system consists of RC strip footings.

Masonry cores were taken and tested for average and lower material strength.
Due to the age of the building, no formal structural drawings were available. In addition, the current usage of the building required high security provisions and therefore restricted access for the surveyors and minimum interventions. In this regard, the structural information that could be relied upon was rather limited and a lot of assumptions had to be made in the detailed assessment process.

The main structural deficiencies identified after the Rapid Visual Screening are: vertical and horizontal structural irregularity, low and most likely insufficient in-plane shear capacity of the masonry walls, possible soft storey mechanism at first level and uncertain force-transfer capacity of the floor-wall connection. Vulnerability of non-structural components was also identified but it is outside the scope of the current paper.

The building is situated in Ankara, which, according to the European Seismic Hazard Map and the current seismic design code of Turkey, is in a region with moderate seismicity. Figure 2 shows a map of Western Turkey with ground acceleration in main rock with an exceedance probability of 10% in a 50-year period. The soil is classified as Soil Class D according to FEMA 154.

3. INITIAL ELASTIC ANALYSIS AND PRELIMINARY RETROFIT SCHEME

Since the first assessment phase found various deficiencies in the building, an Outline Retrofit design was necessary for the client to be able to understand the approximate cost, disruption and work required to retrofit the building to ensure Life-Safety (LS) of the occupants. At this stage, very limited time was available to perform complex analysis and it was considered more important to provide the client with enough information, even though preliminary, to discuss the need for more sophisticated analysis. With results obtained from simplified elastic FE models with equivalent horizontal forces, we were able to communicate to the client that by investing a bit more time in a justifiable non-linear model, the seismic response of the building will be much better analysed, the uncertainties and conservativism will be reduced and the retrofit solution could be substantially optimised.

3.1 FE elastic analysis

The preliminary elastic FE model was built in SAP2000. It consisted mainly of shell elements for the walls and the slabs, with exception for a few RC columns, which were modelled with beam elements. All elements were modelled with their actual section size. In Figure 3 different section types are presented with different colours. All RC elements were assigned elastic materials with elastic moduli $E$
which correspond to the reported compression concrete strength. The URM walls were modelled with two different elastic materials. The walls which were characterised with significant vertical irregularities and lots of openings, or in general were expected to be much weaker compared to the majority of the walls, were considered not to contribute to the overall structural stiffness and were assigned a material with a negligible elastic modulus. This applied also to the thin partition walls. The rest of the URM walls were modelled with a material with a 50% reduction of the elastic modulus that corresponds to the reported average compressive strength of the masonry. The shell elements with same section type but different material properties are presented in Figure 3 with different shades of one colour. The strength of the timber roof structure was ignored and the staircase was not modelled. The model was fixed at the base and was fully elastic. The masses due to self-weight, permanent and variable actions were modelled via uniformly distributed loads over the concrete slab shell elements. Equivalent mass-proportional seismic horizontal forces, which corresponded to the assumed spectrum, were applied. Static linear analysis was performed.

Although very simplified and conservative, the preliminary elastic analysis helped us identify some important features of the seismic response of the building. For example, the results showed that in global Y direction the building is much weaker than in global X direction and that torsional modes have a significant contribution to the overall structural behaviour. What is more, the comparison of the shear forces in the walls obtained from the elastic analysis and the rocking $V_r$ and bed-joint sliding $V_{bjs}$ capacity of URM according to ASCE 41-13 showed that more than half of the piers are expected to fail in an earthquake with the considered intensity, even if an upper bound of the wall capacity is assumed.

### 3.2 Preliminary retrofit solution

The preliminary analysis and calculations led to a retrofit strategy which could improve both, strength and ductility. Thus, several retrofit schemes were considered and their pros and cons discussed with the client. The proposed solutions included techniques with fibre-reinforced polymers, post-tension tendons, filling in the openings of selected walls, construction of new RC shear walls, steel bracing and selective weakening by splitting the L-shaped building into two rectangular structures through a seismic gap. Together with the objective of improvement to the structural response, the selection of the retrofit scheme was governed also by requirements for building functionality and appearance. Due to the restricted access and tight security, it was difficult to relocate staff for a long period, construction works and downtime on the 2nd floor had to be minimised as much as possible, and modifications of external facades, especially the front one, had to be avoided not to affect the historic appearance of the building. Eventually, the most suitable solution was found to be the installation of embedded steel braces along selected walls, which will form a new system with lateral-load resisting capacity in support of the existing shear walls. What appealed most to the client was that this retrofit solution involved dry and relatively short construction process with minimum disturbance of the functionality of the building and was based on construction techniques common for the local construction practice.

Figure 4 presents schematically a view of the chosen retrofit design. It was also clearly explained to the client that from a practical point of view and the rehabilitation objectives which corresponded to Life-Safety performance level, cracking and limited repairable damage to the walls will not be fully avoided.

The steel braces consist of vertical, horizontal and diagonal elements installed at certain locations, so as to form continuous vertical diaphragms with no disruption of the functionality of the building. The intent was to both strengthen the existing continuous walls and fix vertical irregularities. The braces were designed with UPN section and steel square tubes and were intended to be embedded in the masonry when placed alongside it. In this way, the interior will undergo minor changes, since the newly installed braces will be fully hidden behind the finishing. The diagonals were intended to carry both tension and compression. The horizontal steel elements will be installed on the top and bottom sides of the RC slabs and will be anchored to them to secure continuous load path of the horizontal forces down the vertical braces. The position of the new steel braces was selected such as to improve the global structural response to seismic loading, to increase structural capacity, to minimise the effect of torsion and to minimise functional disruption.
The preliminary design of this retrofit solution was based on the built simplified elastic model. The steel braces were added to the model with elastic frame elements. Two cases were analysed: one with the stiffness properties of the masonry walls as already assigned, and another case for which all masonry walls with insufficient shear strength capacity were ignored by assigning to them a material with negligible stiffness. Based on these very conservative assumptions, the steel sections were designed according to EN 1993-1-1:2005 and the layout of the braces modified where necessary. The approximate cost of the retrofit was calculated and discussed with the client. At this stage, the out-of-plane capacity of the URM walls was only checked in terms of design parameters and requirements defined in the design provisions. With these preliminary results at hand, we were able to communicate to the client how beneficial to the project would it be to extend the design phase with additional more detailed and sophisticated numerical models and analysis procedures.

4. NON-LINEAR PUSH-OVER ANALYSIS

In the next stage of the project, the conservativism and the uncertainties of the preliminary analysis were significantly reduced by introducing non-linear numerical models and performance-based retrofit design. The target performance level was Life-Safety in accordance with ASCE 41-13 – “Major cracking. Noticeable in-plane offsets of masonry and minor out-of-plane offsets. Transient drift sufficient to cause non-structural damage. Noticeable permanent drift.” In other words, moderate damage should be expected for the defined seismic intensity but human life is not in danger and the repair is feasible with the involvement of manageable resources.

4.1 Capacity of the URM wall components

The non-linear force-displacement response of the URM piers and spandrels and their damage limit states were determined based on the recommendations of FEMA 306, FEMA 307, FEMA 356 and ASCE 41-13. An automated procedure was developed, which identifies the predominant behaviour mode of a certain masonry component or the sequence of activated failure modes and calculates the corresponding backbone force-displacement curve.

FEMA 306 considers a variety of failure modes of different masonry components and prescribes empirical formulae for the calculation of their capacities and force-displacement response. For the purpose of illustration, Figure 7-9 and Figure 7-14 from FEMA 306 are repeated here in Figure 5. According to the hierarchy of strength, the behaviour modes are classified in four groups: Solid Wall behaviour mode, when the weakest link is the foundation interface where rocking or sliding is likely to
occur before any other spandrel or pier damage; Weak Pier, for which the damage is concentrated in the piers (rocking, toe crushing, bed-joint sliding, diagonal tension); Weak Spandrel, for which damage in the spandrels occurs before any damage in the piers; and Weak Joints, which is characterised by diagonal cracks in the panel zone between the piers and the spandrels. The FEMA manual also provides some ideas on how to determine whether a pier rocking mechanism will develop along several storeys or within a storey between the spandrels (Figure 5 b). The evaluation procedures in FEMA 306 are supported by the explanatory information in FEMA 307 based on experimental data from cyclic in-plane tests on URM walls. It focuses on the relationship between toe crushing and bed-joint sliding and how to predict which of them will govern the response of the masonry component.

As far as out-of-plane behaviour of walls and piers is considered, FEMA 306 and ASCE 41-13 prescribe permissible wall aspect ratios which should be met. The URM wall components were checked and almost all of them, with the exception for a few thin partitions, were found to meet the ratio limits. Since, it is now fully recognised and confirmed by recent experiments that the out-of-plane displacement capacity of the walls is crucial for their out-of-plane response, a design procedure proposed by Priestley et al. (2007) was also implemented in this regard. It calculates the out-of-plane capacity of the walls, assuming P-Δ dominated response and rigid body motion deformation profile. It also takes into account the dynamic filtering effect of the building and the theoretical amplification due to the wall response. The checks performed following this procedure, considering the variability of the input data, also showed that out-of-plane failure of the masonry walls is not expected to be the governing failure mechanism.

After a critical review and compilation of the data from the referenced sources, the following steps were taken to calculate the capacity and the non-linear response of the masonry components.

1) Materials - the material properties of the masonry walls were updated from the preliminary analysis. The elastic and shear moduli were calculated as prescribed by FEMA 356, Chapter 7. The expected compressive strength was obtained by multiplying the minimum one with \( f'_{me} = 1.3 f_m \) and the expected elastic modulus was calculated as \( E_{me} = 550 f'_{me} \). The elastic shear modulus of the masonry was assumed to be \( G_{me} = 0.4 E_{me} \). All material properties were checked for consistency with the guidelines in EN 1996-1-1;

2) Geometry of the masonry components and axial load – the effective height of the piers and the effective width of the spandrels were determined, following recommendations by FEMA 356 for differential displacements and deformation pattern of the wall components. The vertical
axial forces in the piers were obtained from the elastic 3D FE model described above but this time with no reduction of the elastic modulus among the masonry walls. Upper and lower bound of the axial force were estimated – the former corresponded to self-weight plus minimum permanent loads and maximum upward axial force due to global structural overturning, evaluated from the preliminary analysis, and the latter corresponded to self-weight plus maximum permanent loads;

3) In-plane capacity for individual behaviour modes – the capacity of the different behaviour modes of each component was calculated with the empirical formulae in FEMA 306 and FEMA 307. The following failure mechanisms were considered: rocking $V_r$, bed-joint sliding with bond and friction $V_{bjs1}$ and with friction only $V_{bjs2}$, diagonal tension $V_{td}$ and toe crushing $V_{tc}$. Axial compression capacity of the piers was also checked and for all of them it significantly exceeded the expected demand;

4) Predominant behaviour mode of piers – an automated procedure was developed which determines the predominant behaviour mode of the piers or the sequence of them from the individual capacities already calculated and the pier aspect ratio $1.25 < L/h_{eff} \leq 1.25$, where $L$ is the length of the wall/pier and $h_{eff}$ is its effective height. An example is given in Table 1. It again follows the recommendations by FEMA 306 and FEMA 307.

Table 1 Determination of the predominant behaviour mode of piers and their capacity

<table>
<thead>
<tr>
<th>Pier</th>
<th>$c$ [%]</th>
<th>$d$ [%]</th>
<th>$e$ [%]</th>
<th>$HO$ [%]</th>
<th>$LS$ [%]</th>
<th>$CP$ [%]</th>
<th>$c$ [%]</th>
<th>$d$ [%]</th>
<th>$e$ [%]</th>
<th>$HO$ [%]</th>
<th>$LS$ [%]</th>
<th>$CP$ [%]</th>
<th>Failure mode</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>ground Wy1</td>
<td>0.6</td>
<td>0.3</td>
<td>0.50</td>
<td>0.1</td>
<td>0.22</td>
<td>0.3</td>
<td>0.6</td>
<td>0.4</td>
<td>0.8</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>$URM2A$</td>
<td>wall-pier rocking</td>
</tr>
<tr>
<td>1st Wy1</td>
<td>0.6</td>
<td>0.54</td>
<td>1.0</td>
<td>0.1</td>
<td>0.41</td>
<td>0.54</td>
<td>0.6</td>
<td>0.4</td>
<td>0.8</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>$URM2A$</td>
<td>wall-pier rocking</td>
</tr>
<tr>
<td>2nd Wy1</td>
<td>0.6</td>
<td>0.43</td>
<td>0.86</td>
<td>0.1</td>
<td>0.32</td>
<td>0.43</td>
<td>0.6</td>
<td>0.4</td>
<td>0.8</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>$URM2A$</td>
<td>wall-pier rocking</td>
</tr>
<tr>
<td>ground Wy2</td>
<td>0.6</td>
<td>0.09</td>
<td>0.18</td>
<td>0.1</td>
<td>0.07</td>
<td>0.09</td>
<td>0.6</td>
<td>0.4</td>
<td>0.8</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>$UJR2B$</td>
<td>bed-joint sliding</td>
</tr>
<tr>
<td>1st Wy2</td>
<td>0.6</td>
<td>0.4</td>
<td>0.8</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>0.6</td>
<td>0.4</td>
<td>0.8</td>
<td>0.1</td>
<td>0.3</td>
<td>0.4</td>
<td>$URM2A$</td>
<td>wall-pier rocking</td>
</tr>
<tr>
<td>2nd Wy2</td>
<td>0.6</td>
<td>0.32</td>
<td>0.66</td>
<td>0.1</td>
<td>0.24</td>
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<td>0.4</td>
<td>0.8</td>
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<td>0.3</td>
<td>0.4</td>
<td>$UJR1H$</td>
<td>wall-pier rocking</td>
</tr>
</tbody>
</table>

5) Simplified force-displacement relationship and acceptance criteria – ASCE 41-13 proposed updates to the Masonry Provisions with respect to FEMA 306 and FEMA 356. These updates were first reviewed with respect to the non-linear analysis modelling parameters of URM, the generalised force-deformation relationships and the acceptance criteria to determine to what extent they would influence the results from the non-linear numerical analysis. In general, the latest standard acknowledges the fact that experimental data has shown somewhat larger displacement capacity of the walls past the initial cracking. This has partly affected the defined acceptance criteria. Other differences are that the residual strength ratio factor $c$ is not a fixed number but it is explicitly calculated as $V_{bjs2}/V_{bjs1}$ for bed-joint sliding and $V_t/V_r$ for rocking and that the slope of the generalised relationship after the elastic response is negative, to take into account the influence of $P-\Delta$ effects. These updates were found to have negligible influence on the numerical representation of the response of the components with predominant rocking response mode and a perceptible effect on those with bed-joint sliding failure mechanism. At the same time, a trial with a smaller FE model showed that the simulation of strength degradation in a lumped plasticity FE model with SAP2000 was associated with numerical problems, the complication of which could hardly be justified within the objectives of the project. The redistribution of forces when the plastic hinges start to yield could be efficiently captured with a plateau in the force-displacement relationship after the first crack. With this in mind, the generalised force-deformation relationship from FEMA 356 was adopted, together with the acceptance criteria specified in the same document.

4.2 Description of the FE model

The elastic model already built for the preliminary analysis of the as-built structure was modified, to include the non-linear response of the masonry components. Figure 6 shows a general view and Figure 7 illustrates how the piers were modelled. Similar to the preliminary analysis, the walls with negligible strength were assigned negligible stiffness. This time, the shells that modelled the piers were replaced by frame elements with hinges. The frame elements were assigned elastic material, equivalent pier
section sizes and were released for the out-of-plane resistance. Shear plastic hinges were defined with properties for each pier following the procedure described above. The properties of each pier were input into the analytical procedure for the definition of the predominant behaviour mode, which in turn, produced the force-displacement relationships to be assigned to the hinge properties. First, the pier geometry was identified, following recommendations by FEMA 356 and FEMA 306 for effective pier height. Second, a lower bound of the permanent axial load on each pier was added, following the static analysis of the preliminary elastic model. After that, the developed automated procedure calculated the capacity of each behaviour mode and, based on their comparison, constructed the most probable force-displacement curve. The final step was to incorporate acceptance criteria onto this curve. The nodes of the shell elements at the levels where the frames (piers) are connected, were constraint to account for the actual widths of the piers. All other shell elements remained as modelled in the preliminary analysis, together with the RC columns.

Figure 6 View of the 3D non-linear FE model

Figure 7 Modelling of the piers

Non-linear static push-over analysis was carried out to estimate the behaviour of the structure subjected to lateral loading. The push-over analyses with mass-proportional force profile were run after the application of all permanent gravity loads. Since the structure is not symmetric and so is its response to lateral loads, eight different analysis were performed: in X direction only, in Y direction only, in X and Y direction simultaneously with 30% reduction of the intensity in one of the directions, and four other
combinations with the reverse load vectors. In this way we could capture the weakest and the least ductile structural response.

4.3 Non-linear structural response

The response of some of the URM walls from the push-over analyses of the as-built structure are presented in Figure 8. The analyses showed that the weakest storey is the last one and most of the damage is concentrated there. This is explained by the fact that the thickness of the walls on the second floor is considerably less compared to the lower storeys. In the global X direction, the structure was much stronger and showed almost 90% of the capacity needed to meet the expected seismic demand. In the global Y direction, however, LS performance levels were reached at demand of less than 20% of the expected one.

4.4 Optimisation of the retrofit scheme

After the non-linear response of the as-built structure was analysed, the steel braces, previously designed for structural strengthening, were added to the model. Compared to the preliminary analysis of the retrofit solution, this time no masonry walls were further excluded from the model, nor their capacity ignored. As expected, the non-linear analysis showed that the preliminary retrofit solution was rather conservative, the as-built structure had more capacity to be exploit and the design of the steel braces can be optimised. An example is given in Figure 9. The masonry piers have not reached LS performance levels and the steel sections are with less than 50% utilisation.

In order to optimise the retrofit solution, the sections and the configuration of the steel braces, a lot of iterations were performed. As a result, two braces in the longitudinal direction of the building were found to be redundant. An additional brace was introduced on the west façade in order to minimise the
rotational response of the building, the effect of which was more pronounced in the non-linear models. This new brace led also to the reduction of a few interior braces on the second floor and the steel sections in general were reduced. An example of the effect of the optimisation is shown in Figure 10.

As for the foundation system, the provided information was insufficient to perform a trustworthy assessment of its capacity at this stage of the project. One thing is certain, the newly design steel braces will introduce additional forces, for which the existing foundation need to be checked. However, the layout of the optimised retrofit solution was performed also with consideration for reduction of the effect of the additional loads, especially when it comes to upwards forces. In the next stage of the project, when the detailed design of the steel braces will be performed, the existing foundation will be thoroughly assessed, it will be properly incorporated in the non-linear model and the retrofit solution may be modified if needed.

Preliminary retrofit solution

Optimised retrofit solution

Figure 10 Optimisation of the retrofit design

5. ECONOMICAL AND SUSTAINABLE RETROFIT SOLUTION

By combining advanced non-linear analysis with code-compliant procedures, an improved cost-efficient retrofit design was proposed with a substantial reduction in materials and building works cost. The amount of steel needed to construct the strengthening bracing system was reduced by more than 50% compared to the preliminary solution, and the building works were estimated to cost 40% less. The new retrofit strategy was not only less expensive, but also more sustainable. The reduction in the materials and the construction efforts will have a favourable impact on the environment, which can be measured with the carbon footprint. Calculations showed that the improved retrofit solution is associated with 49% less carbon emissions with respect to the preliminary retrofit scheme and more than 85% less with
respect to a newly built structure of a similar type. This is illustrated in Figure 11.

Since, the advanced analysis of the seismic response of the structure incorporated damage-based criteria, the client was also advised on how to assess the seismic performance level for which the retrofit system was designed with respect to the damage risk and eventual post-earthquake repair. In other words, reduction in the initial interventions and their cost may result in the increase of the post-event repair and replacement cost. When combined with the target performance, a minimum of the expected total cost of the long-term maintenance of the building was targeted.

Figure 11 Comparison of the carbon emissions associated with the newly-built identical structure, the preliminary retrofit solution and the optimised retrofit solution

6. CONCLUSIONS

The paper describes a case study that demonstrates the benefits to the client and the engineer which advanced structural analysis can bring to a project. Although, advanced engineering procedures are often claimed to be too complicated to be practical, they can substantially improve the design when wisely used and, in some cases, like the seismic assessment of URM structures, they are the most reliable analysis tool.

The case study described is a pre-code URM three-storey structure in a region with moderate seismicity, which, as part of a larger project, had to be assessed and adequately strengthened. In the first stage of the project, commonly used simplified procedures were adopted and elastic numerical model, associated with a lot of conservativism was used. In the second stage, however, the full capacity of the URM walls was exploited and their non-linear response was incorporated in performance-based analysis. As a result, the initially proposed retrofit solution was substantially optimised, the cost of the intervention and the expected downtime was significantly reduced, together with the associated carbon footprint. Consideration was also given to the fact that the reduction in the retrofit intervention cost, leads to an increase in the post-earthquake repair and replacement cost and the target performance level should be set as to lead to the minimum total cost in the long term.

7. ACKNOWLEDGMENTS

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8. REFERENCES


