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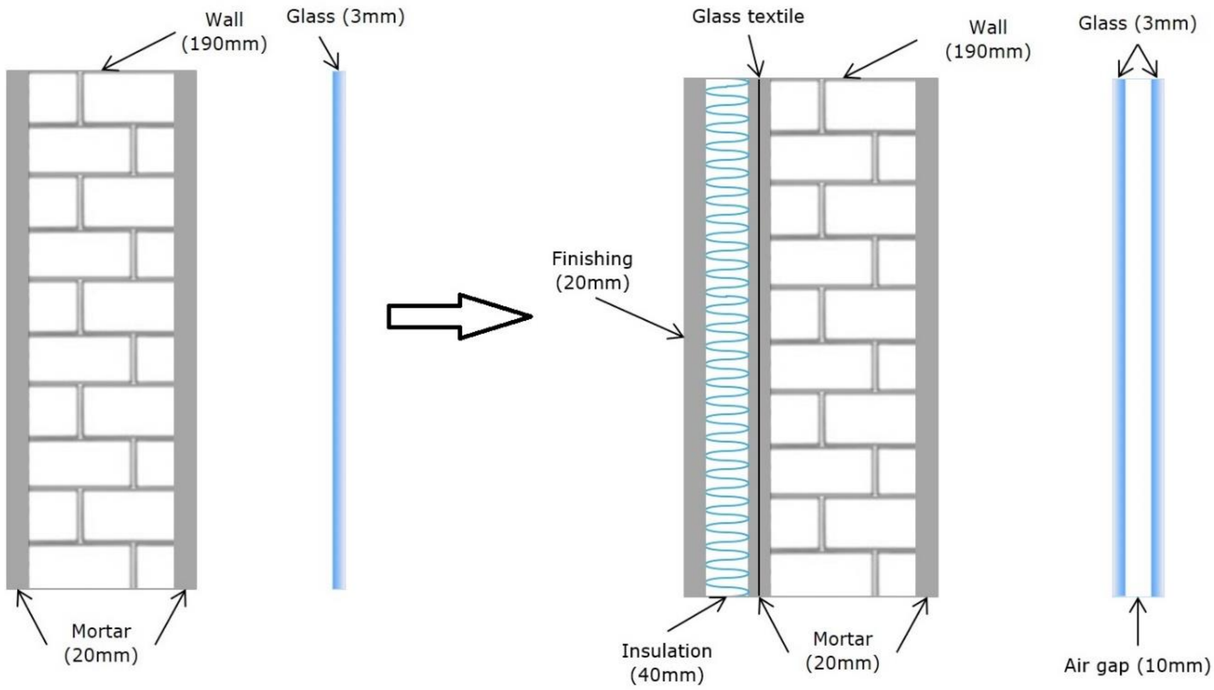
Combined seismic and energy upgrading of existing buildings using advanced materials

Case studies on Reinforced Concrete Buildings in South Europe

Gkournelos, D. P., Bournas, D. A., Triantafillou, T. C.



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Abstract

Of the current EU building stock, 80% was built before the 90's, while 40% are pre-60's and a considerable amount being even older and classified as cultural patrimony, thus requiring preservation techniques if we want to maintain this cultural heritage for our future generations. Upgrading the existing EU buildings and the cultural heritage ones is becoming increasingly important due to: (1) their poor seismic performance during recent earthquakes (i.e. Italy, Greece) that have resulted in significant economic losses, severe injuries and loss of human lives; and (2) their low energy performance which increases significantly their energy consumption. However replacing a big part of the existing buildings is prohibitively expensive or not allowed for historical heritage buildings and would have a significant societal and environmental impact, their lifetime extension requires considering both seismic and energy retrofitting.

The Exploratory research project *iRESIST+* explores a novel concept, by applying a hybrid structural-plus-energy retrofitting solution which combines inorganic textile-based composites with thermal insulation systems for building envelopes. In this report, the *iRESIST+* concept is examined through a number of case studies conducted on model buildings designed according to outdated regulations. Specifically, seismic and thermal analyses were conducted prior to and after the application of selected retrofitting schemes in order to quantify the positive effect that retrofitting could provide to RC buildings both in terms of their structural and energy performance. Advanced materials, namely the so-called Textile Reinforced Mortar (TRM) was used for providing seismic retrofitting by means of TRM jacketing of the masonry infills around the RC frames. Moreover, following the application of the TRM jackets, thermal insulation materials were simultaneously provided to the RC building envelope, exploiting the fresh mortar used to bind the TRM jackets. In addition to the externally applied insulation material, all the fenestration elements (window and doors) were replaced with new high energy efficiency ones. Afterwards, an economic measure, namely the *Expected Annual Loss* (EAL) was used to evaluate the efficiency of each retrofitting method, but also to assess whether the combined seismic and energy retrofitting is economically feasible. From the results of this preliminary study, it was concluded that the selected seismic retrofitting technique can indeed enhance significantly the structural behaviour of an existing RC building and lower its EAL related to earthquake risks. Finally, it was found that the combined seismic and energy upgrading is economically more efficient than a sole energy or seismic retrofitting scenarios for seismic areas of south Europe.

1 Introduction

Of the current EU building stock, 80% was built before the 90's, while 40% are pre-60's and a considerable amount being even older and classified as cultural patrimony, thus requiring preservation techniques for maintaining this cultural heritage for our future generations. Upgrading the existing EU buildings and the cultural heritage ones is becoming increasingly important due to: (1) their poor seismic performance during recent earthquakes in south Europe that have resulted in significant economic losses, severe injuries and loss of human lives; and (2) their low energy performance which increases significantly their energy consumption (buildings are responsible for 40% of EU energy consumption).

Since replacing existing buildings is prohibitively expensive or not allowed for historical heritage buildings and would have a significant societal and environmental impact, their lifetime extension requires considering both seismic and energy retrofitting. It is noted that the annual cost of repair and maintenance of existing European building stock is estimated to be about 50% of the total construction budget, currently standing at more than €300 billion (estimated to be €342bn according to the European Construction Industry Federation-FIEC key figures 2018, EU28). To achieve cost effectiveness, the exploratory research project iRESIST+ is being developing a novel approach, by proposing a hybrid structural-plus-energy retrofitting solution which combines inorganic textile-based composites with thermal insulation systems for reinforced concrete and masonry building envelopes.

This technical report presents a preliminary study for the two most seismic member states, namely Italy and Greece, countries very rich also in cultural heritage buildings. More than half of these building have been built before 1980, specifically 52% for the case of Greece and 70% for Italy. This implies that their design followed codes and techniques that have now changed considerably in terms of seismic requirements and were actually built before the introduction of demanding thermal standards. Consequently, both the seismic and the energy performance of these buildings is inferior to those built according to the current design standards (i.e. the Eurocodes, EPBD).

The problem of seismic deficiency is more important for countries with higher seismic activity (i.e. Greece, Italy, Portugal), whereas the building energy performance is actually critical for every EU country. Both, however, lead via different routes to the same outcome, which is high economic losses, either realized through human casualties and heavy damage in the infrastructure or through high energy consumptions and thus large amounts of money spent to satisfy the increased energy needs for heating and cooling. Especially for the energy sector, according to the European Commission database, buildings are responsible for 40% of energy consumption and 36% of CO_2 emissions in the EU. Whereas new buildings generally need fewer than three to five litres of heating oil per square meter per year, older buildings consume about 25 litres on average, with some requiring up to 60 litres per square meter per year.

This introductory chapter describes initially a number of the most common deficiencies that are observed in the existing EU buildings and affect either their structural or energy performance. Then the state-of-the-art retrofitting techniques aiming at enhancing both those aspects are presented, whereas finally the iRESIST+ integrated seismic and energy retrofitting approach is examined in a series of case studies of typical EU building configurations.

1.1 Deficiencies in existing buildings

1.1.1 Structural

A number of the typical deficiencies observed in old buildings and affect their structural seismic performance is listed in the following sections.

1.1.1.1 Obsolete codes and design methods

Design standards have not been the same since the first day they were introduced, but instead they have evolved during the years along with the evolution of the science and the accumulated experience. Especially when it comes to earthquake loads, the changes that can be observed in the active codes of every era are so essential that buildings can even be classified into different strength categories according to their year of construction.

The first seismic provisions were introduced in the 1920's, following some major earthquakes in the US and Japan and were based upon a simple simulation of the ground motion with static lateral loads. In Greece, the first national-wide code was implemented in 1959, which was afterwards further enriched in 1984 and 1993 and then in 1999 the EAK-2000 (Greek Seismic Code) was issued with a few additions in 2004. At the same time, the European Commission had started already from the 1970's the Eurocode project in order to provide standardisation in the construction sector in the European countries.

However, since the majority of the buildings is older than most of the contemporary regulations, it is clear that their seismic resistance is also much less compared to newer structures. This is due to the fact that the seismic actions considered in old buildings are less than 50% compared to the ones accounted for nowadays, the ideas of regularity in plan were missing, the computational models used for the calculation of the internal forces were simplistic and highly inaccurate and no special measures (detailing provisions, capacity design principles etc.) to provide adequate ductility for the structure at hand were applied.

The obvious consequence of the above is the under-designing of the bearing elements of a building. Some common deficiencies that can be explained because of that are given below:

— The use of very small column sections in old structures, reinforced with very little amount of longitudinal and transverse reinforcement. Their reduced strength can easily be exhausted during an even low-intensity earthquake.

— The inadequate anchorage of both the longitudinal and transverse reinforcement leading to premature brittle failure modes.

— The over-designing of beams compared to the columns they are supported to, hence the lack of capacity design, can lead to soft-storey collapse mechanisms.

— The lack of capacity design in shear within the elements can lead to brittle failure mechanisms thus compromising the ductility of the structure.

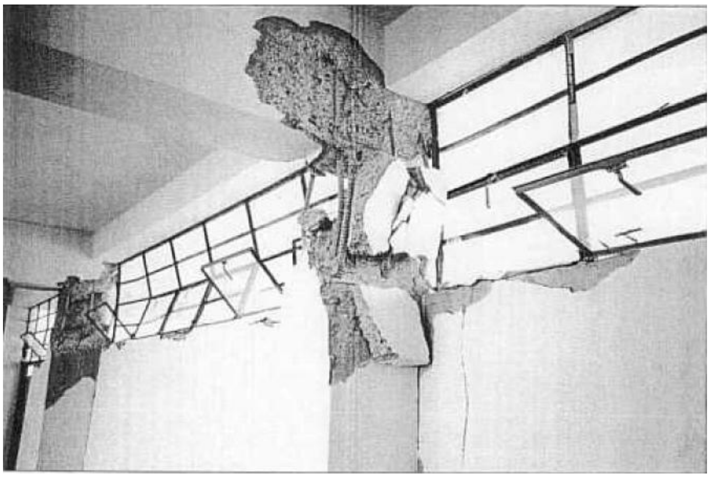
— The absence of properly detailed shear-walls able to provide adequate strength and ductility to the structure.

All the above contribute to the fact that old structures have roughly half or even less earthquake resistance compared to the new ones.

1.1.1.2 Short columns

One very common problem commonly found in old structures, is the existence of short columns. A short column effect occurs when a column is prevented from deforming laterally along a percentage of its height, leaving only a part of it unrestrained. As a result, the unrestrained part is much stiffer compared to a full height column, thus attracts higher forces than the ones designed against and eventually fails in shear in a brittle way (see Figure 1).

Figure 1. Short column failure



Source: Earthquake Engineering Research Institute]

A short column effect can occur when there is a skylight running along the whole length of a bay over the infill wall or when there is an opening in the infill wall close to a column. In old structures, where the columns have small cross-sections in most cases, the surrounding walls completely change the global response by providing a different lateral load bearing system, since their stiffness is higher than that of the columns. The above, combined with the fact that the walls were omitted from the computational models leads to a totally different response than the one that the building was initially designed for. Therefore, the short column effects were never accounted for and the elements affected from them, not protected against them.

The short column effect can be thought of as the worst side-effect the infill walls can cause on a structure. In the past, the existence of strong infill walls has prevented in many cases structures from collapsing, providing one last defence barrier against earthquakes. However, when short columns are formed because of the walls, then they provide an easily achievable collapse mechanism rather than saving the structure.

1.1.1.3 Soft storey mechanism

Another common configuration that can be found in old structures is that of supporting the superstructure on pilotis. That way, large spaces for parking are available while a sense of floating and lightness in the architecture itself is achieved. On the other hand, the existence of pilotis however, leads to non-uniformity in height, since the ground floor has much lower strength and stiffness compared to the above ones.

This fact, during an earthquake leads to the concentration of the damage in the ground floor, while the higher ones remain practically intact. Therefore, what has been observed many times in past earthquakes, is the complete disappearance of the ground floor in buildings with this configuration (see Figure 2).

This configuration is still used today in Greece when it comes to the typical 5 to 6 storey-high residential buildings. However, newer buildings have stiff shear walls which provide most of their lateral stiffness along with larger heavily reinforced columns. Consequently, in this case, the infill walls are considerable weaker than the reinforced concrete frame, therefore the uniformity in height is not compromised.

The case studies in this report address this problem and try to provide a simple way to solve it without extreme interventions.

Figure 2. The soft storey failure mechanism

Source: http://inderc.blogspot.gr/2012/04/seismic-regulations-versus-modern.html.

1.1.1.4 Influence of stairways

Stairways can also, in many cases, lead to undesirable failure modes. This happens mainly due to the fact that they form a stiff truss-like mechanism, able to resist lateral loads, that attracts high internal axial forces during an earthquake. However, they too were rarely (if ever) taken into account during the analysis of old structures. As a matter of fact, the forces that they carry during an earthquake can cause severe damage to them or to the elements they are connected to.

More specifically, shear sliding can take place at the construction interfaces of the flight, concrete crushing can occur to the connection of the landing beam and the flight or even total separation of the flight due to the cyclic tension and compression. Furthermore, it is very common to observe shear cracking at the landing beam, local damage to the floor slab due to locally high stresses and of course the formation of short columns with all their negative effects as have been discussed earlier. Figure 3 contains altogether the most common failures that can be observed in staircases during earthquakes.

Since, they comprise a stiff lateral load bearing system, stairways also move towards them the centre of stiffness of each floor. Hence, when they are located away from the building centre, thus away from the floors' centre of mass, torsional effects are also introduced in the structure leading to internal forces that again have not been accounted for. To minimize this problem in new structures, it is advised to position the stairways near the floors' centre of mass. Moreover, if possible, it should be considered to decouple the stairway from the rest of the building so as to get more reliable results.

In existing structures however, the problem is obviously much larger since an existing situation with its inherent problems has to be dealt with. Therefore, it is highly advised to consider the stairways in the computational model during the analysis of the structure at hand, in order to find the best way to minimize the risk that they can bring about.

Figure 3. Common earthquake induced staircase damages

Source: ascelibrary.org

1.1.2 Energy

Energy efficiency regulations are not something new, as their first appearance can be dated back to the late 1950s and the early 1960s in Scandinavian countries mainly due to their cold weather. For many other countries, the development of energy efficiency requirements was a result of the 1970s oil supply crisis and they were further increased after the Kyoto Protocol aiming to reduce CO_2 emissions (Laustsen 2008). Today, a number of CEN and ISO standards exist, which are incorporated in many countries' energy efficiency regulations.

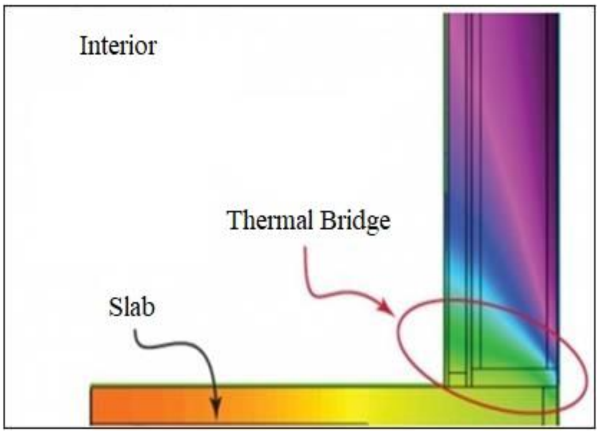
However, either due to the late adoption of energy efficiency standards from many countries, or because of the construction of structures prior to their coming to force, there are many buildings that have been constructed without having the energy efficiency ideas in mind. In Greece for example, the Code for Energy Efficiency of Buildings (called KENAK), only came into force in 2010, thus there is a great percentage of energy deficient buildings.

This section contains many common deficiencies and their respective problems that can be observed in old, existing buildings built without any special energy efficiency measures.

1.1.2.1 Lack of insulation

Perhaps, the most common problem in old structures is the complete or partial absence of insulating material in the building shell. According to ELSTAT (Greek Statistics Agency), 45.6% of the residencies in Greece do not have any kind of insulation at all (Daskalaki et al. 2016). When this is the case, the thermal resistance of the structure is extremely low, thus the indoor temperature closely follows the outdoor one. Consequently, large amounts of energy have to be spent in order to regulate the indoor air temperature as the thermal losses are very significant.

Figure 4. Heat losses in a thermal bridge



Source: http://www.luminosityengtech.com/joomla/index.php/applications/building-inspection/insulation-and-thermal-bridges.

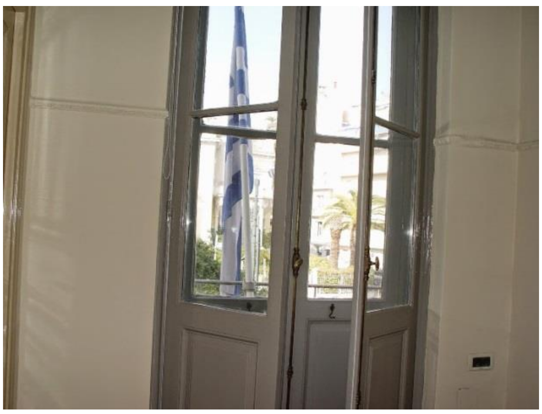
What is also observed in older structures, which do have some insulation, is the extended existence of thermal bridges. As a result, the overall thermal resistance of the object is greatly reduced, thus compromising the thermal envelope of the structure (see Figure 4).

1.1.2.2 Inefficient fenestration

Another source of heat losses in existing buildings is the fenestration surfaces. Naturally, windows and doors have a lower thermal resistance than the surrounding walls, so they provide an easier path for the heat to escape or infiltrate. Today, high energy performance double pane windows with high quality aluminium frames are used in most buildings, so that their resistance is as high as possible and the heat transfer minimal.

In older structures however, it is very common to see single pane fenestration surfaces with wood or steel frames. Apart from having a very low thermal and radiation resistance, they also have significant losses due to air penetration, since they do not seal completely when closed (see Figure 5). The above result again in great losses through the doors and windows and of course in larger energy needs to regulate the indoor temperature and humidity.

Figure 5. Single pane balcony door



Source: http://monoseis-teakk.blogspot.gr/.

1.1.2.3 Inefficient mechanical equipment

Apart from the passive means to achieve thermal efficiency in a building, there are also the active ones, which have mainly to do with the mechanical equipment that can in that direction. Here too, as expected, older buildings have a variety of problems to show, because of their inefficient and old equipment.

First of all, in most older buildings, central furnaces or boilers are used to heat up water, which is then distributed throughout the building, typically by water circulating through pipes and thus providing the necessary heating energy. These systems fuelled by heating oil or natural gas, are very old and therefore have a very low efficiency compared to today's alternative heating systems. As a matter of fact, excessive amounts of fuel are wasted just to overcome the systems' losses rather than heating the actual internal spaces.

On the other hand, when it comes to satisfying the cooling needs, individual air conditioning units are most commonly used, if any at all. Depending on the time of their installation – since in most cases, this happened quite a few years after the building construction – these units can also be old, poorly maintained and inefficient compared to the contemporary ones (see Figure 6).

Figure 6. Old air conditioning unit

Source: http://monoseis-teakk.blogspot.gr/.

Apart from heating and cooling, domestic hot water needs in old buildings are met in most cases through boilers, either electricity or fossil fuel based. Again, these systems are commonly inefficient, thus adding more to the already high energy needs of old buildings. Solar panels, that can cover these needs with zero consumption, are rarely found in old structures, unless some kind of retrofitting has been applied.

1.2 Retrofitting solutions

1.2.1 Structural

After the understanding of the structural deficiencies in existing buildings by the academic and general civil engineering community, a number of different techniques were and still are being developed aiming at the upgrading of existing structures. These methods are either targeted at upgrading the performance of existing elements such as beams and columns in terms of strength, stiffness or ductility, or at creating a totally new lateral load resisting system such as new reinforced concrete walls, steel braces or upgrading the existing infill walls so as to behave as a reliable resisting system. Moreover, in special cases, techniques that involve base isolation or dampers can be used, although their application in common residential structures is rather unlikely and therefore will not be examined hereafter.

It is also worth mentioning, that in those cases were the upgrading of a structure is proven to be completely uneconomic, a total demolition and reconstruction might be preferred. After all, it is common sense, that a newly built structure will always be more reliable than a retrofitted one, no matter how well-designed and applied is the retrofitting scheme.

Since this report is concentrated on retrofitting techniques, the following paragraphs will provide the reader with some of the most common seismic upgrading techniques that can be employed on old reinforced concrete structures.

1.2.1.1 Upgrading of individual elements

The simplest idea of retrofitting is none other than the upgrading of individual elements, which for the case of reinforced concrete structures that we examine, are the beams, columns and shear walls of a structure. These techniques are most effective when the building at hand has already a considerable lateral resistance and phenomena such as brittle failures need to be avoided rather than when a significant strength upgrade is necessary.

1.2.1.1.1 Reinforced concrete jacketing

Figure 7. Reinforced concrete jacketing of a column and its supported beams

Source: https://www.indiamart.com/drdconstruction/rehabilitation-or-structural-repairing-work.html.

One of the first ideas that were developed in the field of structural retrofitting, was the jacketing of an existing element with an exterior layer of reinforced concrete. Through the proper detailing of the jacket, almost all the properties of an element can be affected, like its bending, shear and axial strength, its stiffness and even its ductility by the additional confinement that the jacket can provide. Figure 7 shows an example where reinforced concrete jacketing has been applied on a column and the two beams that it supports.

The application of concrete is done by shotcreting, so as expected, the disruption of the occupants can be rather significant. However, it is a simple technique that has been proven to work satisfactorily in the field, as it has been used in many applications over the past years.

1.2.1.1.2 Externally applied steel reinforcement

Instead of using reinforced concrete externally, exactly the same effects can be achieved, if steel plates are bolted on the outer surfaces of the elements (see Figure 8). The obvious drawback of this method is the corrosion of the externally applied reinforcement, unless special measures are taken.

Figure 8. Retrofitting in RC members with steel plates

Source: ascelibrary.org.

1.2.1.1.3 Fibre reinforced polymer-based solutions

Another similar method to enhance the performance of individual elements, is by their upgrading using fibre reinforced polymers or FRPs in short. As with RC shotcreting, FRP-based solutions can be employed to affect most engineering properties of a member. The following list contains the most common uses of FRPs in RC structural retrofitting.

- Increase of the bending resistance in RC beams by applying/gluing FRP strips or sheets on the faces of the element to act as external tensile reinforcement. To achieve a better anchorage, near surface mounted techniques (NSM) can be used, which include however more laborious interventions. In the case of seismic loads, the usefulness of this method is rather reduced because of the inherent difficulty to extend the FRP reinforcement beyond the beam-column joints where the moments are maximum. That is why this method is most commonly used in the upgrading of beams against gravity loads.
- Increase of the shear resistance in beams and columns by wrapping them with FRP sheets. That way, it can easily be guaranteed that no shear failure will occur in any element and therefore the ductility of the structure will not be compromised.
- Wrapping of RC columns in order to achieve a greater degree of confinement. This results in higher axial strength and most importantly considerably higher ductility without affecting at all the element's bending strength and stiffness (see Figure 9).

It is worth mentioning that FRPs are not free of disadvantages. Because of their nature, FRPs behave linearly elastic – so no fully ductile behaviour can be achieved when they are used as tensile reinforcement – and the epoxy resins they contain exhibit significant strength loss when they are subjected to high temperatures, therefore it is necessary to protect them against fire. Furthermore, their application requires skilled and trained technicians and of course their price can be quite high.

However, they comprise a highly reliable, thoroughly tested and implemented in practice strengthening technique that minimally affects the dimensions of the retrofitted elements and whose application disrupts much less the occupants of the structure. In general, when it comes to shear strengthening or providing additional confinement in columns for ductility reasons, FRPs are most probably the most efficient way of achieving those goals.

Figure 9. FRP jacketing of a column

Source: http://www.valentinecorp.com/structural-strengthening/.

1.2.1.1.4 Textile reinforced mortar-based solutions

Figure 10. TRM jacketing of a column to be experimentally tested

Source: http://www.strulab.civil.upatras.gr/resources/photo-gallery.

The last method of upgrading existing RC members, that will be presented, is the one which uses textile reinforced mortar, or TRMs in short, to achieve the same effects as with that of the FRP-based solutions. Their philosophy is exactly the same and the only two differences are that they use textiles instead of sheets as their load-bearing mechanism and mortars instead of epoxy resins as their binding material. The resulting material has the same behaviour with FRPs but slightly reduced strength and stiffness and relatively higher dimensions. However, the cost of TRMs is considerably lower than that of FRPs mainly due to the high cost of the epoxy resins and their resistance to high temperatures (Tetta and Bournas 2016, Raoof and Bournas 2017a, b) extremely increased because their binding material is an inorganic mortar rather than the organic epoxy resins (Triantafillou et al. 2006, Koutas et al. 2019).

TRMs can be used to enhance the shear strength of RC elements and provide confinement in columns just like FRPs. For what concerns seismic retrofitting of RC columns, TRM jackets have the same effectiveness with FRP jackets (Bournas et al. 2007, 2009). There have been many experiments in the past proving the above (see Figure 10), but the application of TRMs in engineering practice is much less extended than that of FRPs.

1.2.1.2 Addition of new lateral load resisting systems

The above-mentioned techniques are efficient and economic as long as no extreme strength upgrade is needed. When this is the case however, more "aggressive" techniques have to be employed that completely change the lateral load resisting system rather than just upgrading it. The next paragraphs explain the most common methods of this kind.

1.2.1.2.1 Reinforce concrete infilling

One efficient way to greatly increase a RC structure's resistance against lateral loads, is to select specific bays and infill them with reinforced concrete, thus converting the initial frames to a wall-like configuration. The selection of the bays to be transformed must be done very carefully, taking into account the serviceability restrictions of the building (e.g. obligatory existence of openings) as well as structural engineering ones (e.g. the floor shear centre should not move away from the centre of mass).

Figure 11. Building with RC infilled bay tested at the ELSA lab in JRC

Source: Poljanŝek et al. (2014).

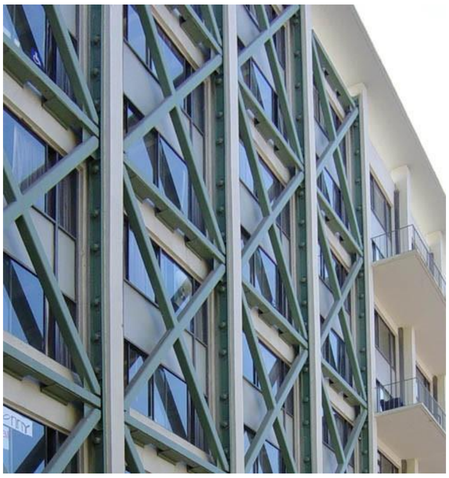
Obviously, the implementation of this method, requires specific measures to be taken at the interface of the frame with the new wall to ensure optimal interaction and trained personnel to perform these tasks. However, this technique has been proven to be very effective and economic when it comes to multi-storey RC buildings, especially those with soft storeys. This was also thoroughly examined within the SERFIN project, which took place at the Joint Research Centre of the European Commission in Ispra and included the testing of a full-scale 3-storey RC building (see Figure 11). According to the findings of that experiment, this method can increase up to five times the resistance to earthquake loads (Poljanŝek et al. 2014).

1.2.1.2.2 External steel bracing

Instead of using reinforced concrete to create wall-like elements in a selected bay, a steel bracing system can be used to create a vertical truss along that same bay (see Figure 12) and achieve the same result. Depending on the dimensioning of that bracing system, the seismic resistance of the building can be increased several times.

Again, special measures need to be taken in order to ensure the optimal interaction between the new and the old elements so that the transfer of the internal forces can occur in a reliable way. This technique though, has been successfully used in many cases in the past and has shown satisfactory behaviour.

Figure 12. Building retrofitted with external steel braces



Source: https://en.wikipedia.org/wiki/Seismic_retrofit#/media/File:ExteiorShearTruss.jpg.

1.2.1.2.3 Strengthening of masonry-infilled RC frames with TRM Jackets

The last retrofitting method to present here, and also the one that was used for the needs of this report is the strengthening of Masonry-Infilled RC Frames with TRM. This method was developed and tested by Koutas et al. (2015a,b) and Koutas and Bournas 2019. It takes advantage of the already existing infill walls that exist in buildings and provides a way to enhance their behaviour and make them more reliable by applying layers of TRM on their faces, thus creating a wall-like lateral load resisting mechanism at a considerably lower cost.

After a set of experiments were completed (see Figure 13), it was found that the proposed method can increase the lateral strength, the stiffness, the energy dissipation and the deformation capacity of the retrofitted structure. Moreover, it is very important to note that this technique prevents the falling of debris from the wall when crushing occurs and also contributes – although the exact amount of how much is yet to be experimentally tested – to the out of plane stability of the wall. Both these side effects are extremely essential for the security of people.

The retrofitting technique comprises of the following steps as presented by Koutas et al. (2015a):

1. Local destruction of the infill walls at the interfaces with the columns and shear strengthening of the latter along their whole height. This has to be done in order to avoid shear failure at the columns upon the crushing of the wall in its corners and TRM or FRP jackets can be used for that matter. It is important to note here, that during the experiment of the retrofitted frame, only the column corners had been retrofitted in shear, something that did not prevent the shear failure but only moved it at the unretrofitted part.

2. Reconstruction of the wall to bring it back to its initial condition.

3. Application of the first layer of TRM layer on the face of masonry infills. Because of the limited width of the TRM textile, around three patches will have to be applied starting from the bottom and accounting for an overlap of around 300 mm.

4. Application of textile anchors on the top face of the bottom slab and the bottom face of the top one at the interfaces with the TRM layer in order to enhance the anchorage of the latter to the RC frame.
5. Application of the second layer if needed and if accessible, wrapping of the overhanging textile parts around the column corners.

Figure 13. Retrofitted model tested at the University of Patras

Source: Koutas et al. (2015a).

Apart from the rest, this report could also be thought of as an applicability and feasibility check of the above technique in almost real-life scenarios. After all, since it is so new, it has first to be verified within the safe zone of case studies before its starting to be implemented by engineers of the practice.

1.2.2 Energy

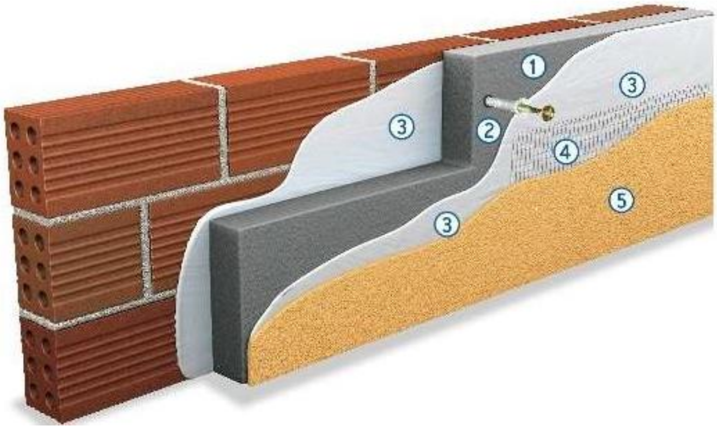
As with seismic, energy upgrading of existing structures is progressively becoming more and more necessary. In Europe for example, the European Commission has already issued the energy strategy targets for 2020, 2030 and 2050 which among others dictate a radical cut in the greenhouse gas emissions and an increase in the share of renewable energy consumption. In order to achieve these goals, it is obvious that actions need to be taken also in the field of energy consumption of buildings.

A building's energy needs can be reduced by enhancing its outer shell, thus increasing its ability to maintain a given temperature and/or by operating on its mechanical equipment either with the form of upgrading or the complete replacement, so as to use less energy for heating, cooling, lighting and hot water. In the next parts, the most common upgrading methods aiming at the energy upgrade of an existing structure will be presented.

1.2.2.1 Upgrading of the thermal shell

The most logical way perhaps to upgrade thermally an existing structure, without insulation, is to intervene on its shell, meaning its external walls and fenestration surfaces. When it comes to RC frame structures, the trend in EU south countries was and still is to build the infill walls with hollow bricks and afterwards apply layers of plastering on the outermost surfaces. In older buildings, it is common to have 20 mm walls without any insulating material; therefore, it would seem natural to apply some kind of external thermal insulation. This could be done very easily by attaching, for example, expanded polystyrene (or any other insulating material) on the outside face of the wall and then protect with a thin finishing layer of lime mortar (see Figure 14).

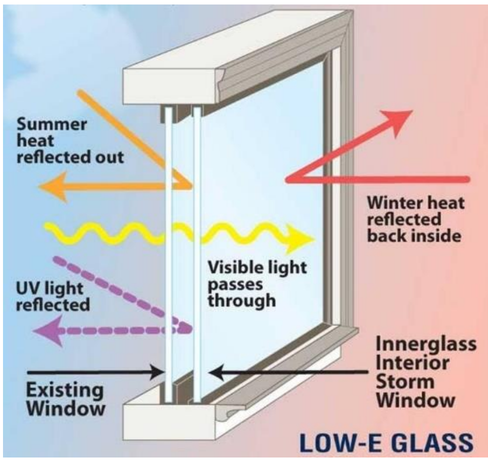
Figure 14. Application of insulation on an existing wall externally



Source: http://www.davas.gr/default.aspx?Index=0&Id=102&LangId=1.

At the same time, in old buildings it is imperative to completely replace the fenestration surfaces with newer aluminium frame, highly efficient windows and doors (see Figure 15). That way, the thermal losses will be greatly minimized as well as the energy needs for heating and cooling.

Figure 15. High-performance, double-pane window



Source: http://www.stormwindows.com/why-interior-storm-windows/.

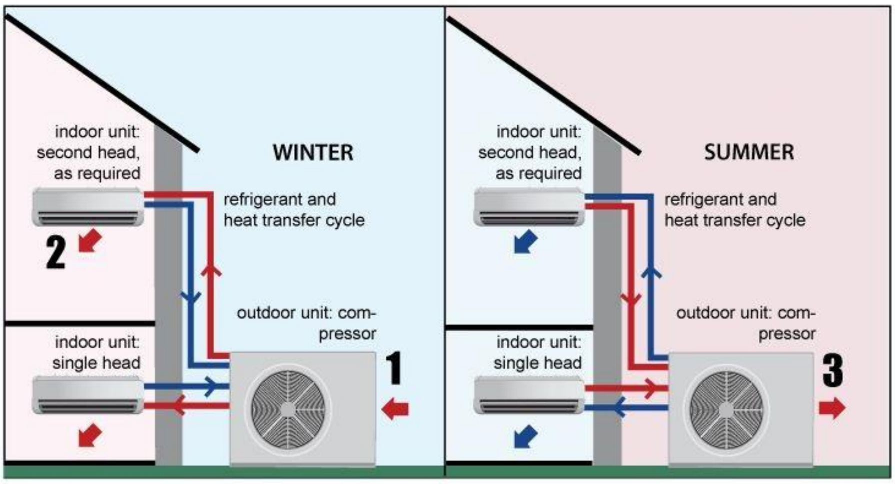
Of course, these kinds of retrofitting techniques need a relatively high initial investment. In this direction, the idea of state loans with close to zero interest rates or European funded programmes provide energy retrofitting alternatives for the owners. After all, such investments are always guaranteed to return the money spent and of course help the countries achieve the targets set by the European Commission.

The case studies presented in this report, assume that the energy upgrading scheme includes only measures for the enhancement of the structure's thermal shell and no interventions whatsoever on its HVAC systems. This was done mainly for the sake of simplicity and in order to avoid coupling many parameters of different nature.

1.2.2.2 Replacement of the mechanical equipment

Another typical way for enhancing the energy performance of old building is by replacing its mechanical equipment, which in most cases is very old, poorly maintained and inefficient. Nowadays, highly efficient heat pumps combined with centralized systems or individual units can satisfy a building's energy needs at low consumptions since their coefficient of performance is normally between 300% to 400%, meaning they produce 3 to 4 times more heating/cooling energy than the energy they consume (see Figure 16).

Figure 16. The working cycles of a heat-pump



Source: http://ryanfaas.com/heat-pump-installers-kent-can-market-online/.

Solar panels can also be used to provide hot water or even produce electricity to cover some of the residence's needs (see Figure 17). Fuelled by the sun's radiation, they provide a totally green way of energy production, thus helping in the direction of reducing the green-house gas emissions.

Figure 17. Solar panels installation on a residence's roof

Source: https://www.zenenergy.com.au/residential/solar-solutions/.

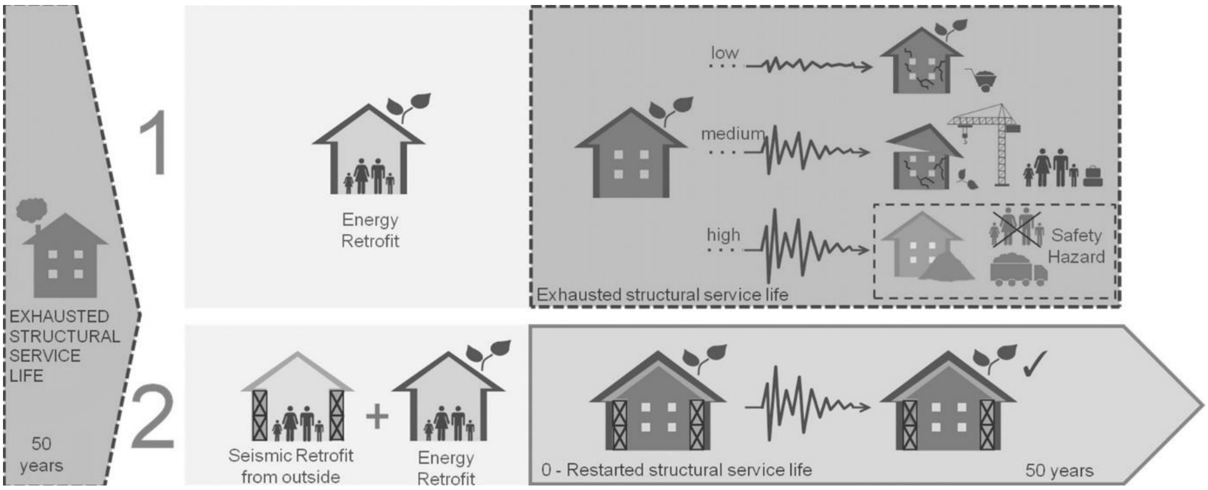
Apart from the above, many other modern methods for more efficient heating, cooling and hot water, have been or still are being developed. It is very important to implement all these techniques in real practice as soon as possible, since their application can really make a difference in reducing considerably our energy footprint. Needless to say, economically assisting measures by the states towards such investments should be promoted in this case too.

1.3 The integrated approach

So far, we have been talking about the common deficiencies of existing buildings in terms of seismic or energy performance and some existing solutions to them, without saying anything about how the retrofitting schemes should be applied chronologically. The trend, up to this day, has been to apply each type of retrofit separately, without taking into account the possible interconnection between the two (Calvi et al. 2016). This dependence, however, does exist as a potentially high seismic risk can affect the environmental impact of an existing building (Belleri and Marini 2015).

More specifically, a building with a sole energy upgrade will always be prone to structural damage if it is located in an area of high seismicity. In that case, if an earthquake was to occur, the structure would undergo damage that, depending on the intensity, could even lead to a collapse, thus jeopardising the very lives of its inhabitants and transforming the initial investment to practically a waste of funds (see Figure 18). On the other, less usual, hand, a building that has been retrofitted only seismically, will be future-proof in terms of structural performance, but will always be wasting a lot of energy trying to overcome the inherent heat losses due to its old construction practices.

Figure 18. Energy versus integrated energy and seismic upgrading



Source: Belleri and Marini 2015 (modified).

The obvious way to overcome all the above-mentioned problems, is to stop thinking of the two types of upgrading as separate, but instead as tightly connected to each other. This means, that both of them should be applied at the same time, so that at the end of the day, we end up with a building that is both seismic and energy proof. Of course, this integrated approach demands a higher initial investment, which might not be available in many cases. However, if one takes into account the lower construction costs (than those if the same upgrades were to be applied separately), due to labour and scaffolding costs, as well as the economic benefits that come of applying the retrofits simultaneously, it is actually the more reasonable choice to follow, as it will be demonstrated in later chapter. For that reason, it is highly recommended that the state should consider funding up to a certain point such retrofitting efforts.

The following table sums up the disadvantages that lie in each possibly selected retrofitting scheme:

Table 1 Disadvantages and advantage of various retrofitting schemes

Retrofitting scheme	Possible drawbacks	Possible benefits
Energy retrofitting	Investment is lost and life-safety is compromised when destructive earthquakes occur (for seismic areas).	Energy efficient buildings
Seismic retrofitting	Running energy costs remain high, especially in harsher climates	Safe building in seismic areas
Seismic and Energy Retrofitting independently applied	Higher initial investment is needed, which might not be available.	Both life safety and energy efficiency are provided
Integrated/Combined Energy and Seismic	Overall cost could be reduced	Both life safety and energy efficiency are provided

Needless to say, the *intensity* of the applied retrofitting scheme should always meet the unique needs of each building. For example, in cases of low seismicity, maybe only few structural interventions (or none) could be sufficient or in buildings located in harsher climates, a more aggressive energy retrofit might be more efficient. It is not de facto that we should always upgrade a given building to be both more seismically resistant and energy efficient as much as possible, but instead assess the available data in order to come up with the best and most reasonable solution.

The concept of combined seismic and energy retrofitting was proposed and investigated experimentally initially for the case of masonry subjected to out-of-plane or in-plane loading by Triantafillou et al. (2017, 2018), and Karlos and Triantafillou (2018), who introduced the combination of TRM with thermal insulation material. Bournas (2018 a, b), Mastroberti et al (2018) and Gkournelos et al. (2019) proposed a similar system for the concurrent seismic and energy retrofitting for the case of RC building envelopes. This concept is further explored in the following chapters of this report via a series of case studies.

2 Case Studies

The main work done for the purposes of this report mainly consists of a number of case studies run on a model building, accounting for various structural configurations. This model building is not an existing structure but was designed using old regulations active in Greece before 1985. This standard was chosen because of its availability to the author, but also because it quite accurately describes the norm that was followed in southern Europe at that time (See Appendix A). Therefore, it is assumed that the model buildings used hereafter are able to represent the typical old RC structures that can commonly be found in south Europe.

Three different configurations were considered so as to take into account the effect of the building height as well as the non-regularity in height. These configurations include a 2-storey and a 5-storey building with infill walls in all their floors plus a 5-storey building with a pilotis configuration that is with infill walls in all its floors except for the base one. Especially, the last case was – and still is – quite popular, mainly because of the free space it provides to meet the parking needs of the residents. In all cases, single pane windows and no thermal insulation were assumed to exist.

The sections that follow describe in detail the specifics of the case studies as well as the retrofitting measures that are taken. Furthermore, the seismic and the energy modelling procedures using OpenSees and EnergyPlus are thoroughly explained along with their details.

2.1 Configurations

2.1.1 Structural

The structure that was selected to be analysed is a regular in plan, reinforced concrete building (see Figure 19). There are 4 bays, 5 metres wide in the X-direction and 2 bays, 6 metres wide in the Z-direction, yielding a total floor area of 240 m^2, that could accommodate 2 to 3 apartments. The three different structural configurations can be seen clearly in figure (see Figure 20).

Figure 19. Plan view of the building models

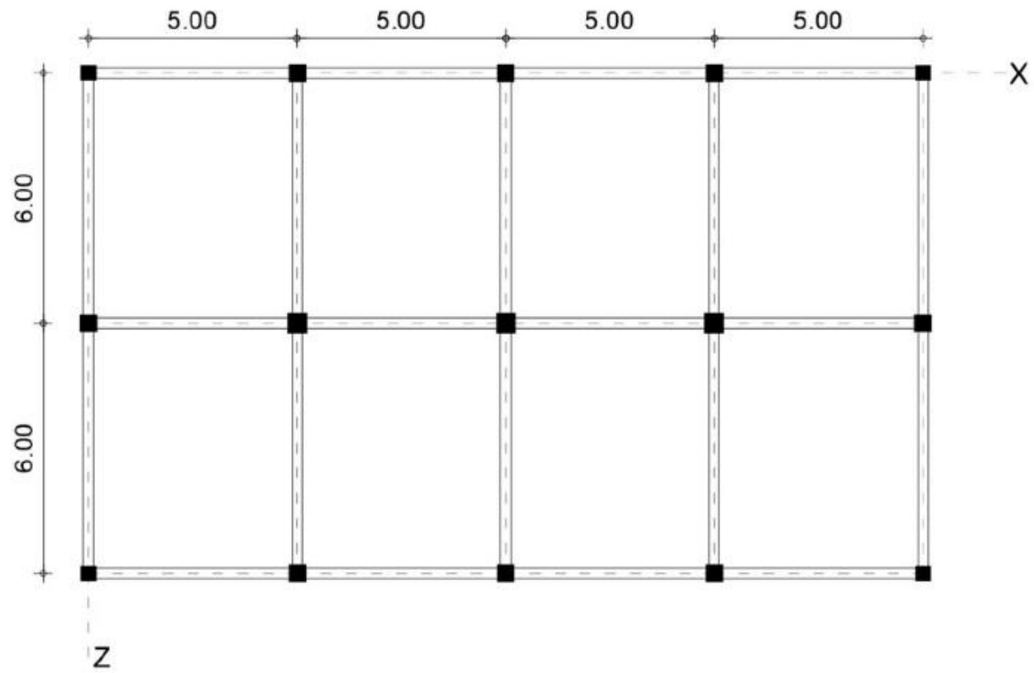
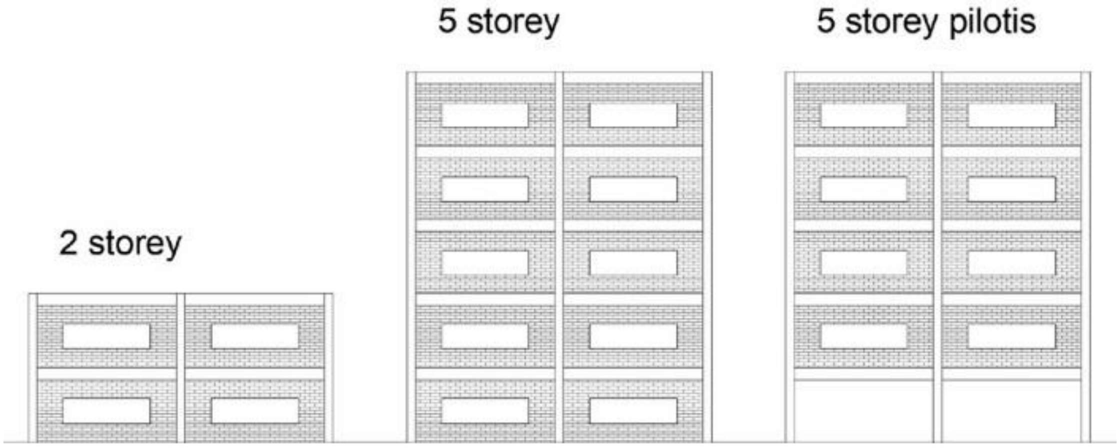


Figure 20. Front view of the building models

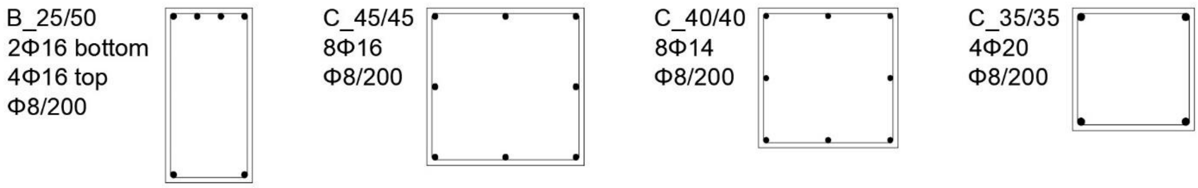


2.1.1.1 Element detailing

Using the Greek seismic provisions existing prior to 1985, the three building configurations were detailed, accounting for a lateral load of 6% of the building weight. The concrete strength was assumed to be C16/20 and the steel quality S400. Practically, this design was done twice, once for the 2-storey and once more for the 5-storey structure, since the infill walls were omitted during this phase – as was done back then and still is done today. The above procedure yielded the following results, which can also be seen in Figure 21.

- The beams in all three cases have a cross section of 250x500 mm with 4Φ16 at the top flange and 2Φ16 at the bottom at the supports (see Figure 21, B_25/50).
- The central columns of the 5-storey building are 450x450 mm with 8Φ16 (see Figure 21, C_45/45).
- The side columns of the 5-storey building are 400x400 mm with 8Φ14 (see Figure 21, C_40/40).
- The corner columns of the 5-storey building are 350x350 mm with 4Φ20 (see Figure 21, C_35/35 as in the 2-storey).
- Shear reinforcement was Φ8/200 for all the elements in all the three cases.

Figure 21. Steel detailing of concrete members



2.1.1.1.1 Wall typology

Infill walls are known to considerably alter the seismic performance of a structure, in case the latter is relatively flexible, as is the case with old buildings that do not have strong shear walls. Therefore, especially when assessing such structures, it is imperative to take their contribution into account, something that cannot be done easily and reliably.

Figure 22. Section of the infill walls in the models

Source: http://e-oikodomos.blogspot.gr/2011/05/blog-post_18.html.

In the case studies presented herein, it is assumed that only the infills located at the perimeter of the building contribute to its lateral stiffness, since the internal ones are in most cases much thinner and also not always embedded inside concrete frames. It is also assumed that all the outer walls have a central opening that covers 25% of their total area, in order to account for the existence of balcony doors and windows. These walls are built with 9-hole bricks (9 cm x 9 cm x 19 cm) and their section can be seen in

Figure **22**. The thickness is 19 cm and goes up to 23 cm after the application of mortar and finishing on the wall faces.

2.1.1.1.2 Retrofitting measures

For the structural retrofitting of the model buildings, the infill wall strengthening with TRM method (Koutas et al. 2015a, b) was chosen. As in the mentioned publication, the same polymer-coated E-glass textile (25x25 mm mesh size and 405 g/m^2 weight) was used for the wall strengthening scheme, along with all the other same measures mentioned (same textiles for the spike anchors and the columns' strengthening, same mortar etc.).

What had to be dimensioned independently was the exact amount (number of layers) of strengthening that had to be applied to each floor. That was decided after running a number of pushover tests (with triangular lateral load) with OpenSees models, where the goal was to increase the lateral resistance as much as possible, while using no more than two layers per side and trying to utilize most of the externally applied reinforcement. The above procedure yielded the following structural retrofitting schemes, shown in Table 2.

Table 2 Selected retrofitting schemes for the case studies

Building type	Retrofitting schemes
2-storey	Both floors are reinforced with 2 layers of 2-sided TRM.
5-storey	The first two floors are reinforced with 2 layers of 2-sided TRM, the 3rd and 4th with 1 layer of 2 sided and last floor with 1 layer of 1 sided TRM.
5-storey pilotis	*One* of the two bays in Z-direction and *two* of the four bays in X-direction (see Figure 19) are infilled with a wall same as those of the above floors but without opening and reinforced with 1 layer of 2-sided TRM. Walls of the second floor are reinforced with 2 layers of 2-sided TRM, of the third with 1 layer of 2-sided TRM and of the fourth with 1 layer of 1-sided TRM. Lastly, the fifth floor was not retrofitted.

2.1.2 Energy

The energy performance of the initial buildings is expected to be poor, since no special energy efficiency measures were taken at the design phase. To improve this situation, the simpler choice of adding insulation externally and changing the existing fenestration surfaces was preferred and no additional actions were done regarding the buildings' existing mechanical systems and electrical equipment. The insulating material that is used for the walls (and the top slab) is a typical 40 mm thick commercial polystyrene with a thermal conductivity of 0.03 W/m/K. On the other hand, the initial 3 mm thick single pane fenestrations surfaces are replaced with double ones (again 3 mm thick) that have a 10 mm air gap, as shown in Figure 23. Thermal upgrading in walls and fenestration surfaces shows the profiles for a wall and a window, before and after the energy retrofitting.

Table 3 contains all the parts of the construction surfaces that fully describe the buildings' envelope as well as the internal floor partitions. Items in italic font are those that are added after the energy retrofitting.

Figure 23. Thermal upgrading in walls and fenestration surfaces

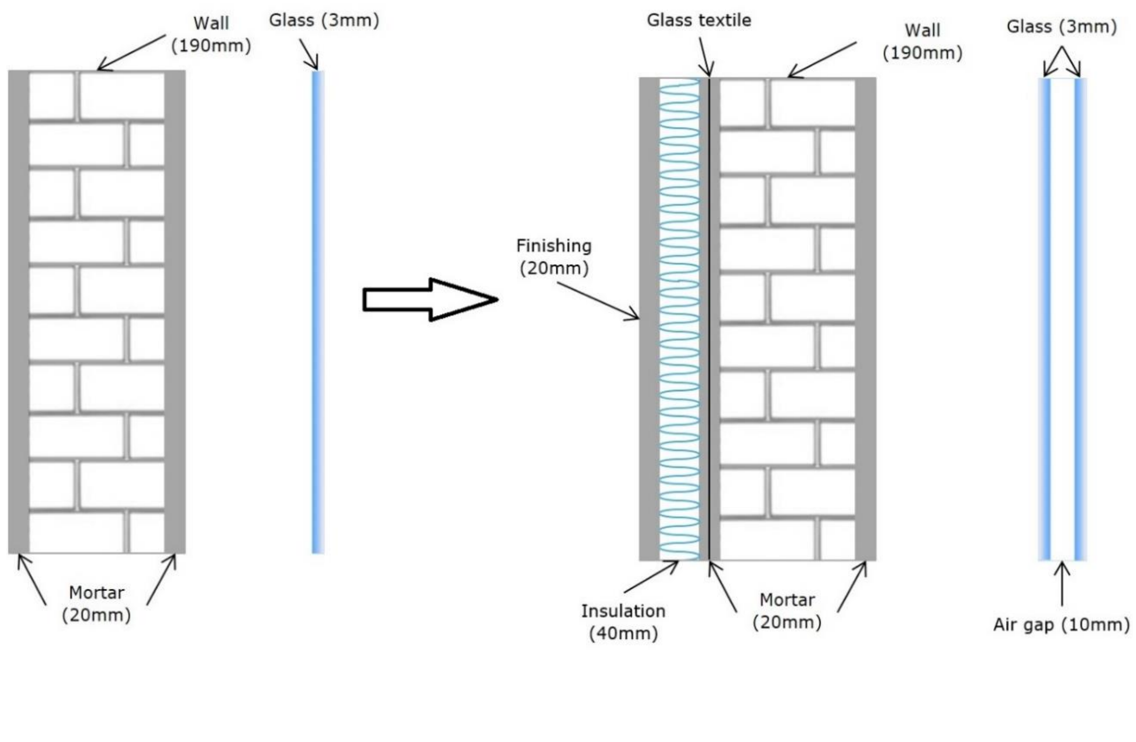


Table 3 Construction surfaces (from outer to inner) for thermal simulations.

Exterior wall	Exterior floor	Exterior roof	Interior floor	Opening
Mortar/Finish	Concrete slab	Covering	Covering	Glazing
Insulation	Covering	*Insulation*	Concrete slab	*Air gap*
Mortar/Finish		Concrete slab	Mortar/Finish	*Glazing*
Infill		Mortar/Finish		
Mortar/Finish				

2.2 Seismic modelling with OpenSees

The earthquake simulations for this report were all carried out using OpenSees, a free, open-source software framework, that allows users to create finite element applications for simulating the response of structural and geotechnical systems subjected to earthquakes. Since this is a software framework and not a commercial finite element program, it requires some extra programming effort from the user in order to create and run even a simple model. The interface is not graphical and the only way of communicating with OpenSees is through a programming language called TCL (standing for "Tool Command Language") to which OpenSees adds functionality regarding structural objects (nodes, elements, materials etc.). Therefore, the user has to program some routines in TCL, in other words create plain text files containing all the necessary commands to create and run a model.

Although it might seem difficult initially to use OpenSees, the fact that its interface is a powerful programming language gives literally limitless possibilities to the user. Apart from that, it is constantly updated with new material properties and elements, following the latest developments in the scientific community. That is why it is widely used from researchers all over the world working on the field of earthquake engineering.

For the purposes of this report, more than 3600 lines of code in TCL were written. These included procedures for the rapid model creation, the loading of the elements with gravity loads, but also functions to accommodate the post-processing of the results. In the following sections, the most important aspects of the modelling with OpenSees are thoroughly explained.

2.2.1 Finite elements and materials

For the building models described above (Fig. 20), only linear members were used for both the RC members (columns and beams) and the infill walls. Therefore, from a geometrical point of view, the finite element models used here are very simple, since they represent rectangular reinforced concrete structures, with no irregularities in plan or elevation and without the addition of stairways. Although, that choice might not be the most representative for the reality, it was preferred so as to minimize any secondary phenomena that could occur and focus only in the behaviour of the buildings before and after strengthening the walls with TRM.

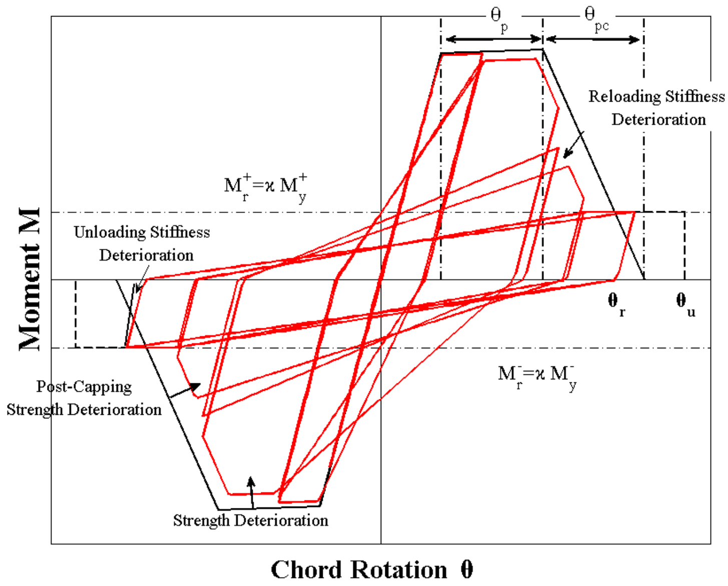
2.2.1.1 Reinforced concrete members

All RC members were simulated using distributed plasticity elements with 5 integration points along their length. At each integration point, a previously specified moment-curvature law was assigned, the properties of which were calculated with an Excel-based program developed in this study. This approach was preferred than the typical lumped plasticity models that are commonly used in RC models, as it was easier and faster to implement in OpenSees. It is noted that phenomena like bar slippage cannot be captured inherently with distributed plasticity models. Moreover, since the integration points were described by pre-specified moment-curvature relationships, the biaxial bending and axial force interaction was also not considered. However, the level of approximation achieved with the proposed scheme was more than adequate for the purposes of this work, given the fact the structural behaviour of the structure is driven mainly by the infill walls, whose simulation contains much more uncertainties.

The *forceBeamColumn* finite element provided by OpenSees was used for all linear members. It is a non-linear element that is based on the iterative force-based formulation and assumes Gauss-Lobatto integration at each integration point. The moment-curvature relation (for both axes) at each integration point was described using the *ModIMKPeakOriented* uniaxial material, which was developed mainly to simulate moment-rotation relations but can also be used for moment-curvature relations through suitable choice of its parameters (see Figure 24. Modified Ibarra-Medina-Krawinkler deterioration model). As far as the axial, torsional and shear behaviours are concerned, these were assumed to be linear elastic and their "elasticity modulus" was chosen according to the concrete properties of the structures; for that reason, the *Elastic* uniaxial material was used.

Lastly, second order effects were considered by using the *Corotational* geometrical transformation for all the concrete column elements. This type of transformation can be used in large displacement-small strain problems and was preferred to the *PDelta* one because of its higher accuracy. For the beam elements on the other hand, a simple *Linear* geometrical transformation was used, since their behaviour does not affect the second order effects in a structure and also because the use of a *Corotational* transformation does not allow the assignment of uniform loads on the elements. Additional information concerning the above elements, materials and geometric transformations can be found in the OpenSeesWiki Command Manual.

Figure 24. Modified Ibarra-Medina-Krawinkler deterioration model



Source: http://opensees.berkeley.edu/wiki/index.php/Modified_Ibarra-Medina-Krawinkler_Deterioration_Model_with_Pinched_Hysteretic_Response_(ModIMKPinching_Material).

2.2.1.2 Modelling of the infill walls

2.2.1.2.1 Initial walls

The most difficult part in the modelling of an old structure is possibly the addition of the infill walls in the structural model. This happens because of the difficulty in obtaining all the material properties needed for an accurate simulation, but also because their simulation itself is not an easy task, which gets even more difficult when openings exist, as is usually the case. Generally speaking, the following ways exist in order to take into account infill walls in a structural model (Furtado et al. 2015):

1. Micro-modelling (see Figure 25).

(a) Simplified micro-modelling where the expanded units are represented by continuum elements and the properties of the mortar and the brick–mortar interface are lumped into a common element.

(b) Detailed micro-modelling where brick units and the mortar are represented by continuum elements and the brick units–mortar interaction are represented by different continuum elements.

2. Macro-modelling (see

3.

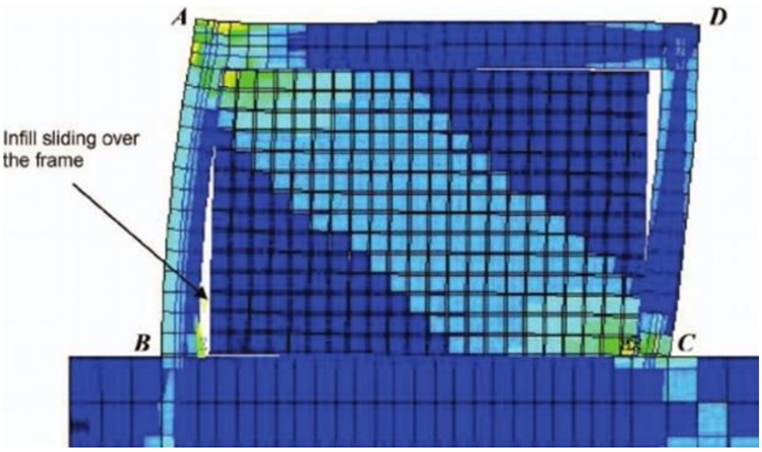
4. Figure **26**).

(a) Simplified single strut models.

(b) Multiple strut models.

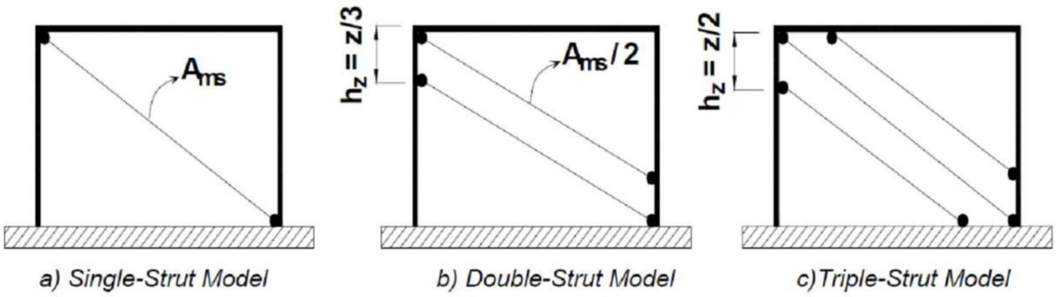
For the purposes of this work and in order to be consistent with the work of Koutas et al. (2015b), the simplest single-strut model was used. With this model, it is *only* possible to capture the *global* behaviour of the analysed structure. The internal forces in the concrete frame elements will therefore *not* be accurate, something that we will have to take into account, when we will reach the point of estimating the losses due to earthquakes. This level of approximation is satisfactory in our case, since both the lateral stiffness and strength of the buildings are coming mostly from the walls rather than the frame members.

Figure 25. Detailed finite element infill model



Source: https://www.researchgate.net/figure/Finite-element-mesh-for-frame-with-weak-infill-analysis-using-DIANA_fig9_235190091.

Figure 26. Simplified strut infill models



Source: https://www.slideshare.net/openseesdays/d2-012-rodriguesosdpt2014.

As in the work of Koutas et al. (2015b), the behaviour of the wall was described using the hysteretic model proposed by Fardis and Panagiotakos (1997) and shown in Figure 27, using *exactly* the same material properties to extract the wall parameters and changing only the geometrical ones. For completeness, the formulas used to obtain these parameters are quoted here, with reference to Figure 27 for the nomenclature.

$$V_{cr} = \tau_{cr} A \tag{2.1a}$$

$$K = GA/H_{cl} \tag{2.1b}$$

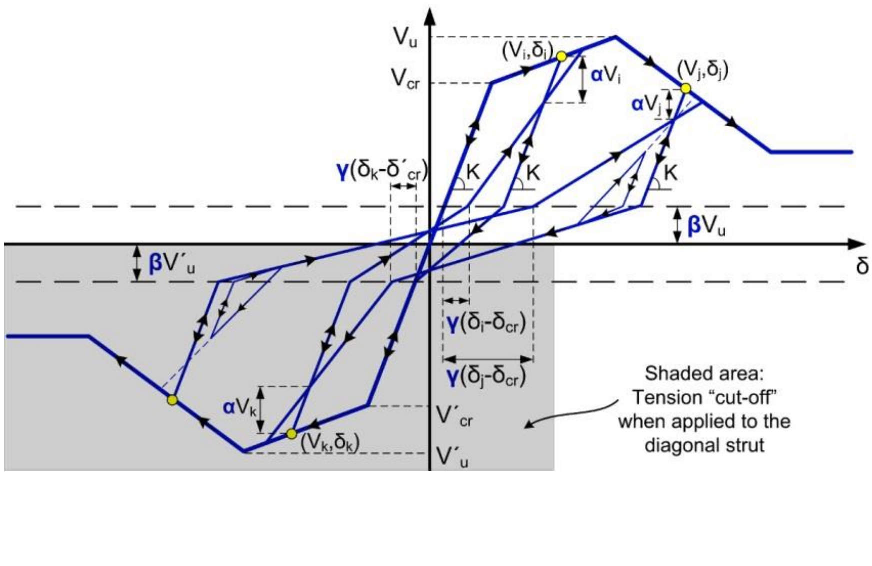
$$V_u = 0.56(\lambda H)^{-0.875} f_w H_{cl} t_w \cot\theta_{str} \tag{2.1c}$$

$$\lambda = \left(\frac{E_w t_w \sin 2\theta_{str}}{4E_c I_c H_{cl}}\right)^{\frac{1}{4}} \tag{2.1d}$$

$$K_u = E_w \left(w_{inf} t_w\right) \cos^3\theta_{str} / L_{cl} \tag{2.1e}$$

$$w_{inf} = \frac{0.175 L_{cl}}{\cos\theta_{str} \, (\lambda H_{cl})^{0.4}} \tag{2.1f}$$

Figure 27. Hysteretic behaviour of wall infill strut



Source: Koutas et al. (2015b).

In the above equations 2.1, the symbols have the meaning that is explained in the following list:

— H_{cl}, L_{cl} and t_w are the clear height, width and thickness of the wall.

— A is the shear area of the wall given by $A = L_{cl}t_w$.

— G is the shear modulus of the wall.

— f_w is the compressive strength of the infill.

— θ_{str} is the angle of the diagonal strut with the horizontal level.

— E_w is the elasticity modulus of the infill at the direction of the diagonal strut.

— E_c is the elasticity modulus of the columns of the outside frame.

— I_c is the moment of inertia of the columns of the outside frame normal to the wall plane.

In addition to the above, the model of Fardis and Panagiotakos (1997) contains 5 parameters that govern the overall shape of the hysteresis loops, namely p_1 (percentage of the negative stiffness of the model in relation to the original elastic stiffness), V_{res} (the residual strength of the infill), α, β and γ. The values proposed by Koutas et al. (2015b) were used in this work, which are $p_1 = 0.015$, $V_{res} = 0.5V_u$, $\alpha = 0.15$, $\beta = 0.2$ and $\gamma = 0.2$.

Lastly, it is important to note that all the above refer to the shear force versus the shear displacement of a wall. To get the relevant forces and displacements on the diagonal direction, the following geometrical transformation needs to be done:

$$V = F \cos \theta_{str} \tag{2.2a}$$

$$\Delta L_{str} = \left(\frac{L_{cl}}{H_{cl}} \sin \theta_{str}\right) \delta \tag{2.2b}$$

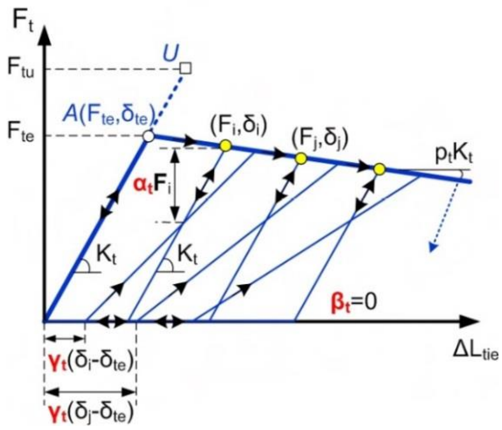
In the OpenSees environment, two members were used per infill (one for each diagonal direction) using the *twoNodeLink* finite element. Both elements were assumed to be activated both in tension and compression, therefore their strength and stiffness properties were simply divided by 2. Their axial behaviour was described using the *Pinching4* material provided by OpenSees, after it was configured to match the properties of the Fardis and Panagiotakos (1997) model.

2.2.1.2.2 TRM retrofitted walls

Following the simple route of the single strut model, the retrofitted walls were also modelled using the same approach. Now, 2 elements were used to describe the wall itself plus another 2 to account for the TRM reinforcement, thus every retrofitted wall was modelled with 4 *twoNodeLink* finite elements. The axial hysteretic behaviour of the TRM reinforcement was described using the model of Koutas et al. (2015b) (see

), which is practically a case of the Fardis and Panagiotakos (1997) model for $K = K_u$ (and $V_{cr} = V_u$), $\beta = 0$ and $p_1 = 0$ yielding a simple bilinear model. Again the *Pinching4* material, provided by OpenSees, was used to describe this behaviour.

Figure 28. Hysteretic behaviour of wall infill tie



Source: Koutas et al. (2015b).

In order for the TRM-tie behaviour to be fully defined, only 4 parameters need to be defined:

— The effective tensile strength of the tie, F_{te}.

— The initial stiffness, K_t.

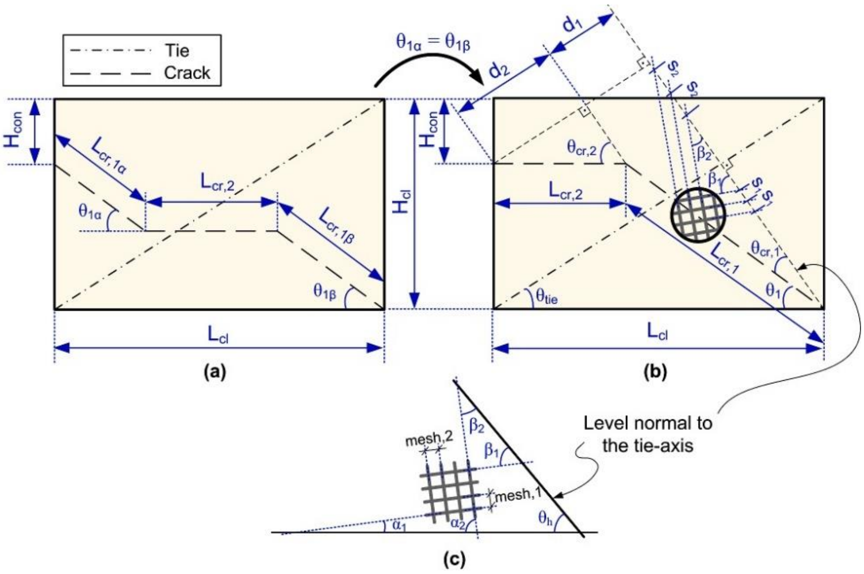
— The shape parameters α_t and γ_t.

According to Koutas et al. (2015b), the tensile strength and stiffness of the tie can be evaluated through the formulae 2.3:

$$F_{te} = \sum_{i=1}^{2} \sum_{j=1}^{2} \frac{A_{t,i}}{s_i} (\varepsilon_{te,i} E_{t,i}) d_j \left[\cot \theta_{cr,j} + (2i-3) \cot \beta_i\right] \sin \beta_i \tag{2.3a}$$

$$K_t = \sum_{i=1}^{2} \sum_{j=1}^{2} E_{t,i} \left\{\frac{A_{t,i}}{s_i} d_j \left[\cot \theta_{cr,j} + (2i-3) \cot \beta_i\right] \sin \beta_i\right\} / L_{e,tie} \tag{2.3b}$$

Figure 29. Geometric representation of the assumed cracking pattern



Source: Koutas et al. (2015b).

With reference to Figure 29, the nomenclature for the equations 2.3 presented above is as follows:

$$s_i = \frac{mesh, i}{\sin \beta_i} \tag{2.4a}$$

$$d_j = L_{cr,j} \sin \theta_{cr,j} \tag{2.4b}$$

$$L_{cr,1} = \frac{(H_{cl} - H_{con})}{\sin \theta_1} \tag{2.4c}$$

$$L_{cr,2} = L_{cl} - L_{cr,1} \cos \theta_1 \tag{2.4d}$$

$$\theta_{cr,1} = 90° - \theta_{tie} - \theta_1 \tag{2.4e}$$

$$\theta_{cr,2} = \theta_h \tag{2.4f}$$

$$A_{t,i} = n_s n_t t_t mesh, i \tag{2.4g}$$

$$\theta_1 = \tan^{-1} \frac{2b_{br}}{d_{br}} \tag{2.4h}$$

In the above formulae, $A_{t,i}$ is the cross-section area of the TRM in the direction i and n_s, n_t, t_t and $mesh, i$ are the number the number of the strengthened sides of the infill panel (thus 1 or 2), the number of TRM layers per side, the thickness of each TRM layer and the spacing between fibre rovings in direction i respectively. Moreover, b_{br} and d_{br} are the dimensions of the bricks. Lastly, according to Koutas et al. (2015b), the parameter $\varepsilon_{te,i}$ is assumed to be 0.8% for one layer of TRM (and then be inversely proportional to the square root of $E_t \rho_t$), $L_{e,tie}$ is taken as 25% of L_{tie} and both parameters α_t and γ_t were taken equal to 0.4.

The existence of TRM also affects the behaviour of the infill struts, thus their properties have to be updated in case the strengthened structure is analysed. All these material property values that were used in this work were those experimentally found by Koutas et al. (2015a,b). Obviously, in real practice, all these values should be experimentally found, something that can be a very difficult task to complete.

2.2.1.2.3 Consideration of wall openings

Modelling a wall infill is a procedure that contains a large number of uncertainties, especially when openings exist in its body, as is almost always the case. When detailed finite element models are used, everything is considered implicitly through the model itself which contains the opening. However, in the more common case, where strut macro-models are used, being able to capture the actual behaviour can be much more challenging.

To overcome this problem typically reduction factors for the strength and stiffness of the infill are assigned. These are based on empirical equations, which given the dimensions of the opening in relation to the infill, provide the requested reduction factor ρ. Moreover, in almost all cases, there is the assumption that the opening should be located in the middle of the masonry wall. A large number of such equations exist, the first of which dates back to 1956 and was developed by Polyakov (1956). The most recent one adopted in this study, was proposed by Chen and Liu (2015), is given by Eq. 2.5 (α_{α} is the area percentage of the opening):

$$\rho = 1 + 2.751\alpha_{\alpha}^2 - 3.17\alpha_{\alpha} \tag{2.5}$$

Figure 30. Wall opening reduction factors from various researchers

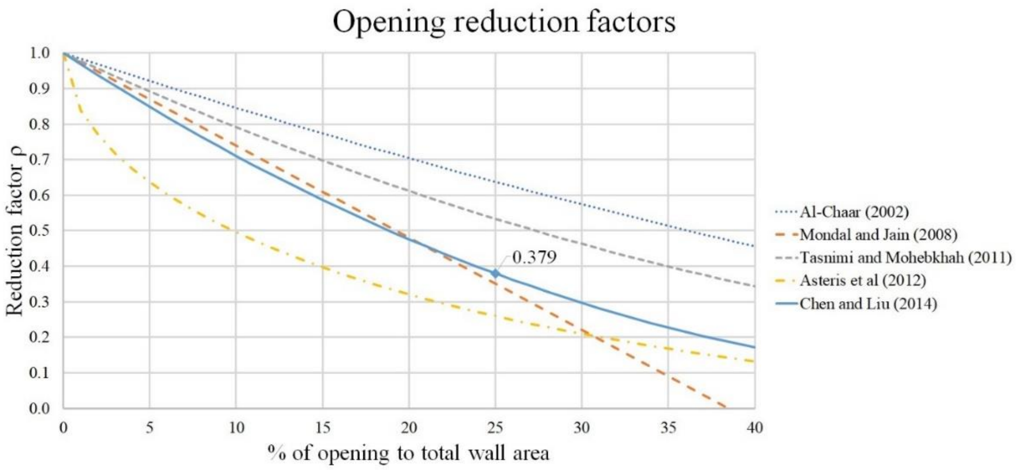


Figure 30 provides a comparison between 5 different empirical equations, all of which are supposed to provide the same reduction factor. As it can easily be seen, the dispersion is *very* large. The reason that the equation by Chen and Liu (2015) was chosen is because it is the latest one and it was created following a large number of detailed finite element studies. The finite element models used were validated first, using experimental results from various experiments; therefore the resulting equation was regarded as the best to choose.

In the case studies of the present report an opening of 25% of the total wall area was assumed. For that value, the reduction factor according to equation 2.5 is 0.379 as it is also shown in Figure 30.

Combining all the previously presented data, the next table provides the properties of the infill strut elements. Note that these are the properties in terms of shear force-displacement, thus not the ones to be used as input in OpenSees. The nomenclature of the first column provides information on the wall orientation (X or Z direction, see Figure 19) and if it is strengthened, the number of layers (1L or 2L) and the number of retrofitted sides of the wall (1S or 2S).

Table 4 Properties of infill struts used in analyses

Case	V_{cr} (kN)	K (kN/m)	V_u (kN)	K_u (kN/m)
Unstrengthened-X	132.1	187038	363.7	21188
Unstrengthened-X	160.3	226834	452.8	23071
Strengthened-X-2L-2S	288.0	359168	405.2	21470
Strengthened-Z-2L-2S	349.3	435587	504.5	23379
Strengthened-X-1L-2S	203.3	257517	384.4	21357
Strengthened-Z-1L-2S	246.6	312307	478.5	23256
Strengthened-X-1L-1S	167.7	222278	374.1	21272
Strengthened-Z-1L-1S	203.4	267571	465.7	23164

Similarly, the same procedure is followed for the infill tie elements. The shear force-displacement relations are again presented for the strengthened cases only. Comparing the two tables, it is interesting to note that the application of TRM greatly enhances the properties of the infill struts, so that in the end the total increase in the wall strength comes mainly from the enhancement of the struts' properties rather than those of the ties.

Table 5 Properties of infill ties used in analyses

Case	V_{max} (kN)	K (kN/m)
Strengthened-X-2L-2S	81.0	8669
Strengthened-Z-2L-2S	87.1	8410
Strengthened-X-1L-2S	60.6	4588
Strengthened-Z-1L-2S	65.2	4451
Strengthened-X-1L-1S	30.3	2294
Strengthened-Z-1L-1S	32.6	2225

2.2.2 Earthquake simulation

Having defined reliably the structural model, the next step is to start running the simulations. First, the gravity loads have to be defined and after that, the earthquake analyses can begin. Concerning the gravity loads, the typical $G + 0.3Q$ combination was assumed to be acting on the structure prior to the earthquake. These loads were assigned uniformly to the beam elements of each floor, assuming the well-known trapezoidal distribution from the slab to its supporting beams. After these definitions, the earthquake analyses were run and their details are given in the next sections.

2.2.2.1 Simulation details

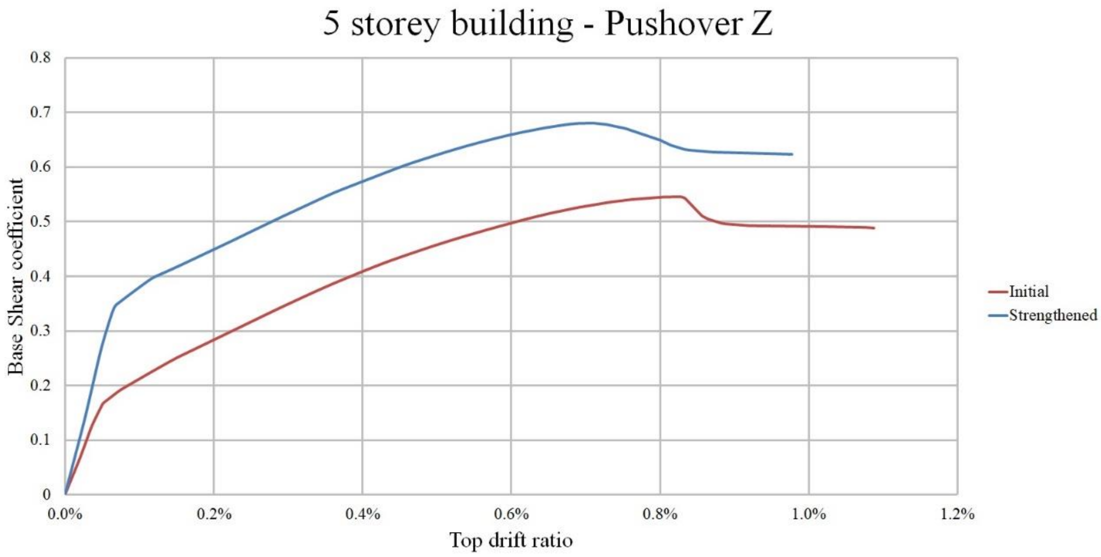
For the purposes of this work, a large number of non-linear, time-history analyses were run, using a total of 11 real earthquake records, mainly from the Greek territory, but also from Italy, Turkey and also some well-known from the rest of the world. These analyses were non-linear both in terms of geometry and material behaviour.

Because of the large number of analyses (which were also very time-consuming), as well as the abundance of their results, only the weak direction of the building was examined, namely the Z-direction as is shown in Figure 19. This choice was made due to the fact that, that direction had only 2 bays (compared to the 4 of the X-direction) and thus the building was more prone to be damaged from lateral loads. For the loss evaluation and the retrofitting schemes however, it was reasonably assumed that the damage and the relevant retrofitting would be evenly distributed to both directions.

Moving to the specifics of the time-history simulations, a *Newmark* integrator and a `Newton` solution algorithm were used to obtain the solution at each time-step. As far as the latter is concerned, it was selected to be half the sampling time-step of the accelerogram. However, since these types of analyses can often run into instabilities, a TCL procedure was programmed to automatically change the solution algorithm, namely by lowering the time-step or allowing for more iterations to overcome such problems.

Apart from the time-histories, a number of pushover analyses were needed to run in order to verify that the behaviour of the structure is reasonable. Moreover, they were used to correlate the structural damage from the earthquake to specific damage states so as to evaluate the direct earthquake losses. Figure 31 shows the pushover curves for the initial and retrofitted 5-storey building assuming a triangular distribution of the lateral loads to the floors.

Figure 31. Pushover curves for initial and retrofitted 5-storey structure



2.2.2.2 Earthquake records

The following 11 earthquake records were used in the time-history analyses run for the purposes of this report:

— **El Centro** (1940), the most famous earthquake that occurred in the Imperial Valley in south-eastern Southern California, caused widespread damage to irrigation systems and led to the deaths of 9 people.

— **Friuli** (1976), occurred in the Friuli region in northeast Italy near the town of Gemona del Friuli. Up to 978 people were killed, 2,400 were injured, and 157,000 were left homeless.

— **Kalamata** (1986), occurred in the town of Kalamata in Greece, caused the injury of 300 and the death of 22 people.

— **Loma Prieta** (1989), occurred in Northern California and was responsible for 63 deaths and 3,757 injuries.

— **Roma** (1990), in the central part of Italy.

— **Aegion** (1995), occurred in the northern part of Peloponnese in Greece, near the town of Aegion and caused the death of 26 people.

— **Kobe** (1995), occurred in the southern part of Hyōgo Prefecture, Japan killing 6434 people.

— **Athens** (1999), occurred in the city of Athens in Greece causing the death of around 150 people.

— **Sakaria** (1999), occurred in north-western Turkey, killed around 17,000 people and left approximately half a million people homeless.

— **Kefalonia** (2014), occurred in the island Kefalonia of the Ionian see in Greece.

— **Lefkada** (2015), occurred in the island Lefkada of the Ionian see in Greece and caused the death of 2 people.

At this point, it is necessary to say that when analysing a specific building, the engineer should test it against earthquakes that can take place at the structure's location. Therefore, he/she should use accelerograms from past earthquakes that have occurred around that very location, since different faults around the world, produce motions with distinct characteristics. Of course, that might not always be possible due to the unavailability of the needed data. This however, is rapidly changing with the employment of earthquake databases in many countries all over the world, so as to provide the necessary data to the design engineers. In our case now, a less precise approach was followed and earthquake records from many different places were used, as the buildings analysed did not have a specific location and were actually case studies aimed at representing a wider family of RC buildings situated in seismically active areas.

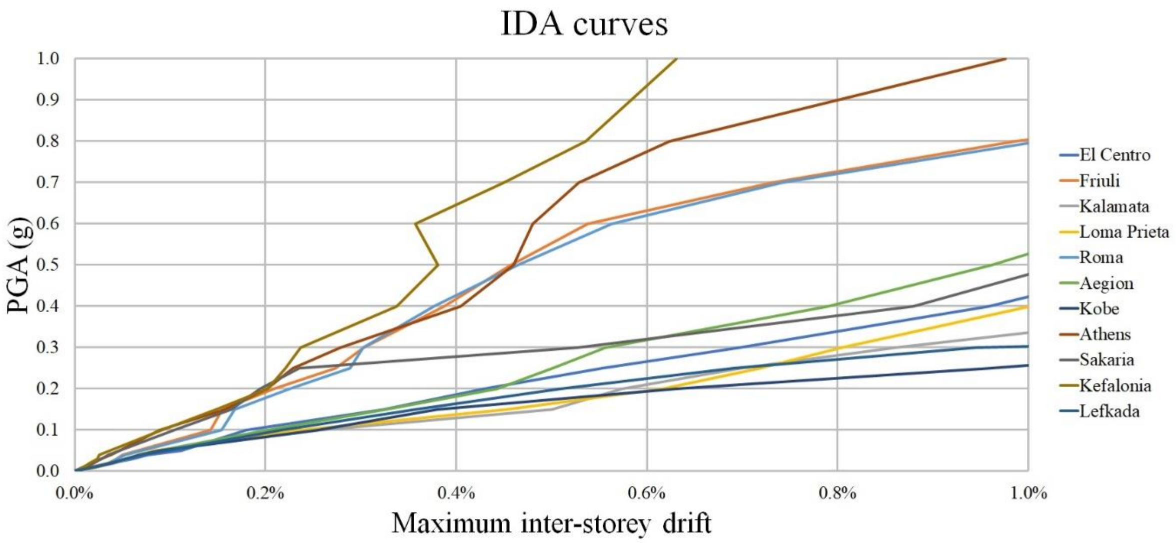
2.2.2.3 Incremental dynamic analysis

In order to examine the behaviour of the buildings in a wide range of ground motion intensities, the above mentioned accelerograms were scaled from very small to very large *Peak Ground Accelerations* (or *PGA*s in short), using a procedure called *Incremental Dynamic Analysis* (Vamvatsikos and Cornell 2001). According to that method, an earthquake record is scaled down to a small *PGA* and is then applied to the structure. During this analysis, an engineering parameter (or many), capable of describing the structural damage, is recorded and its maximum value is stored. Then the analysis is repeated for larger *PGAs* until a limit set by the user is reached, or structural collapse occurs. The latter can be considered to take place when the solution algorithm goes unstable, provided that the structural model is correct.

In the present study, the 11 records were scaled from *PGA*s as low as 0.001g up to the extreme value of 1.0g (of course this was not reached in all cases due to collapse). A total of 17 analyses were run per earthquake, yielding a total number of 187 per building. Taking into account that 3 building cases were analysed twice (initial and retrofitted situation), 1122 non-linear time-history analyses were required. In all cases, the selected damage parameter was the maximum inter-storey drift (which is the most typical damage parameter) observed among all floors, which in almost all cases was that of the 1st floor.

After applying the above procedure, a "capacity" curve was created for each record by plotting the *PGA* (intensity measure) on the vertical axis and the maximum inter-storey drift (damage measure) on the horizontal axis. Figure 32 shows these curves for the 11 earthquake records that were used in this work for the 5-storey retrofitted building.

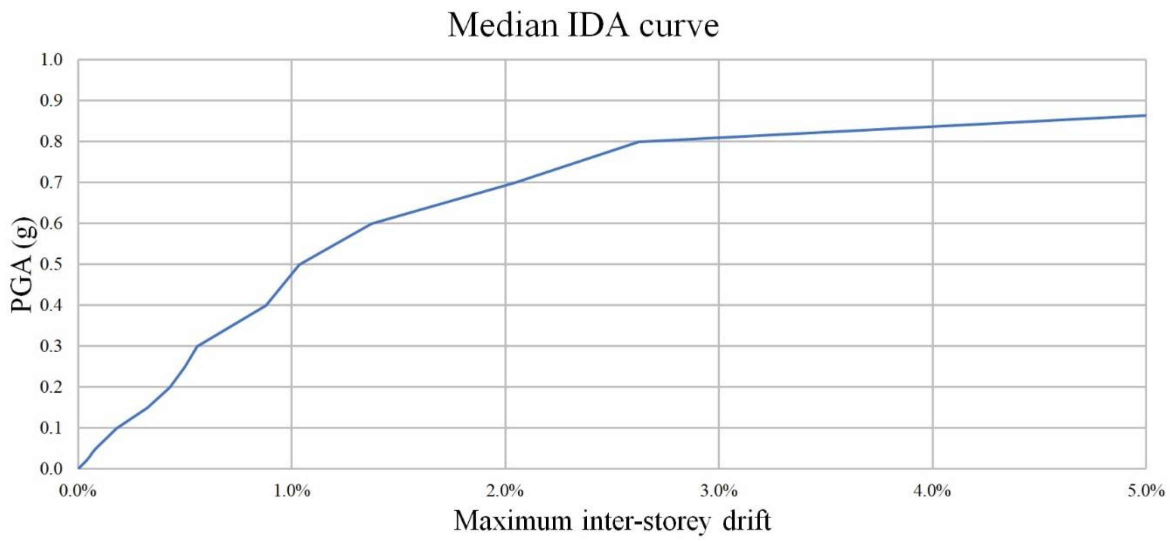
Figure 32. IDA curves for retrofitted 5-storey building for 11 earthquakes



It is clear that the response of the structure is totally different when applying different ground motion accelerograms. This happens because each earthquake is unique in terms of duration, amplitudes and frequency content, causing a considerably different structural response. In order to bring together all these different curves and construct a single one, a median curve is constructed. The median is preferred to the mean because its evaluation is, unlike the mean value, still valid if some earthquakes lead to collapse, thus yielding an infinite inter-storey drift. This curve, for the 5-storey retrofitted building, is given in Figure 33.

The high dispersion that is observed in Figure 32 can be reduced, if instead of the *PGA*, the "first-mode" spectral acceleration is used as the intensity measure (Vamvatsikos and Cornell 2001). However, in this work, the *PGA* was preferred as an intensity measure, so as to simplify the evaluation of the earthquake losses that will be presented in the next chapter.

Figure 33. Median IDA curve for retrofitted 5-storey building



2.3 Energy modelling with EnergyPlus

For the energy simulations of this work, another free open-source software, namely the so-called EnergyPlus was used. It is a whole building energy simulation program that engineers and researchers can use to model both energy consumption – for heating, cooling, ventilation, lighting and plug and process loads – and water use in buildings. Similarly to OpenSees, EnergyPlus is also console-based, meaning that it simply reads text input files and writes output to files.

A thermal analysis with EnergyPlus starts by defining the geometry of the structure, namely the surfaces that make up its distinct thermal zones. The layers of all surfaces plus any existing fenestration have to be given in detail together with their thermal properties. Then, the electrical equipment, lighting and hot water equipment are described to account for their thermal loads, as well as the mechanical systems responsible for heating, ventilating and air conditioning (HVAC). With everything defined, the user can then start a yearly simulation to estimate the building's energy consumption and needs (using weather files for the area the building is located, if there are available) or run design-day simulations to design the HVAC systems.

In this report, EnergyPlus was used in the simpler way that does not need the detailed modelling of the HVAC systems. Apart from the material and geometrical properties that were fully defined, the control of the inner temperature was done using an *IdealLoadsAirSystem* object, a simplistic way provided from the software to calculate thermal loads without modelling a full HVAC system. This component can be thought of as an ideal unit that mixes zone air with the specified amount of outdoor air and then adds or removes heat and moisture at 100% efficiency in order to meet the specified controls (EnergyPlus™ Version 8.7 Documentation, Input Output Reference).

2.3.1 Simulation details

2.3.1.1 Zoning

The first important action that has to be made in an EnergyPlus simulation is the thermal zoning of the analysed building. Depending on the detail level of the simulation, the thermal zones can be separate rooms, apartments, floors or even the whole building. Higher level simulations produce more accurate results, are more intensive and are mainly used when detailed HVAC modelling is already included. On the other hand, lower level simulations are less intensive and more appropriate when global energy properties are needed (e.g. the total energy consumption of a building). In our case, the zoning of the buildings was done floor-wise, since only the total energy needs were needed to be calculated in each case.

2.3.1.2 Materials and constructions

Four different materials were defined and used in EnergyPlus to model the various surfaces, found in the structures analysed. These, along with all the necessary thermal properties are given in Table 6. Their combination/layering results in the formation of the construction surfaces, which fully define the shells that close around each thermal zone were given in Table 3 (repeated here in Table 7 for the sake of clarity). Again, the new layers added after the retrofitting are shown in red, italic case to distinguish the initial and the retrofitted configuration.

Table 6 Materials used for thermal simulations

Material	Thickness (mm)	Conductivity $(W/m/K)$	Density (kg/m^3)	Specific heat $(J/kg/K)$
Masonry	190	0.51	1500	790

Concrete	150	2.50	2400	1170
Mortar/Finish	20	0.87	1800	1090
Insulation	*40*	*0.03*	*43*	*1210*

Table 7 Construction surfaces (from outer to inner) for thermal simulations.

Exterior wall	**Exterior floor**	**Exterior roof**	**Interior floor**	**Opening**
Mortar/Finish	Concrete slab	Covering	Covering	Glazing
Insulation	Covering	*Insulation*	Concrete slab	*Air gap*
Mortar/Finish		Concrete slab	Mortar/Finish	*Glazing*
Infill		Mortar/Finish		
Mortar/Finish				

2.3.1.3 Equipment

The HVAC equipment was modelled using an *IdealLoadsAirSystem* object, as mentioned earlier. This ideal system was configured so as to keep the inner zone temperature between 20°C (winter heating point) and 25°C (summer cooling point) and was by default 100% efficient. This level of efficiency was not realistic – too high for heating systems with burners and too low for cooling systems with AC units – therefore the consumptions that the program provided were processed to account for these different efficiency levels. The heating system (natural gas fuelled) was assumed to have a coefficient of performance (COP) of 0.65, while the AC units 3.1.

Apart from the HVAC systems, in order to accurately measure the total energy needs of a building, the loads from the electrical, lighting and hot water equipment need to be accounted for. The following typical values for residential buildings were used for the above energy needs:

— The lighting load was assumed to be $8\,W/m^2$.

— The electrical equipment load was taken as $11\,W/m^2$.

— The hot water energy needs were assumed to be $3\,W/m^2$.

Lastly, since no other means of indoor air renewal system were included in the analysis, it was assumed that the indoor air is replaced by fresh, outside air at a constant rate. This step is necessary otherwise we would implicitly assume that the indoor air would always remain the same, something that is of course unrealistic. Air replacement can take place in a number of ways, like when windows are open, when air leaks exist or when specialised mechanical system perform this task and is absolutely necessary for health reasons. To take this phenomenon, it was assumed that indoor air is replaced by outdoor at a rate of 0.5 Air Changes per minute (ACH). This value is typical for residential buildings and ensures a good air quality level.

2.3.2 Cases run

Putting together all the above, yearly energy simulation were run for the 3 buildings examined, before and after retrofitting. For each case, 4 different weather files, thus building locations, were used, from the Italian territory, namely:

— Bergamo – cold case with an average yearly temperature of 11.9°C.
— Florence – medium cold case with an average yearly temperature of 14.2°C.
— Rome – medium warm case with an average yearly temperature of 15.3°C.
— Catania – warm case with an average yearly temperature of 17.1°C.

The reason that Italian cities were preferred to Greek ones, although Greek seismic codes were used, was mainly due to the better availability of EnergyPlus weather files. After all, it could be presumed that Greek, Italian and generally south European RC framed buildings of the same era have similar seismic resistance.

As an example, for the case of the 5-storey building, located in Florence before the retrofitting, the EnergyPlus analysis yielded the results shown in Table 8.

Table 8 Results of EnergyPlus analysis for 5-storey building, located in Florence

Heating needs	170473 kWh
Cooling needs	19512 kWh
Other equipment needs	78851 kWh
Total energy consumption	268835 kWh
Consumption per unit area	224.0 kWh/m^2

The primary heating and cooling energy needs are shown in Figure 34. Note that these are *not* consumptions, as in Table 8, but only the necessary thermal loads needed to maintain the thermal balance during each month of the year. As expected, heating needs are higher during the winter and cooling needs during the summer.

Lastly, Figure 35 shows the fluctuation of the indoor and the outdoor temperature during the year, as obtained by the weather file used in the analysis. Again as expected, the monthly average outdoor temperature ranges between 5°C and 24°C, while the indoor is always between 20°C and 25°C, as it is regulated by the HVAC system.

Figure 34. Heating and cooling energy needs, 5-storey building, Florence

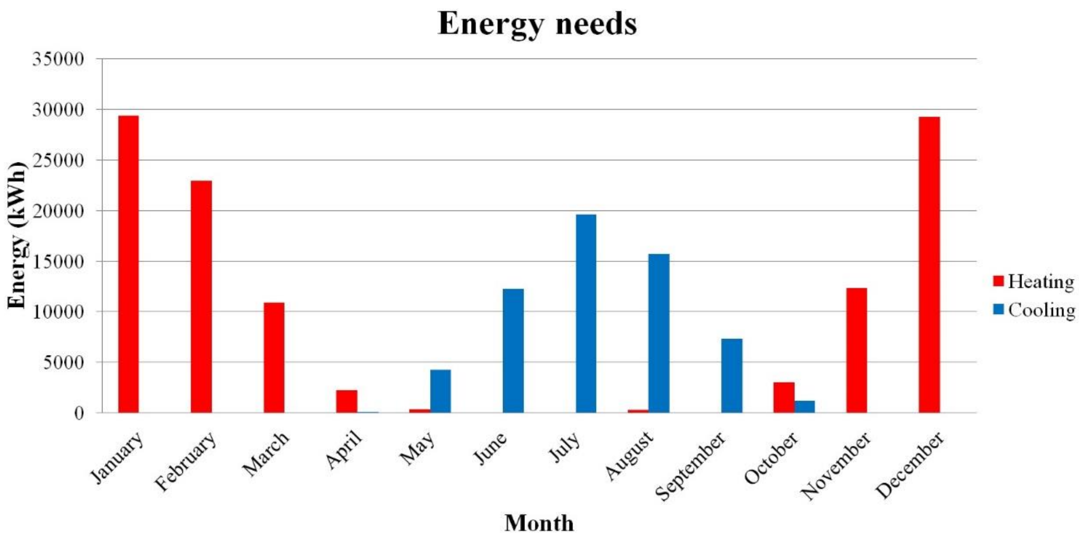
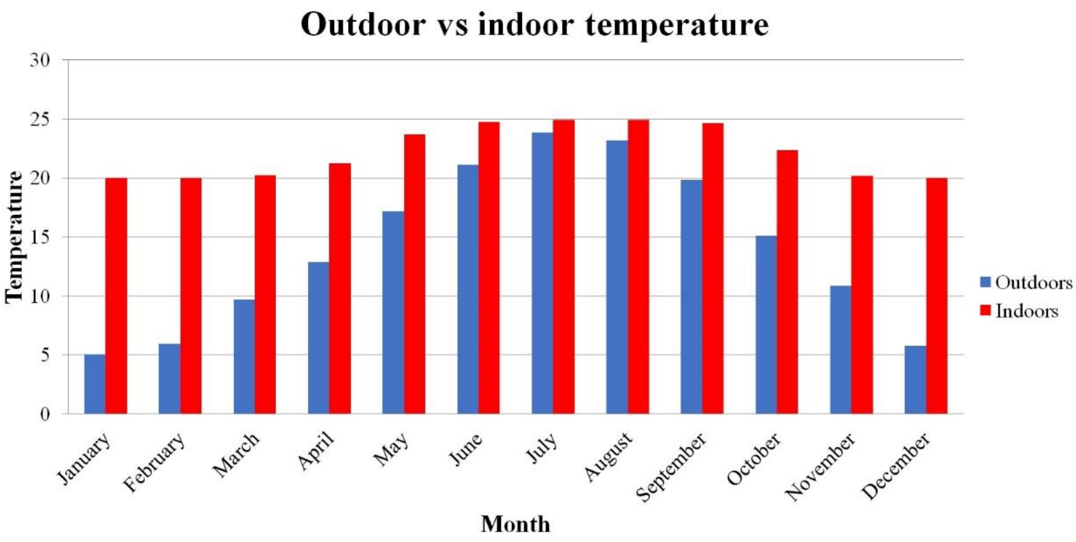


Figure 35. Indoor and outdoor temperature, 5-storey building, Florence



3 Economic Losses Estimation

Undoubtedly, the structural and energy analyses are an essential part of the engineer's work when it comes to the assessment and retrofitting of existing structures. However, if applied without any further processing of their results from the economic point of view, they cannot provide adequate data for the deciding whether retrofitting is financially feasible. For that matter, a sort of economic analysis has to be conducted and both the structural and the energy performance of the examined building need to be somehow "translated" into economic measures. Then, the retrofitting costs, considering a number of different upgrading solutions, have to be estimated and the most profitable one to be selected.

Obviously, the above include firstly the determination of the seismic and energy losses of a given structure. This is an easy task to do when it comes to the energy losses, since the only extra information required to determine them reliably, is the energy prices (for electricity, oil, gas etc.) of the country the building is located in. The same does not apply however, for earthquake loss. Apart from the difficulty of "connecting" somehow these losses with engineering parameters, which are provided by the structural analyses software, there is always the question of how reliable these economic measures will be.

In the following sections, the loss estimation process is described in detail for both cases, earthquake and energy losses. Then, a simplified method for approximating the retrofitting costs is presented, as well as the payback time for each retrofitting scheme examined. Finally, the results from the case studies considered in this report are presented in detail.

3.1 Estimating the losses

For the economic classification of an existing structure, the idea of the *Expected Annual Loss* (*EAL*) has prevailed, which is simply the money that the given building loses during a year. This measure is often expressed as a percentage of the structure's total value, thus an *EAL* of 1% means that, each year, the building loses 1% of each initial value.

A simple approach for the integrated assessment of energy efficiency and earthquake resilience was recently presented by various researchers (Calvi at al. 2016, Mastroberti et al. 2018, Bournas 2018b). Following an identical approach the current study considers the total EAL as the sum the annual energy consumption multiplied by their relevant energy unit costs (denoted as EAL_e) and the expected annual seismic loss (denoted as EAL_s) leading to the simple formula:

$$EAL_t \cong EAL_e + EAL_s \quad (3.1)$$

Whereas Eq. 3.1 assumes that EAL_e and EAL_s are uncoupled, in reality under seismic loading damage is also expected to occur to the thermal envelope of the building. However, since the EAL_s is mainly affected by the lower intensity earthquakes, as we will demonstrate later, it is not expected that this approximation will have any negative impact on the final output.

3.1.1 Seismic loss

The estimation of the seismic loss is undoubtedly a difficult task to carry out reliably, although a number of techniques have been proposed over the years from the researchers worldwide. What is very important to understand, is that this task is closely related to the structural analysis methods used to characterise the building's earthquake resistance. One could, for example, use a method that implies simple static pushover analyses or a totally different one that demands first the completion of advanced time-history simulations using detailed finite element models.

In this work, the earthquake simulation process included a large number of non-linear time-history analyses (*NLTH*), run on a rather simplistic finite element model. Therefore, the procedure to obtain the earthquake losses in our case should address the specific nature of the structural analysis that preceded this step. Using, for example, a highly advanced method to obtain these direct losses would be completely unnecessary, as the structural model itself would not be able to provide that very method with the input needed and even if it could, that input would be at least unreliable. For that exact reason, it was decided to use the inter-storey drifts as those engineering parameters that would help us *connect* the analyses' results with their respective economic losses. This decision was made based on the fact that inter-storey drifts are able to capture structural damage in a global level, as was needed in our case, where the finite element models were not so much refined to consider the structural damage element-wise.

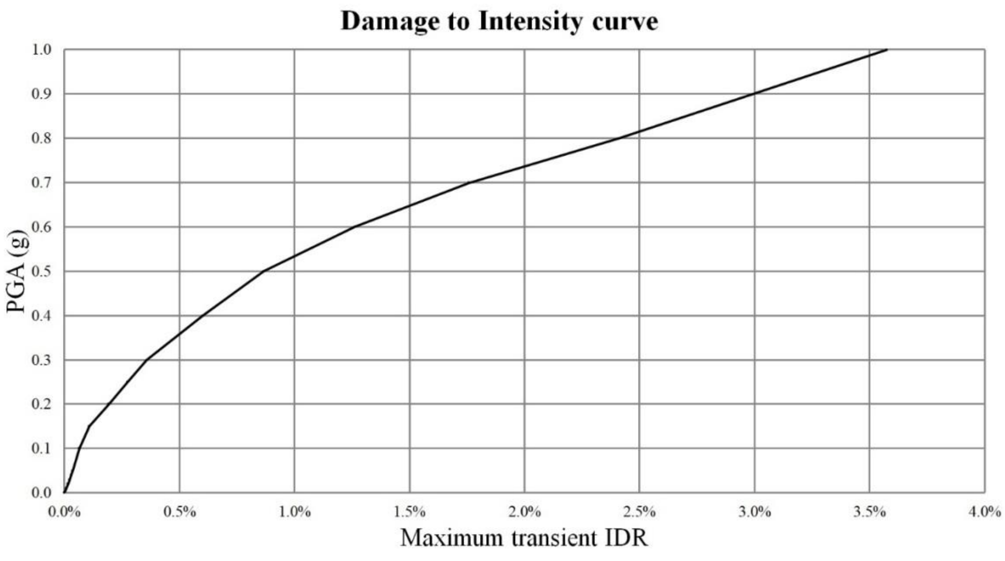
The general procedure followed to obtain the earthquake losses is thoroughly explained in the paragraphs that follow and comprises of the step-wise process presented next:

1. From the *NLTH* analyses, create a curve correlating the damage of the structure with the seismic intensity.
2. Define a "Damage to Loss" function using an existing model.
3. Using 1 & 2, construct the "Loss to seismic intensity" curve for the structure (Vulnerability curve).
4. Combine 3 with the seismicity of the region to obtain the "Annual Probability of Exceedance vs Loss" curve and integrate it to get the EAL_s.

3.1.1.1 Step 1

Using the maximum recorded inter-storey drift as the selected *Damage Measure* (*DM*) and the *PGA* as the respective *Intensity Measure* (*IM*), we can easily create the curve needed for the first step. This is the median IDA curve that was presented earlier in section 0 and is shown in Figure 33. Such a curve is repeated here with Figure 36 for the sake of completeness.

Figure 36. Structural damage vs Earthquake Intensity curve



3.1.1.2 Step 2

Creating a "Damage to Loss" function requires the correlation of the selected damage measure – the maximum transient inter-storey drift – with an economic loss amount. In other words, this function should answer how much is the economic loss for a structure given a specific value of that same damage measure. Obviously, answering such a question, given only a single number, is quite complicated, there are however simplified ways to achieve it.

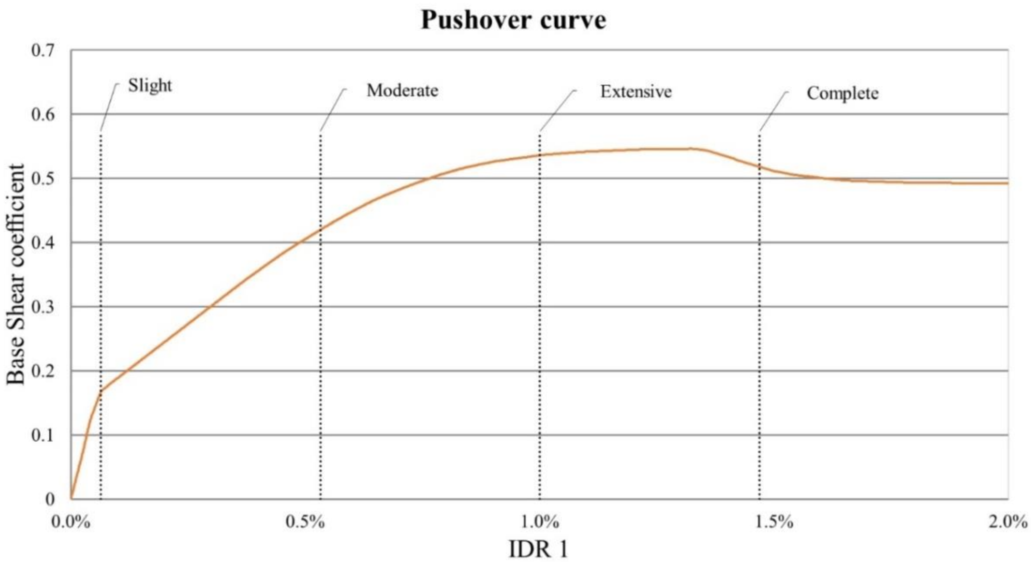
In this work, the idea of the damage states was selected as a means to accomplish that very connection. Specifically, the well-known damage states of HAZUS (1999) were used, which are given next in Table 9. These states were initially created for the United States but were later adopted for the European area as well. Moreover, they are supposed to include both structural and non-structural damage and despite their simplicity, they have been found to give very satisfactory results, even when compared with more complex methods.

Table 9 Damage states used in HAZUS99 (1999)

Damage state	Repair/Replacement cost
Slight	2%
Moderate	10%
Extensive	50%
Complete	100%

The obvious drawback when using such damage states is that they are described in a qualitative way, thus some further processing is needed to correlate these qualitative terms with values of the selected damage measure. This processing was done here by running pushover analyses and trying to "locate" these damage states on the pushover curve, therefore map each damage state to a specific value of the inter-storey drift. Figure 37 shows such a curve, which has the drift of the first floor on the horizontal axis – this was the largest one – and four vertical lines at specific drift values, which are supposed to represent the four damage states. The positioning of the vertical lines was done following the procedure explained hereafter.

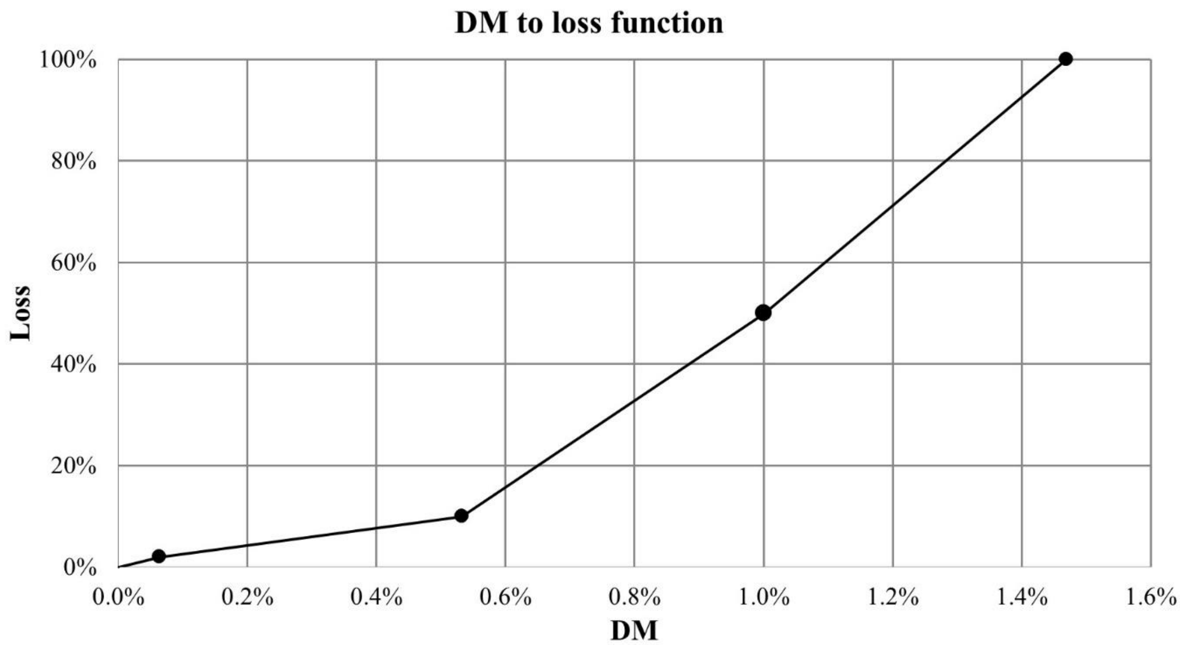
Figure 37. Damage states *on* the pushover curve



— The *Slight* damage is assumed to be at the point where the structure's linearity ends.
— The *Complete* damage is positioned after the maximum recorded force, at the point where the force is 95% of the maximum.
— The *Moderate* and *Extensive* damage states are evenly distributed in the space between the 2 extreme lines.

Having mapped the qualitative damage states to specific values of the damage measure, we can easily construct a continuous function that gives the loss for any value of that damage measure. Such a curve is shown is Figure 38. The four black dots represent the four damage states presented earlier.

Figure 38. Damage to Loss curve



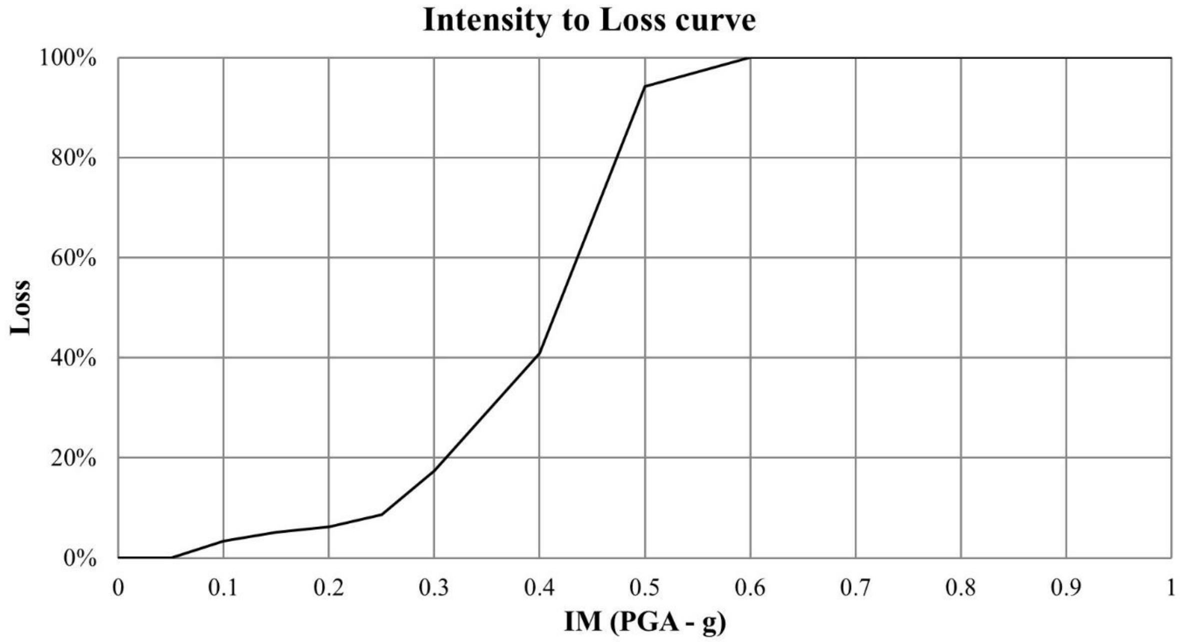
3.1.1.3 Step 3

The "Damage to Intensity" curve constructed in the first step and the "Damage to Loss" function created in the second have the same horizontal axis, that is the maximum transient inter-story drift or in other words, the selected damage measure (DM). Therefore, it is very easy to combine these two curves and construct a new one that will connect directly the *PGA* (the selected intensity measure or IM) with the earthquake losses. Such a curve is given in Figure 39.

Such a function resembles the vulnerability curves that are widely used in this field of earthquake engineering with one difference however. The curve presented here is building-specific, as opposed to the traditional vulnerability curves, which actually are smooth, statistical functions that represent a family of buildings.

What is more important though, is that such a curve, once constructed, can be used directly for the seismic categorisation of the building examined. That could be done for example by computing – using the *PGA* of the area of the building – the expected loss and then classifying the structure according to that loss, posing at the same time an upper acceptable limit, above which seismic strengthening would be compulsory. Such a seismic categorisation method would be more meaningful and give more insight to the engineer, than using just a single EAL_s value which is incapable of describing the broader picture.

Figure 39. Earthquake intensity to Loss curve



3.1.1.4 Step 4

The last step in calculating the economic losses due to earthquake is to combine the previously computed building-specific "Vulnerability curve" with another function capable of representing the building location's seismicity. Such a function is the "Annual Probability of Exceedance vs IM" curve that gives for every value of *PGA*, the annual probability of an earthquake of such or larger *PGA* occurring in the building's location.

To construct this function, the Eurocode 8 recommendation was adopted (Eurocode 8). According to this code, at most sites the annual rate of exceedance, $H(a_{gR})$, of the reference peak ground acceleration a_{gR} may be taken to vary with a_{gR} as is given by equation 3.2. In this equation the value of the exponent k depends on seismicity but is generally of the order of 3.

$$H(a_{gR}) \cong k_0 a_{gR}^{-k} \tag{3.1}$$

Parameter k_0 can be computed easily if the a_{gR} that corresponds to a probability of exceedance of 10% in 50 years is known – these values are given in the national annex of each country for every location. Because of this equation tending infinity as a_{gR} tends to zero, it is assumed that the maximum value that $H(a_{gR})$ can take is 10%, thus resulting in zero loss for an earthquake that has an annual frequency of 10% or more. Such an assumption is very reasonable and is adopted by many researchers on the topic. An example curve constructed this way is given in Figure 40; note that the limit of 10% results in a plateau.

Now this function along with the one constructed during step 3 (see Figure 39), also have the same horizontal axis. As a result, we can also combine them and create a new one that has the Loss on the horizontal axis and the Annual probability of exceedance on the vertical one. The resulting curve, which is also the final outcome of the 4-step process, connects the annual probability of exceedance of an earthquake with the economic loss that the building will confront. Such a curve is given in Figure 41; note that the starting point on the vertical axis is 10% as explained earlier.

Figure 40. *PGA* to annual probability of exceedance function

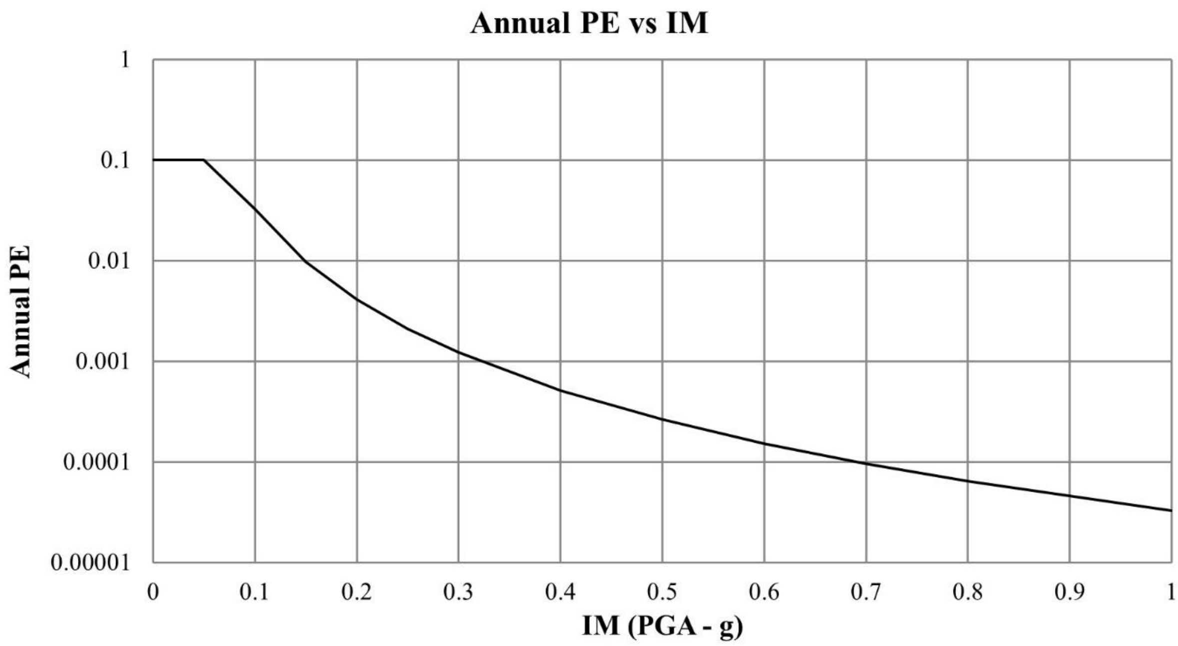
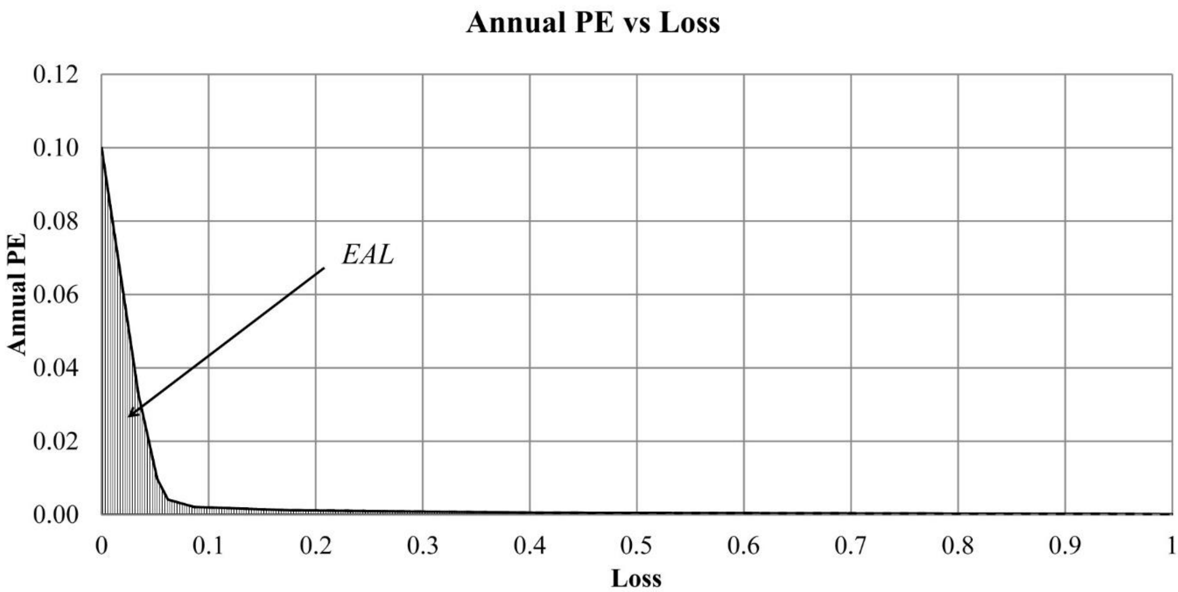


Figure 41. Loss to annual probability of exceedance function

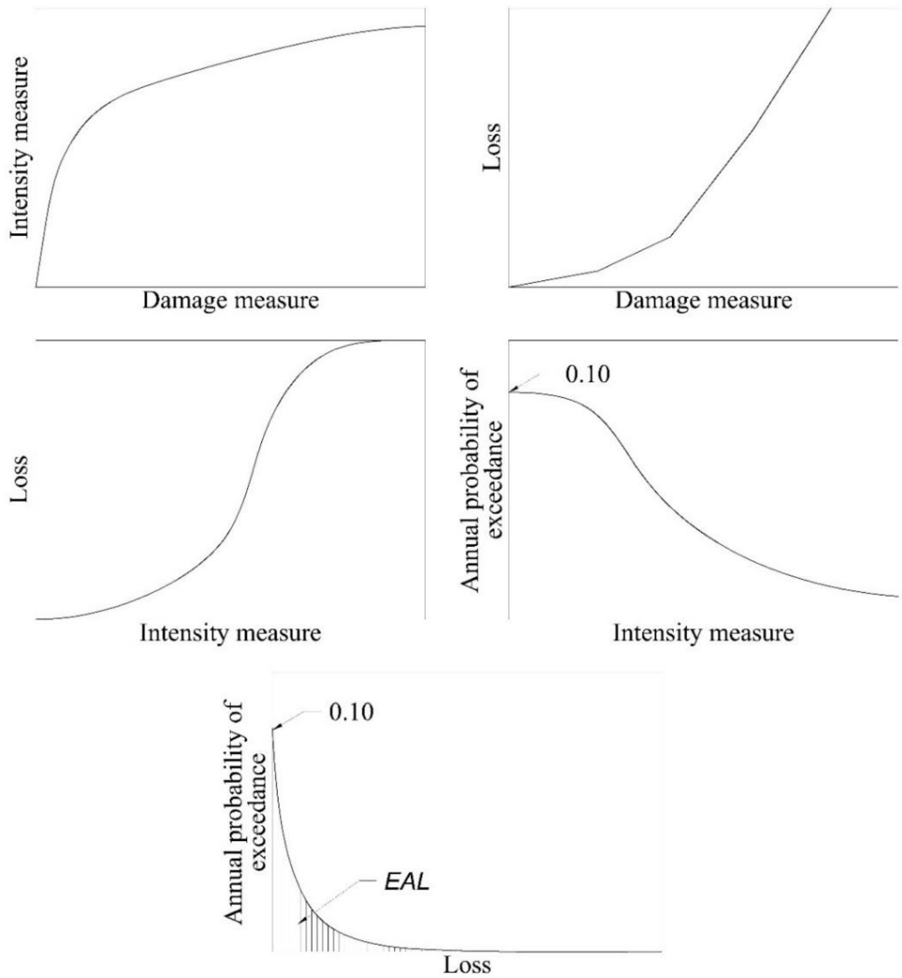


Having reached this point, the evaluation of the *Expected Annual Loss* due to earthquake, comprises of simply integrating this function from zero to infinity. The result of this computation will be the annualised losses due to earthquakes striking the building's location, expressed as a percentage of the structure's total value. It is also evident at this point, that the final value of the EAL_s is affected more from lower-intensity earthquakes that have a higher rate of occurrence rather than extreme events that are unlikely to happen. Therefore, the assumption made in equation 3.1 can be regarded as correct.

3.1.1.5 Remark

The above explained procedure can successfully provide us with the parameter EAL_s, as was our initial intention. However, it should be noted that a number of assumptions have been made so as to simplify the whole process and reach the final result. Therefore, more attention should be paid on the comparisons that are going to be made between the different case studies, rather than the absolute values of the EAL_s themselves. In any case, as it was also mentioned at the start of this chapter, evaluating the EAL_s with a high degree of reliability is a very difficult thing to do. It is true though, that even by making a number of assumptions so as to ease this task, some interesting results can be drawn using this controversial economic loss measure. Figure 42 shows briefly the complete procedure followed to obtain the EAL_s.

Figure 42. Procedure to compute EAL_s



3.1.2 Energy loss

Estimating the energy loss of a building is a much easier and more straightforward task to carry than evaluating the earthquake loss. In most cases, the software used to conduct the energy analyses, could provide to the designer the total needs for heating, cooling, hot water and electricity. Theses energy needs are often expressed by their total amount of $kWhs$ necessary in the period of a year. Having these values, the final energy cost evaluation for the structure for one year – thus the EAL_e – simply comprises of the multiplication of these energy values with their respective unit cost in $€/kWh$. To go one step further and express the annual energy cost as a percentage of the structure's value, it is obvious that this value needs to be known.

In the context of this report, where EnergyPlus was used for all the energy simulations, the results of the analyses were the total number of the necessary $kWhs$ for each different energy need (heating, cooling, and electricity – includes the hot water needs). Since Italian cities were used as the reference locations for the case studies, the Italian energy prices for natural gas and electricity were used. According to the European Commission Website, these prices are given in Table 10[1]; only gas and electricity are of our concern in our case.

Table 10 Energy prices in Italy

Energy type	**Unit price**
Heating oil	$0.108\ €/kWh$
Natural gas	$0.084\ €/kWh$
Electricity	$0.234\ €/kWh$

As far as the building values are concerned – as they are necessary for the evaluation of the EAL_e as a percentage – these were taken from the website immobiliare.it. According to its data, the mean building prices for the selected Italian cities are given in Table 11.

Table 11 Building prices in Italian cities

City	**Building values**
Bergamo	$2132\ €/m^2$
Florence	$3610\ €/m^2$
Rome	$3111\ €/m^2$
Catania	$1400\ €/m^2$

At this point, it is important to note that, when it comes at assessing a building from its energy point of view, the EAL_e as a percentage can give a false impression. Consider the case where two buildings are compared, building *A* located in a very expensive location (e.g. Florence) and *B* in a much cheaper one (e.g. Catania), with half the real-estate prices. Assume also, that building *B* consumes less, say half, energy per square metre than *A*. Obviously, building *B* is more energy efficient, however both *A* and *B* will have the same EAL_e value because of their different values. Therefore, when the goal is the energy classification of a structure, then using the EAL_e as a percentage of its total value can lead to paradox results. On the other hand, a pure energy-based classification with the total consumption is much more reliable.

[1] Heating oil price is actually $1.157\ €/L$. The price in $€/kWh$ is evaluated assuming that heating oil contains approximately $10.72\ kWh/L$.

3.2 Retrofitting costs

In order to decide whether a retrofitting technique is economically feasible or when two or more of them need to be compared, the retrofitting costs have to be computed. Again, there are many levels of approximation to that evaluation, ranging from using simple global values (cost per m^2, per floor etc.) up to considering each and every detail of the retrofitting scheme and computing accurately the final cost. Of course, a rougher approximation is used at the first stages of the design process and more accurate calculations are repeated later, as is the case with every engineering procedure.

Taking into account the general level of approximation used in earlier stages (structural modelling, loss evaluation etc.) of this work so far, it would be unwise to try to compute the retrofitting costs in a highly detailed manner. For that reason, it was decided to adopt similar retrofitting costs with those presented by Mastroberti et al. 2018 and Bournas et al. 2018b, which were estimated after contacting engineers of the practice in Greece and Italy, and which are illustrated in Table 12.

Table 12 Retrofitting costs

Retrofitting scheme	**Cost**
Energy retrofitting	$80\ €/m^2$
Seismic retrofitting	$60\ €/m^2$
Integrated approach	$105\ €/m^2$

With reference to Table 12, the energy retrofitting cost comprises of both adding external insulation and substituting the fenestration surfaces. On the other hand, the seismic retrofitting cost takes into account the TRM wrapping of the outside walls of a structure and is *only* applied to those floors that are being retrofitted. However, probably the most important conclusion that is drawn from Table 12, is that the integrated retrofitting is 25% cheaper than the energy and the seismic applied at different times. This happens mainly because certain expenses like labour, scaffolding etc. are paid only once.

3.3 Case studies run

In this sub-chapter, the results from structural and energy analyses with OpenSees and EnergyPlus, respectively are presented. Last but not least, for each case examined, the pay-off time is evaluated in order to economically assess the outcome of the upgrading techniques.

3.3.1 Earthquake analyses

The procedure described in section 0 was followed during the seismic modelling process and that of subsection 3.1.1 for the extraction of the EAL_s values. For the three case studies considered, Table 13-15 contain respectively these EAL_s values both before and after the application of the seismic retrofitting scheme with TRM.

As it can be seen in Tables 13-15, the structural retrofitting does decrease the EAL_s values. The reduction is small for sites with low seismicity and increases in more seismically active areas. In order to understand the magnitude of the decrease better, let us suppose that the buildings' value per unit area is $2500\ €/m^2$, therefore the 2-storey structure is worth $1200000\ €$ and the 5-storey one $3000000\ €$. Supposing that the area they are located has a *PGA* of 0.3g (probability of exceedance 10% in 50 years or return period 475 years), then in absolute values, each year the 2-storey building will save

2640 €, the 5-storey 5100 € and the 5-storey with pilotis 84000 €, after applying the proposed strengthening scheme. One therefore should not be mistaken by the phenomenally low EAL_s values when expressed as percentages.

Table 13 EAL_s values before and after retrofitting – 2-storey building

Site seismicity	EAL_s- % initial	EAL_s- % retrofitted
0.1	0.16	0.07
0.2	0.26	0.14
0.3	0.43	0.21
0.4	0.66	0.33
0.5	0.93	0.47

Table 14 EAL_s values before and after retrofitting – 5-storey building

Site seismicity	EAL_s- % initial	EAL_s- % retrofitted
0.1	0.21	0.15
0.2	0.35	0.28
0.3	0.63	0.46
0.4	0.93	0.67
0.5	1.27	0.89

Table 15 EAL_s values before and after retrofitting – 5-storey building with pilotis

Site seismicity	EAL_s- % initial	EAL_s- % retrofitted
0.1	0.30	0.17
0.2	1.35	0.29
0.3	3.23	0.43
0.4	4.66	0.62
0.5	6.64	0.82

It is also worth noting, that the initial 5-storey building with the pilotis configuration has extremely higher EAL_s values than the other two cases. As a result, once more it is shown, that pilotis configurations can be very dangerous when combined with under-designed old structures. Table 16 contains altogether the savings in absolute values for the three cases assuming the retrofitted buildings are located in a location with 2500 €/m^2 price.

Table 16 Money savings after strengthening with TRM for all cases

Site seismicity	2-storey	5-storey	5-storey pilotis
0.1	1080 €	1800 €	3900 €
0.2	1440 €	2100 €	31800 €
0.3	2640 €	5100 €	84000 €
0.4	3960 €	7800 €	121200 €
0.5	5520 €	11400 €	174600 €

3.3.2 Energy analyses

The energy analyses were conducted as explained in section 2.3 and their loss evaluation followed the procedure describe in sub-section 3.1.2. The energy consumptions in kWh/m^2 for the three selected case studies and the selected different locations are given in Table 17.

Table 17 Annual energy consumptions in kWh/m^2 for all cases

City	2-storey		5-storey		5-storey pilotis	
	Initial	Insulated	Initial	Insulated	Initial	Insulated
Bergamo	343.9	187.8	280.3	168.4	338.9	182.4
Florence	275.6	156.3	224.0	139.8	267.3	149.6
Rome	226.1	131.0	183.6	118.1	217.6	125.3
Catania	174.7	107.6	143.3	99.6	166.3	103.9

As it can be seen, the selected retrofitted scheme can effectively reduce the energy consumption of the buildings considered. This reduction varies between 30% and 46% and could be even higher if a more extended retrofitting scheme had been employed (e.g. thicker insulation material, replacement of the mechanical equipment). Since the energy costs are known (see Table 10), the absolute costs can be computed (Table 18) and then the EAL_e using the known building values (Table **19**).

At this point, it is worth noting what was also mentioned in section 3.1.2. If we consider the case of the 2-storey building, we see that its consumption, when located in Bergamo is roughly double the one that it has when located in Catania ($343.9\, kWh/m^2$ versus $174.7\, kWh/m^2$). However, since the prices in Bergamo are 52% higher than those in Catania, the resulting EAL_e for the two cases are very close and actually, that of Catania is even higher. It is evident therefore, that the EAL_e is *not* a suitable measure of a buildings energy efficiency.

Table 18 Annual energy costs in € for all cases

City	2-storey		5-storey		5-storey pilotis	
	Initial	Insulated	Initial	Insulated	Initial	Insulated
Bergamo	19250 €	12769 €	42120 €	30848 €	38629 €	26229 €
Florence	16830 €	11690 €	37337 €	28588 €	33755 €	24203 €
Rome	14983 €	10739 €	33596 €	26658 €	30089 €	22502 €
Catania	13198 €	9971 €	30209 €	25343 €	26658 €	21334 €

Table 19 EAL_e values (%) before and after retrofitting

City	2-storey		5-storey		5-storey pilotis	
	Initial	Insulated	Initial	Insulated	Initial	Insulated
Bergamo	1.88	1.25	1.65	1.21	1.51	1.03
Florence	0.97	0.67	0.86	0.66	0.78	0.56
Rome	1.00	0.72	0.90	0.71	0.81	0.60
Catania	1.96	1.48	1.80	1.51	1.59	1.27

As far as the actual savings are concerned, these are highly dependent on the building's location. As anyone would expect, when a building is located in harsher, colder climates, its energy bills are higher, as well as the respective savings in case thermal upgrading is applied to it. That is why, for the case of the structure being located in Bergamo, the actual savings are considerably higher – the application of the proposed energy retrofitting scheme to the 5-storey building will save 11272 €, if it were in Bergamo and 4866 €, if it were in the much warmer Catania. Table 18 contains the actual savings in € for all the cases.

Table 20 Money savings after energy upgrading for all cases

City	2-storey	5-storey	5-storey pilotis
Bergamo	6481 €	11272 €	12400 €
Florence	5140 €	8749 €	9552 €
Rome	4244 €	6938 €	7587 €
Catania	3227 €	4866 €	5324 €

3.3.3 Pay-off time evaluation

One very important characteristic of each retrofitting scheme, which defines whether it is economically feasible or not, it's the pay-off time. This is simply the time needed for the owner to take back their initial investment, considering the yearly savings of the applied scheme. This parameter can also be used for the economic comparison of different retrofitting techniques and help the engineer decide which one to implement, provided that all the examined choices guarantee the almost same level of enhancement, of course.

In this case, the pay-off time was computed for the three building cases combined with each of the four locations used for the thermal analyses, yielding 12 combinations of building-location. For the most accurate evaluation of the seismic loss, the actual four locations' *Peak Ground Accelerations* were used (with return period 475 years as defined in Eurocode 8). These values were taken using the on-line Interactive Seismic Hazard Maps of Italy, a WebGis application, developed by the "National Institute of Geophysics and Volcanology", that can provide *PGA* values (and many other information as well) for the whole Italian territory. Table 21 contains the PGAs for the four Italian cities considered.

Table 21 PGA values for the Italian cities used in the case studies

City	*PGA*
Bergamo	0.11g
Florence	0.13g
Rome	0.14g
Catania	0.21g

The pay-off times for the 36 combinations (city-building-retrofitting scheme) were evaluated using the results of the earthquake and energy analyses presented earlier. Moreover, the retrofitting costs described in section 3.2 were used. Table 22 contains the pay-off times for the energy, seismic and the integrated retrofitting scheme. the notation selected for the 2-storey, 5-storey and 5-storey with pilotis buildings are 2s, 5s and 5sp, respectively.

Table 22 Pay-off times in years for each city-building-scheme combination

Retrofitting Scheme	Bergamo			Florence			Rome			Catania		
	2s	5s	5sp	2s	5s	5sp	2s	5s	5sp	2s	5s	5sp
Energy	5.9	8.5	7.7	7.5	11.0	10.1	9.0	13.8	12.7	11.9	19.7	18.0
Seismic	30.1	34.2	12.4	17.2	18.3	4.1	18.2	19.8	3.8	33.8	43.1	3.0
Combined	6.8	9.3	**7.0**	**7.4**	**10.1**	**5.1**	**8.6**	**12.2**	**5.3**	12.4	**19.4**	**4.9**

Table 22 reveals a very important aspect of the retrofitting schemes, once we compare them to each other. Specifically, consider the case of energy retrofitting alone and that of the integrated approach, since this is the comparison that makes the most sense. In all the columns where the last cell is painted, the integrated retrofitting scheme has a *shorter* pay-off time than that of the energy retrofitting scheme. In other words, the initial investment made by the building owner will be returned *faster*, if they upgrade their property both energetically and seismically rather than enhancing it only in terms of its energy efficiency.

Of course, the integrated retrofitting scheme demands a somewhat higher initial investment. According to this study, for the 2-storey building, the energy retrofitting scheme needs 38400 €, while the integrated 50400 € (31% more expensive). For the 5-storey, the respective values are 96000 € and 120000 € (25% more expensive). Therefore, taking into account the faster return period, it is highly worth it, for any old building owner, to invest a slightly larger initial amount of money and apply an overall retrofitting to their structure. That way, the upgraded building will be much safer and economical to live in and of course, will have a much smaller energy footprint.

4 Conclusions

In the final chapter of this report, the most important conclusions that can be drawn from its results are presented. First, the performance of the TRM strengthening technique is assessed after its testing in a close to real-world situation. Furthermore, the efficacy of the integrated seismic and energy retrofitting scheme is evaluated based on the results of the present case studies. Last but not least, a number of recommendations for future work on the topic are given, so as to further increase the scientific knowledge on the promising topic of the combined seismic and energy upgrading of existing structures.

4.1 Assessment of the seismic retrofitting scheme

From the results obtained in this work, it was found that the proposed strengthening method can indeed enhance the seismic performance of old, non-seismically designed RC buildings, reducing their damage during earthquake events. Using the EAL_s approach, it was found that a 5-storey, RC building with masonry infill walls, located in a high seismicity area (*PGA* 0.3g with return period 475 years), can save up to 5100 € *every year* after receiving TRM jacketing of the infills. Moreover, it was found that, as expected, the seismic retrofitting effectiveness increases or decreases for areas of higher and lower seismicity, respectively. It is also worth noting, that this technique can be applied with great success in RC buildings with a pilotis configuration. Simply by closing with TRM-reinforced infills specific frames, soft-storey mechanisms can be fully avoided and the overall behaviour of the structure can be enhanced.

A last, very important aspect of the TRM strengthening technique, which was not able to be examined in the context of this work, is the dramatic increase of the lateral capacity of the infill, as demonstrated in a very recent study by Koutas and Bournas 2018. Moreover, experiments that were conducted on small wallette elements have already shown the great increase that TRM can bring about to the out-of-plane capacity of such elements, from practically zero to a substantial amount (Papanicolaou et al. 2007, Kariou et al. 2018). It is therefore reasonably expected, that the proposed scheme will also bring about a similar enhancement to the out-of-plane behaviour of the infill walls in masonry infilled RC structures. The importance of this fact is extremely high, if one takes into account that in many cases, it is the falling debris from the destroyed masonry walls that endanger people's lives during large earthquake events.

4.2 Effectiveness of the integrated approach

Another question that this report wanted to answer was whether the integrated seismic and energy rehabilitation of existing RC buildings is an effective approach. The results of the case studies showed that in most cases, actually it is economically more effective to follow the integrated approach than upgrading a building only in terms of energy efficiency or seismic resistance, as the initial investment is paid back faster. Specifically, for deficient RC buildings located in seismic areas (say *PGA* > 0.10g), the integrated approach should be preferred because:

— There are significant savings in money terms as the labour and scaffolding expenses are paid only once. This results in a 25% lower initial investment than the case of applying the seismic and energy upgrading separately.

— The building is “armoured” against future seismic events and the energy investment is safe. Otherwise, if energy rehabilitation had been applied only, then a possible moderate to strong earthquake would in addition to the structural damages, have made this investment practically useless.

— Using the EAL approach, it demonstrated that the integrated retrofitting is becoming progressively economically more efficient, as the seismicity increases..

4.3 Recommendations for future work

Clearly, the topic discussed in this report is quite complex and cannot be addressed only through those aspects presented here. In this context and understanding the limitations of this work are presented and a number of recommendations are made. These concern mainly the subject of the integrated seismic and energy retrofitting approach, as well as the infill wall strengthening with TRM method used before and are given in the following list:

- The idea of the integrated retrofitting approach needs to be further tested through additional case studies, using more advanced (and more detailed) structural models and loss evaluations techniques. Different retrofitting methods could be also considered.
- Testing of the integrated retrofitting approach on masonry structures as well, since there is a great number of buildings built this way too.
- Development of a structure categorisation method using the "Seismic Intensity to Loss" curve and definition of the minimum acceptable limits.
- Development of a refined finite element micro-model for the TRM-strengthening technique to make possible more detailed finite element analyses.
- Experimental testing of TRM-strengthened infill walls with openings and development of an analytical model.
- Experimental testing of the out-of-plane capacity of TRM-strengthened infill walls and development of an analytical model.

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Annexes

Annex 1. Greek pre-1985 seismic provisions

For the initial design of the structures used in the case studies, the building codes active in Greece before 1985 were used, namely the following three decrees:

— Decree "Regulation for loads in structures", Official Gazette A325, 31/12/1945.

— Decree "Regulations for the design and building of reinforced concrete structures", Official Gazette A160, 26/07/1954.

— Decree "Seismic provisions for structures", Official Gazette A36, 26/02/1959.

Load combinations

According to the first decree, apart from the permanent loads, a live load of $200\,kg/m^2$ was considered for all the slabs. Regarding the seismic loads, according to the third decree, those were modelled as a static lateral load evenly distributed over the building's floors. The magnitude of that load was 6% of the building's weight, assuming medium seismicity and good soil conditions. For the analysis of the structures, three load combinations were considered, $G+Q$, $G+Q+E_x$ and $G+Q+E_y$ – the first for the vertical loads only and the other two including the lateral load.

Detailing rules

The detailing regulations as described in the second and third decree are presented in the following list.

— The corner rectangular columns must be at least $350 \times 350\,mm$ and have a longitudinal reinforcement of at least $A_{s,min} = 12cm^2$ or 0.8% of the cross-section area.

— Columns in general must have at least 5‰ longitudinal reinforcement, if their aspect ratio is less than 5 and 8‰ if it is above 10. For intermediate values, linear interpolation shall be used.

— For columns subjected to bending with axial force, the longitudinal reinforcement of the least compressed side must be at least 4‰ of the section.

— The maximum amount of longitudinal reinforcement is 3% for concrete qualities B120 and B160.

— The minimum dimension of a rectangular column is $250\,mm$ and the minimum diameter of the longitudinal reinforcing bars is $14\,mm$.

— The concrete cover up to the reinforcement must be at least $15\,mm$ for the slabs and $20\,mm$ for the rest bearing elements.

— The spacing of the transverse reinforcement must be less than the smallest dimension of the cross-section and less than 12Φ (Φ is the smallest longitudinal bar diameter).

Allowable material stresses

The structural detailing at that era was done using the maximum allowable stresses method. These maximum allowable stresses are:

— For the concrete of slabs $60\,kg/cm^2$.

— For the concrete of beams $50\,kg/cm^2$.

— For the concrete of columns $70\,kg/cm^2$ for uniaxial bending and $80\,kg/cm^2$ for biaxial bending.

— For category I steel $1400\,kg/cm^2$ and for category II $2000\,kg/cm^2$.

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