SEISMIC RETROFIT OF BUILDINGS
A GREEK PERSPECTIVE

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ABSTRACT

Some important aspects of the subject of the repair to and the strengthening of buildings, before and after an earthquake, are investigated in this paper. It has to be noted that, far more than that of recent constructions, many older buildings are susceptible to earthquake damage. It is certain that, from the techno-scientific side of problem alone, the subject is incomparably more difficult and complex than that of designing new structures. The structure of the building has to be dealt with as a whole and the process of redesign includes the following three stages: The assessment of the capacity of the existing structure during seismic activity, the process of decision making and the design of a solution for a pre-selected performance level. The method that will be selected for the intervention will have to be included in the plan of the strategy. This will depend on the required levels of strength, stiffness and deformation of the structure. Practical aspects for six main categories of methods of intervention are discussed in this paper. The six main categories are listed as follows: The addition of infilled walls, the addition of new external walls, the addition of bracing systems, the construction of wing walls, the strengthening of weak elements and the incorporation of energy absorbing systems. In particular, recent results from ongoing experimental research programs concerning the retrofitting of existing structures are presented. Finally, certain important aspects of practical application are pointed out that may often puzzle the engineer or may be solved in an erroneous way. These concern either the choice of the most suitable solution for the intervention or the application of technological methods of intervention.

Keywords: Repair, Strengthening, Retrofitting, Earthquake, Seismic.

1. INTRODUCTION

It is irrefutable that our knowledge of the seismic behaviour of buildings is tried every time a forceful earthquake occurs. Certain older beliefs are confirmed and certain others are refuted. "Lessons" from each earthquake, in combination with new technologies and continuing research in the corresponding scientific field, have led to changes in the relevant guidelines, codes and specifications. Consequently, it becomes clear that, for each new construction, the possibility a more equitable and secure strategy exists.

Simultaneously however, the following question justifiably arises: What should happen to the buildings that have been designed and constructed in the past? Conjecture studies have been performed at the University of Patras for the assessment of reinforced concrete buildings that were designed and constructed in Greece before the application of additional clauses in the seismic code (Greek Ministry of Environmental Planning and Public Works, 1985). These studies have shown that a large number of older buildings are in danger of experiencing serious damage, or even collapse, during a strong earthquake. The consequences of devastating earthquakes in Greece in recent years have confirmed the above estimation.

It is sufficient to contemplate that for buildings constructed before 1985 (or before the code revision of 1995) the following points can be noted:

a) Older buildings have been designed for seismic actions that correspond to approximately 50% of that of newer buildings,

b) The design of the load bearing structure often followed architectural lines without restriction and was frequently indifferent to aspects of regularity in geometry, or stiffness in both plan and elevation,

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c) The determination of action effects on structural elements included simple omissions due to a lack of an adequate calculating means, a global structural analysis concerning the whole structure was impossible to perform and two-dimensional effects in plan were rarely taken into consideration and

d) The design of the elements of a structure often followed processes that have in the greater part been revised (inaccurate models, an absence of a capacity design (EC 8, 1995), the significance of ductility, insufficient minimum and maximum structural element detailing requirements, etc.).

Consequently and without exaggeration, from the aspect of life expectation, it represents a constitutional inequality that Greek citizens are separated in two categories depending on the age the building that they reside in (TEE, 2001). This is because the potential risk for buildings constructed before 1985 (or 1995) can be estimated to be a minimum of 2 to 1 and is more likely to be 3 to 1 (Tassios, 2000).

Therefore, it is apparent that considerations for the strengthening of buildings that have been designed with older codes should be put forward. Usually this is only done for those buildings that have suffered damage from a forceful earthquake.

However, as easy as the determination of the problem is, a solution is much more difficult to find. If the seismic strengthening of all buildings constructed before 1995 is unfeasible, then it is not easy to find the answers to the following questions:

- Which buildings have the priority to be strengthened and how will this be determined on an individual basis?
- Up to which point can they be strengthened or is the solution of demolition and reconstruction preferable?
- Which materials, methods and techniques are available in order to intervene and under what specifications are they to be applied?
- What is the most suitable method of strengthening of any given building?
- What calculating background does to the engineer require for documenting choices and what are the required procedures of quality control?

The subject is complex and it presupposes that factors such as the importance, the number of users of the building, the cost of the intervention, the age of the building and the remaining life span of the building should be taken into consideration before determining criteria of acceptance in the redesign of an existing building. For practical and economical reasons, it is reasonable to expect lower criteria than those for new constructions. Characteristically in Greece, as the same criteria of acceptance for the redesign of older buildings is established in accordance with that of new buildings, very few owners decide to strengthen their building, especially during periods of low seismic activity. This is because the increased cost and the difficulty of the intervention that accompanies the decision to meet the seismic capacity level of new buildings, often stops the whole upgrading process.

Therefore, a broad scale strategy for the redesign of older buildings is required. Any strategy must include all the factors in question and should lead to the prioritisation of interventions. The techno-scientific side of the question, which deals with planning the required intervention, is a subject that is far more difficult and complex than that of the designing new constructions. It represents a unique challenge to the engineer and requires a high degree of judgement and prudence since:

(i) Knowledge of the subject is limited and is not sufficiently argued,
(ii) Regulations do not exist,
(iii) The design of an existing structure may be unacceptable but the building exists,
(iv) The basic data that has been estimated during the initial phase of documentation of an existing situation may prove to be inaccurate,
(v) New materials are promoted on the market but their behaviour is still under investigation and
(vi) The specialisation and the experience of contractors for the implementation of the work can be limited and, at times, may be negative.

Independent from the lack of executable criteria for the redesign of buildings in Greece, it appears essential for the prompt strengthening of buildings that offer services of urgent need or have a specialised use (for example, hospitals, telecommunication buildings and schools), so that they could remain in operation after an earthquake. Without doubt, the redesign should not only aim to prevent collapse but also to restrict the deflection in order to avoid damage to the architectural and mechanical elements of the building, as any damage may inhibit the availability of the corresponding services that they offer.

The above discussion began in Greece many years ago (Tassios, 1984) and it has recently resulted in a series of actions. Since 2003, the ANTYK plan of TEE (2001) has aimed to assess the seismic capacity of the existing buildings of the country. A rapid visual screening procedure is used to representatively sample 2% of buildings outside cities and 2% of building blocks inside cities. The data collected is then extrapolated, through the national 10-year census, to cover the whole building stock of Greece. In addition, the Organization for School Buildings and the Ministry of Environmental Planning and Public Works has used a rapid visual screening procedure to inspect all the schools and other public buildings in Greece. Furthermore, the "Provisional Technical Specifications" (Dritsos et al, 2004) for interventions has been published as part of the IOK (Institute of Construction Work Economics) project. To date, the IOK project has completed the technical specifications for 40 intervention works that have been applied in practice and these specifications have been presented as a web site that is available to the engineering community. Finally, the "Guidelines of Intervention" (OASP, 2001) for reinforced concrete and masonry buildings has been published and, following on from this, a 17 member Scientific Committee has produced a first draft Greek Retrofitting Code (GRECO, 2004). The final version of the code should be published in 2 years time. In conclusion, the outlook appears to be optimistic and it is to be hoped that the subject of retrofit design will clarify before long.

2. THE PROCESS OF REDESIGNING EXISTING BUILDINGS

There are three main stages in the process of redesigning existing buildings. The first stage deals with assessment. This should involve the inspection of the building, the identification and documentation of the load bearing structure and the assessment of the seismic capacity of the structure. The second stage deals with the procedure of decision-making. This should include the investigation of the probable form of intervention and the choice of an appropriate solution. The third stage deals with the design of the solution that has been selected. The design should include the structural detailing of the repaired and/or strengthened elements of the structure, the technical description of the work to be carried out and the cost of the solution.

The first stage involves the determination of the structural system, the identification of any damage and the estimation of other factors such as the vertical loads and the mechanical characteristics of the construction materials. The evaluation of any damage to, or imperfection in, individual elements will not be of any benefit if the pathology of the whole structure has not been appraised. This should be confirmed by an analysis that will estimate the seismic capacity of the building. Apart from using a more specialised method to assess the capacity of the structure, either an approximate method or one of the more advanced inelastic analysis methods should be selected to assess the deformation characteristics of the individual elements of the structure (ATC 40, 1996; FEMA 356, 2000; fib, 2003a; fib, 2003b; Otani, 2003). At the end of this stage, the decision for the need to strengthen the structure arises. The choice of a desirable "performance level" is required before this decision can be made. In other words, the desired behaviour of the structure, in relation to a design earthquake level, can be expressed by the probable occurrence of an excessive seismic event during the conventional life span of the building (considered to be 50 years). For new buildings, according to current regulations (EC 8, 1994), it is mandatory that at least "the protection of the life and the affluence of the residents" should be selected, while the probable occurrence of an excessive seismic event should be no higher than 10%. However, for existing buildings, for practical and economical reasons, it would be reasonable to accept lower performance levels. The same could be said after an intervention, as a minimum initial performance level of the structure is required.

The second stage, which deals with the decision-making for the intervention, perhaps involves the most difficult part of the whole process. This is because a number of factors are involved that are not easy to quantify. All the parameters that have to be evaluated can influence the decision between any of the following three critical choices:

- No intervention or repairs only if the structure is damaged,
- The strengthening of the structure and
- The demolition and construction of a new structure.

The repair of the structure can be defined as the process of intervention in a building that has structural damage in order to restore the characteristics of the individual elements and return the structure to its initial condition.

The strengthening of structure can be defined as the process of intervention in a building, with or without damage, which increases the strength of the load bearing structure to a level higher than that of its initial design.

The critical decision between repair, strengthening or demolition and reconstruction, as well as the more specific process of intervention that will finally be proposed, will result from the process of repetitive examination of alternative forms of intervention in order to ensure an acceptable seismic behaviour of the structure.

If strengthening is selected, the search for a suitable solution can be done in one of two ways. In the first way, the strengthening of the whole structure could be considered and the objective would be to decrease the stress in the weaker elements to a level lower than their capacity. In the second way, the strengthening of individual weaker elements of the structure may be considered and the objective would be to increase the strength, the ductility or any other lacking characteristic. The first approach is usually followed when there are many weak elements in the structure and consequently a total confrontation of problem is required. The second approach is usually followed when it has been determined that only certain local weaknesses in the structure should be eliminated.

In any case, for buildings that have suffered damage from a strong earthquake, independent from the above, the pathology of the structural damage indicates irrefutable evidence of the seismic capacity of the building. Thus, in buildings with extensive and heavy damage, the intervention should aim to strengthen the whole structure.

The third stage deals with the design of the intervention. It will include the structural design and the detailing of the repaired and/or strengthened elements. The use of new elements, in collaboration with the older ones, will create new multi-phase composite elements for which the structural design may often be different from the usual process of designing monolithic elements of reinforced concrete. Moreover, the use of new materials, such as sheets or fabrics from fibre reinforced polymers (FRPs), for the strengthening of existing elements creates an interesting field of application. However, this method requires particular attention due to the lack of experience and often-excessive enthusiasm that results from the ease of use of this application in practice. It must be noted that the redesign procedure of a structure should always include the production of detailed drawings of the final study and an estimation of the cost of the intervention. The final choice should be considered as economically beneficial. Unfortunately, it has often been observed that, in practice, certain solutions are exasperatingly uneconomical without any particular reason.

3. STRATERGIES AND METHODS FOR SEISMIC STRENGTHENING

Results from a pushover analysis of a building can be easily converted to terms of base shear and top displacement. Obviously, base shear against top displacement curves, such as those presented in figure 1, can approximately illustrate the capacity of a structure. On the other hand, from the code requirements that describe seismic activity in terms of acceleration or displacement response spectra, the seismic performance demand of a building can be expressed in the same figure by curve (s), depending on the equivalent viscous damping of the structure. Curve (s) designates the boundary between safe and unsafe structural behaviour. In other words, a structure can be considered safe if the curve that describes its behaviour extends into the safe region. Otherwise, strengthening of the structure will be required. Consequently, by increasing either the strength or the stiffness of a structure, a safe solution can be found. A safe solution can also be achieved by either rectifying local weaknesses or by increasing the ductility and therefore increasing the ability of the structure to deform in-elastically.

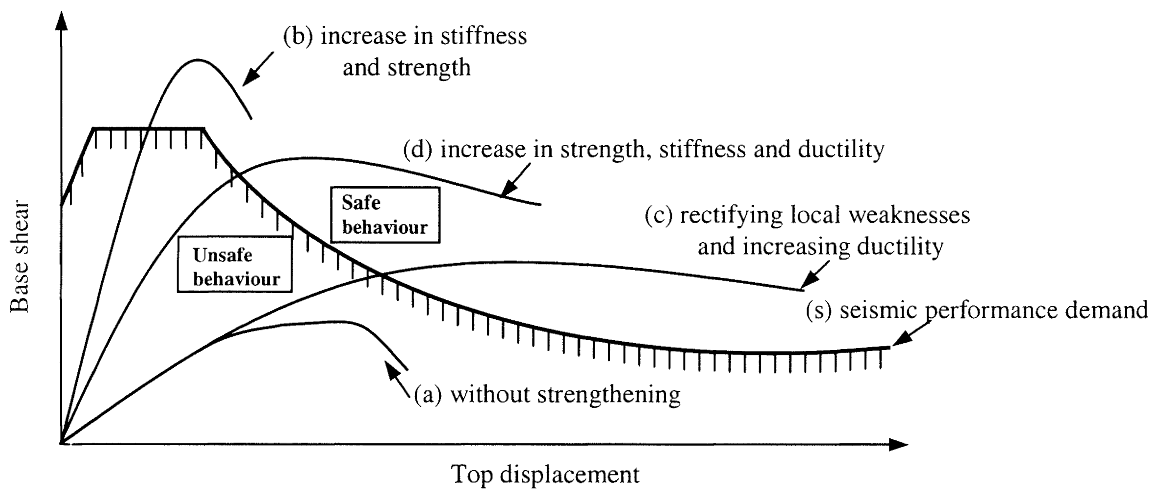


Figure 1. Strengthening strategies.

An interesting alternative would be to seismically isolate the building or to incorporate special dissipation devices. In this way, the response of the structure to seismic excitation will be decreased and, consequently, the required seismic performance will correspondingly decrease.

Plots of base shear against top displacement for the three basic strategies of seismic strengthening are shown above in figure 1. Curve (a) illustrates the behaviour of the structure before strengthening. Curve (b) shows the behaviour of the structure after strengthening by increasing the strength and the stiffness. Curve (c) demonstrates the behaviour of the structure after strengthening by rectifying local weaknesses and increasing the ductility. Curve (d) depicts the behaviour of the structure after strengthening by simultaneously increasing the strength, the stiffness and the ductility.

The choice of the most suitable strengthening strategy and construction method is not always easy. Initially, all alternative processes need to be evaluated while also taking into consideration local conditions or legal, urban or other restrictions. Subsequently, other important factors such as the cost and the duration of intervention, the extent of annoyance to the tenants and the availability of suitable specialised personnel (TEE, 2001) should be evaluated.

It should be pointed out that interventions that involve strengthening by the addition of new elements may change drastically the initial static system of the structure and any decision to intervene should be taken with care. Therefore, a completely new analysis of the structure will be needed and several interventions throughout the entire structure may be required. Consequently, special controls are required at the places of interaction that will ensure the ability of connections to transfer forces between the old load bearing structure and the new elements. Further details on the process of applying of the above methods, as well as other more specific problems that may emerge, can be found elsewhere (UNIDO/UNDP, 1985; Dritsos, 2001).

It is evident that the determination of the number and correct position of new elements will critically influence the effectiveness of any intervention. With a new structure, restrictions stated in the seismic regulation (EC 8, 1995) for the avoidance of an abrupt change in stiffness with height are taken into consideration. Normally, the engineer decides from experience based on simplifications when analytically modelling the structure. However, for cases that have special requirements, the decision should be based on a more precise analytical estimate of the seismic behaviour of the structure. The contribution of existing non-load bearing elements of the structure (for example, infill walls) and elements that are often ignored in the analysis (such as stairwells) should also be taken into consideration.

A variety of methods and techniques are in use today in the practice of seismic strengthening of buildings. More specifically, with regard to structures of reinforced concrete, six main categories of methods can be distinguished depending on the type of additional elements to be used. A multitude of alternative techniques are also available (CEB Bul. 162, 1983; Sugano, 1996; FEMA 356, 2000; Dritsos, 2001). In addition, it may be appropriate for a combination of methods or techniques to be applied in order to find the best technical and most economical solution. The six main categories of methods of intervention are as follows: a) The addition of infilled walls, b) The addition of new external walls, c) The addition of bracing systems, d) The construction of wing walls, e) The strengthening of weak elements and f) The incorporation of energy absorbing systems.

a) Addition of infilled walls

The construction of infill walls within the frames of the load bearing structure, as shown in the example of figure 2, aims to drastically increase the strength and the stiffness of the structure. This method can also be applied in order to correct design errors in the structure and, more specifically, when a large asymmetric distribution of strength or stiffness in elevation or an eccentricity of stiffness in plan have been recognised. Usually, reinforced concrete (ready-mixed or shotcrete) walls manufactured on site are used. Alternatively, for less stiff interventions, prefabricated concrete panels, concrete blocks or masonry walls could be placed.

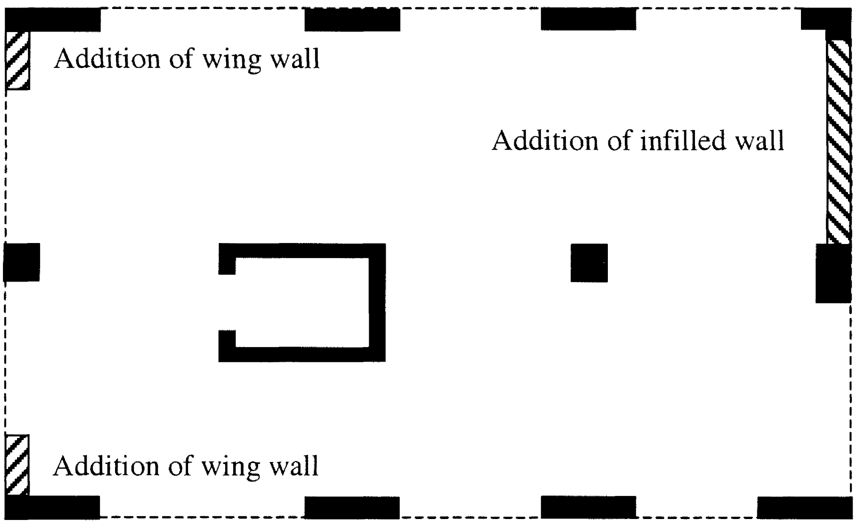


Figure 2. Addition of infilled wall and wing walls.

As shown in figure 3, there are two alternative methods of adding infill walls. Either the infill wall is simply placed between two existing columns or it is extended around the columns to form a jacket. An undesirable increase in stress within the existing columns can be expected when connecting the infilled wall to columns, as the columns become the ends of a new wall consisting of the infilled wall and the two existing columns. Consequently, the second method is specifically recommended in order to increase the strength in this region. In the situation where the existing columns are very weak, a steel cage should be placed around the columns before constructing new walls and column jackets. In situations where a more ductile behaviour of the repaired structure is required, only the upper and lower part of the wall should be connected to the beams and a small void should be left between the new wall and the columns. In all cases, the base of any new wall should always be connected to the existing foundation.

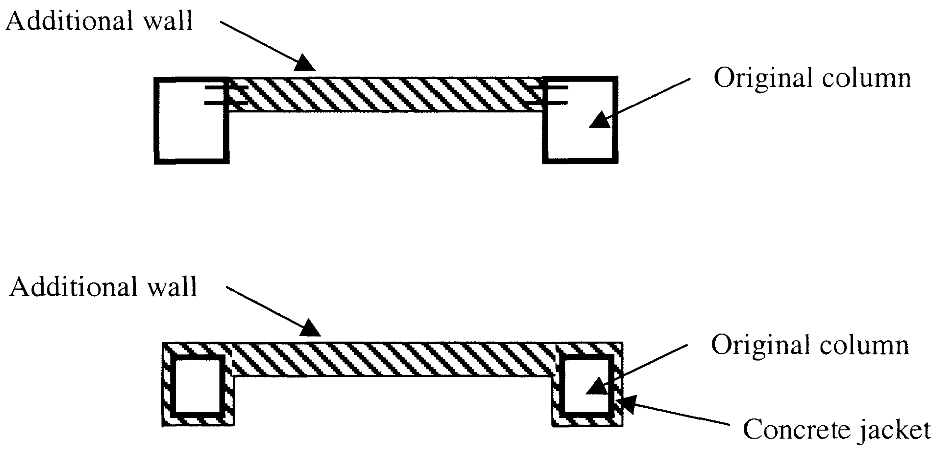


Figure 3. Two alternative methods of adding infill walls.

From the design point of view, an important issue would be the guaranteed transfer of horizontal stress to the new wall during seismic excitation. The existing beams, above and below the new infilled wall, should be checked to see if they have sufficient longitudinal reinforcement for the required transfer of stress from the existing slabs. If the reinforcement has been found to be insufficient, strengthening should include the addition of new horizontal connecting elements. A method for this type of connection would be as follows:
Initially, new horizontal reinforcement bars are placed longitudinally at the level of the beams in every storey. The reinforcement is then welded to strong longitudinal steel plates that have been anchored to the beams. Finally, after suitable preparatory work (scarifying and cleaning) of the beam's surface, the reinforcement can be covered with shotcrete.

Of particular importance would be the control of a sufficient anchorage for the new reinforcement bars. In addition, special measures should always be taken to guarantee the continuity of the interface between the old and the new concrete by placing adequate shear connectors. Adhesive anchors or special mechanical steel dowels are usually embedded into the old concrete, after the surface of the old element has been roughened and cleaned. Special attention to this contact surface should ensure that the shear stress that develops could be undertaken via the mechanisms that will also develop in this area. The estimate of the shear stress between the contact surfaces is usually performed assuming a monolithic connection between the new wall and the existing frame. That is to say, any slippage between the two surfaces can be ignored. In any case, old experimental results (Liauw, 1972) have show that, even when contacts between the new wall and the old frame are few and simply maintain the wall in its place, the contribution of the wall continues to be important.

Alternatively, for less stiff interventions, prefabricated concrete panels, concrete block or brick masonry walls can be placed. It is worth mentioning that the latter solution has been widely applied in Greece for cases of strengthening soft ground floor levels of multi-storey buildings. The technique has proved to be very efficient in increasing the stiffness and enhancing the seismic behaviour of the whole building (Karayiannis *et al.,* 2005). For cases where the infilled masonry walls have window or door openings, experimental results have show that the effectiveness of the technique was significantly reduced (Kakaletsis and Karayiannis, 2003). However, from these authors' results, it can be recognised that by choosing special positions for openings, the reduction in the effectiveness of the technique can be small.

b) Addition of new external walls

In some cases, strengthening by adding concrete walls can be performed externally. This can often be carried out for functional reasons as, for example, in cases when the building must be kept in operation during the intervention works. New cast-in-place concrete walls, constructed outside the building, can be designed to resist part or all the total seismic forces induced in the building. The new walls are preferably positioned adjacent to vertical elements (columns or walls) of the building and are connected to the structure by placing special compression, tensile or shear connectors at every floor level of the building. As shown in figure 4, new walls usually have a L-shaped cross section and are constructed to be in contact with the external corners of the building.

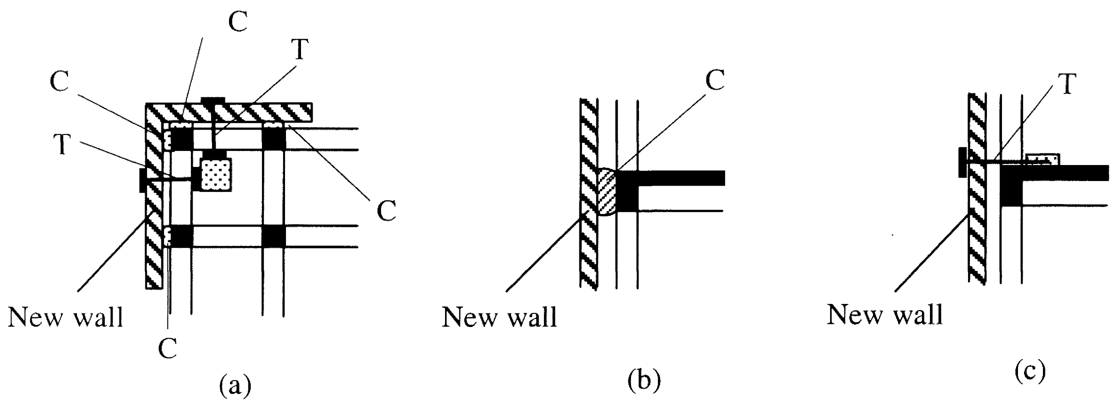


Figure 4. Schematic arrangement of connections between the existing building and a new wall (a) plan, (b) section of compression connector and (c) section of tension connector.

It is important to ensure that connectors behave elastically under seismic design action effects. For this reason, when designing the connectors, a resistance safety factor (γ_{Rd}) equal to 1.4 is recommended (GRECO, 2004). The use of compression and tensile connectors, instead of shear connectors, is strongly recommended as much higher forces can be transferred. It is essential that the anchorage areas for the connectors on the existing building and on the new walls have enough strength to guarantee the transfer of forces between new walls and the existing structure.

A very important issue of the above method concerns the foundation of new walls. Foundation conditions should be improved if large axial forces can be induced in new walls during seismic excitation. In addition, the construction of short cantilever beams protruding from the wall, underneath the adjacent beams at every floor level of the building, as shown in figure 5, appears to be a good solution (Tassios, 2005).

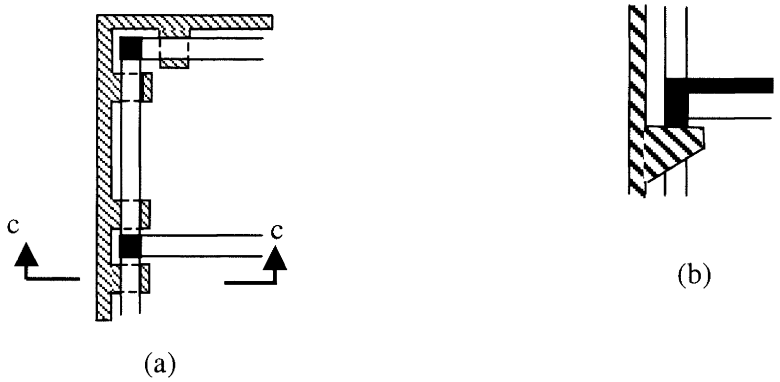


Figure 5. Construction of cantilever beams to transfer axial forces to new walls (a) plan, (b) section c-c.

c) Addition of bracing systems

The construction of bracing within the frames of the load bearing structure aims for a high increase in the stiffness and a considerable increase in the strength and ductility of the structure. Bracing is normally constructed from steel elements, rather than reinforced concrete, as the elastic deformation of steel aids the absorption of seismic energy. Bracing systems can be used in a similar way as that for steel constructions and can be applied easily in single-storey industrial buildings and buildings with a soft storey ground floor level where no or few brick masonry walls exist between columns. Various truss configurations have been applied in practice, examples of which are: K-shaped, diamond shaped or cross diagonal. The latter is the most common and is often the most effective solution. It is a common practice to incorporate eccentric necks on selected steel sections in order to avoid undesirable buckling of critical steel elements (CEB Bul. 162, 1983). It can be recommended that, in order to ensure an adequate contact and a better distribution of stresses to the concrete elements of the frame surrounding the bracing, a continuous steel frame should be placed between the load bearing structure and the bracing system. Alternatively, the bracing elements can be adapted with special provisions to avoid stress concentrations and can be connected directly to the load bearing structure. For reasons of constructional ease, this application can often be applied externally to the existing structure, especially in the cases where it unacceptable to stop or disturb the functioning of the building.

From the design point of view, it is important to correctly choose the frames of the building where bracing will be added. More than one alternative solution must be examined before deciding the final choice. It should be mentioned that, generally, an abrupt discontinuity in elevation should be avoided although it has been noted that, in some cases, a well-designed distribution of bracing within the structure may not follow this rule (Alexakis, 2005).

Particular attention will be required when evaluating the redistribution of stress within the structure. Additional action effects will be induced within the structure and local stress concentrations will occur in the elements around the frame of the bracing. Sufficient strength at the beam-column joints will be required, as this is the region of interaction between the old structure and the new elements. Weak joints are normal in old structures and cannot undertake additional stresses. Therefore, weak joints must also be included in the strengthening procedure and will require modification by incorporating steel elements into the load bearing structure.

Bracing systems have the advantage of small self-weight, speed of construction and do not limit the lighting of spaces. Although this method has the benefit of a widespread application in many countries of high seismic activity (Sugano, 1996), the application of the method in Greece is limited.

d) Construction of wing walls

The construction of reinforced concrete wing walls in continuous connection with the existing columns of a structure, as shown above in the example of figure 2, is a very popular technique in Greece. As presented in figure 6, there are two alternative methods of connecting the wing wall to the existing load bearing structure. In the first method, the wall is connected to the column and the beams at the top and the base of any floor level. Steel dowels or special anchors are used for the connection and the reinforcement of the new wall is welded to the existing reinforcement. In the second method, the new wing wall is extended around the column to form a jacket. Obviously, in this case, stresses at the interface between the new concrete and the existing column are considerably lower when compared to the first method. Moreover, uncertainties regarding the capacity of the connection between the wall and the column do not affect the seismic performance of the strengthened element. Therefore, the second alternative method is strongly recommended.

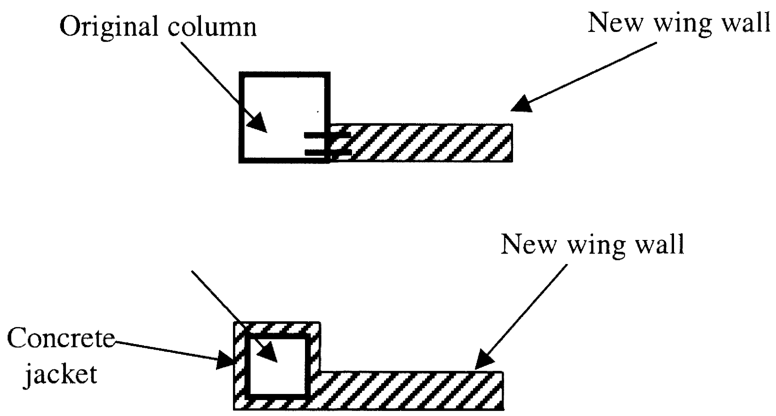


Figure 6. Construction of reinforced concrete wing walls.

e) Strengthening weak elements

The selective strengthening of weak elements of the structure aims to avoid a premature failure of the critical elements of a building and to increase the ductility of the structure. Usually, this method is applied to vertical elements and is accompanied by the construction of fibre reinforced polymer (FRP) jackets or, as shown in figure 7, steel cages around the vertical elements. If a strength increase is also required, this method can include the construction of column jackets of shotcrete or reinforced concrete.

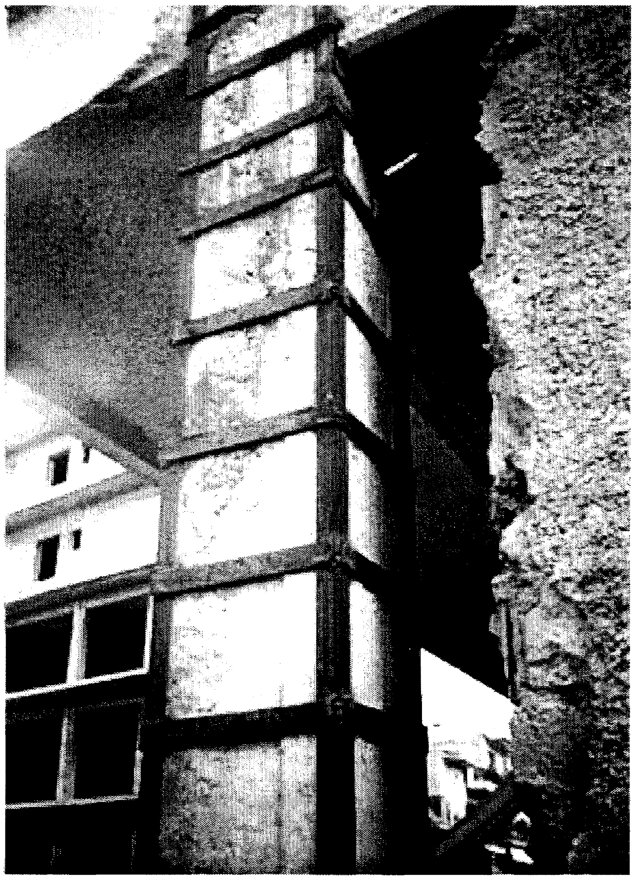


Figure 7. Construction of a steel cage around a vertical element.

In Greece, strengthening of reinforced concrete buildings by placing concrete jackets on selected columns is the preferred engineer's choice. This is mainly because the available technical staff and construction workers can easily execute the technique. In addition, experience from past earthquakes indicates that the method considerably enhances the seismic performance of a structure, even when the construction of the jacket has been performed in a slipshod manner.

Figure 8 presents results from ongoing experimental research programs being carried out at the University of Patras, which concern the retrofitting of existing structures (Bousias et al., 2004; Vandoros, 2005; Vandoros and Dritsos, 2005a; Vandoros and Dritsos, 2005b). Lateral force against top displacement plots for full-scale cantilever column specimens strengthened by concrete or carbon fibre reinforced plastic (CFRP) jacketing are presented for comparison purposes together with plots of an unstrengthened specimen and a monolithic specimen that had the same cross sectional dimensions as the columns with concrete jackets. Figure 8a is the plot of the unstrengthened specimen that had cross sectional dimensions of 250 mm by 250 mm. Figures 8b and 8c respectively present plots of columns strengthened with two and four 0.13 mm thick layers of CFRP. Figure 8d is the plot of a column strengthened by a shotcrete jacket of 75 mm thickness. Figure 8e is the plot of a column strengthened by a shotcrete jacket of 75 mm thickness and, as shown in figure 9, the addition of special bent down steel connectors welded to and between the longitudinal bars of the jacket and the column. Figure 8f is the plot of a column strengthened by a shotcrete jacket of 75 mm thickness, the use of steel dowels and roughening the surface of the original column, as shown in figure 10. This method of connecting at the interface is the recommended procedure in Greece (Dritsos, 2001; GRECO, 2004). Figure 8g is the plot of a column strengthened by a 75 mm thick cast in place concrete jacket and no other connecting measure at the interface. This method of strengthening a column could be considered as the worst concrete jacket strengthening technique and was designed to form a lower limit. Figure 8h is the plot of a column strengthened by a 75 mm thick cast in place concrete jacket that was placed while the column was subjected to 75% of the total axial load. Finally, Figure 8i is the plot of a monolithic column that had the same cross sectional dimensions and reinforcement as that of the concrete jacket specimens.

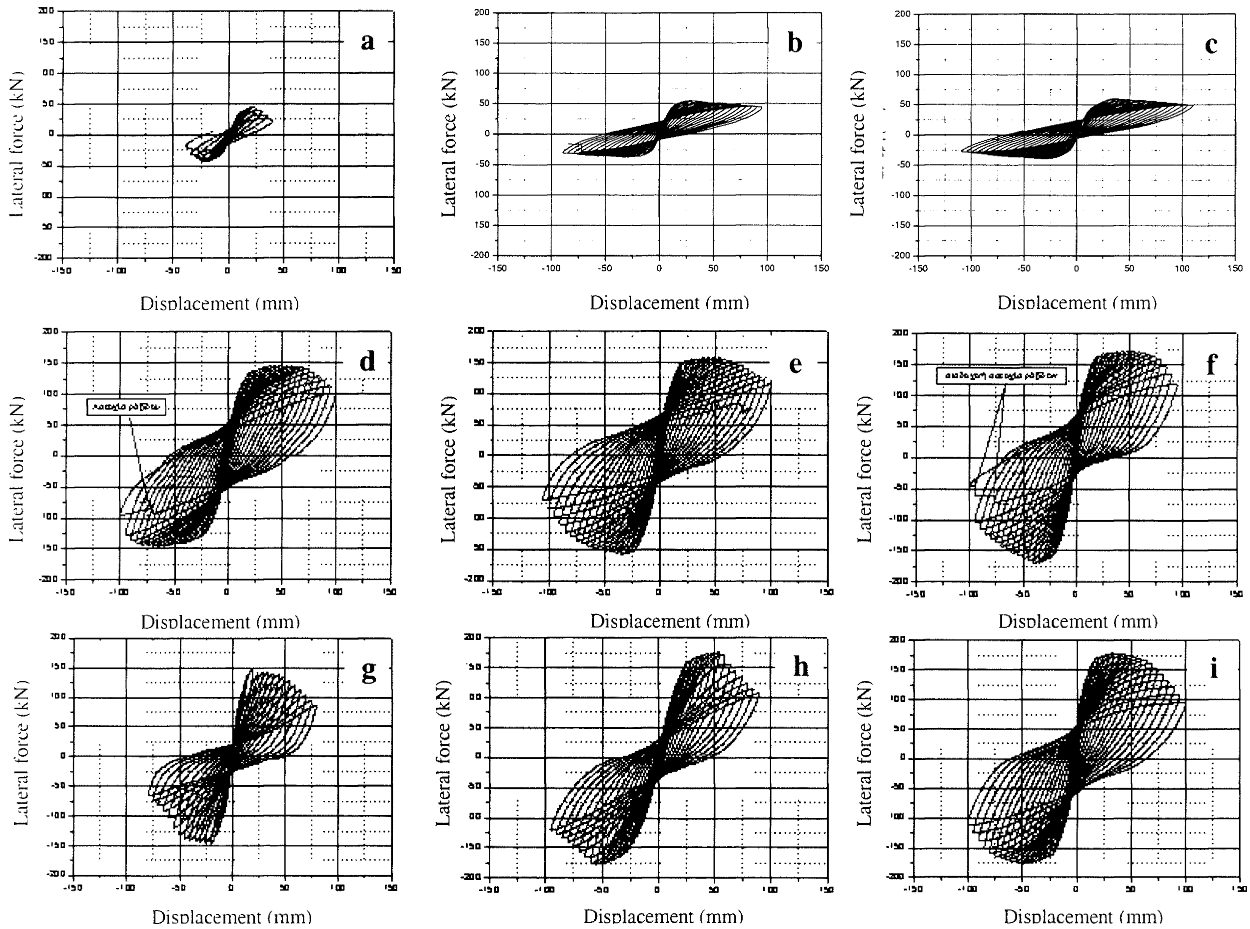


Figure 8. Lateral force against displacement comparisons of various strengthening techniques. (Bousias et al., 2004; Vandoros, 2005; Vandoros and Dritsos, 2005a; Vandoros and Dritsos, 2005b).

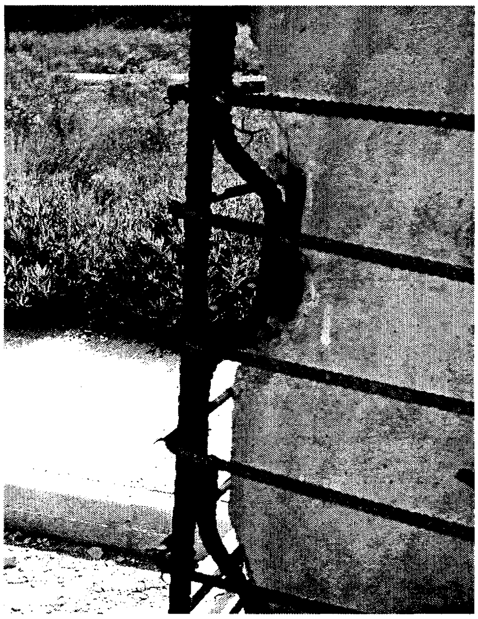


Figure 9. Detail of a bent down connecting bar.

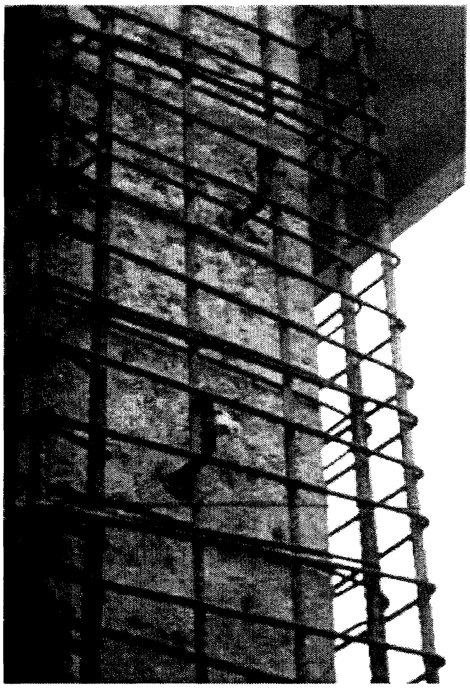


Figure 10. The use of steel dowels and roughening the surface of the original column.

As can be seen from figure 8, the improvement in strength, stiffness and ductility of columns strengthened by concrete jacketing is significant. It is very interesting that even when the concrete jacket was constructed without any specific construction detailing (as in the specimen represented by figure 8f), a considerable increase in strength, stiffness and ductility was observed. On the other hand, it is obvious that jacketing by FRPs only increases the ductility. In addition, by comparing the plots of figures 8h and 8i, it can be seen that, when the column is subjected to an axial load during jacket construction, the stiffness is considerably reduced.

The evaluation of the critical weaknesses of the structure and the choice of a suitable technical intervention are of fundamental importance for the success of this method. Often, a lack of knowledge may lead to unproductive results. Certain issues of practical importance that may help to avoid mistakes in choosing the appropriate technique are as follows:

(1) The strengthening of columns by using FRPs or steel jackets is unsuitable for flexible structures where failure would be controlled by deflection. In this case, the strengthening should aim to increase the stiffness,
(2) Is not favourable to use steel cages or confine with FRPs when an increase in the flexural capacity of vertical elements is required,
(3) The application of confinement (with FRPs or steel) to circular or rectangular columns would increase the ductility and the shear strength and would limit the slippage of overlapping bars when the lap length has been found to be insufficient. However, a significant contribution cannot be expected for columns of rectangular cross section with a large aspect ratio (Antoniades et al., 2003), or those with L-shaped cross sections (Vintzileou and Sigalas, 2003),

(4) In the case of columns that have heavily rusted reinforcement, strengthening with FRP jackets (or the application of epoxy glue) will protect the reinforcement from further oxidation. However, if the corrosion of the reinforcement is at an advanced stage, it is probable that strengthening may not stop the premature failure of the element,

(5) The construction of FRP jackets around vertical elements will increase the ductility but it cannot increase the buckling resistance of the longitudinal reinforcement bars (Plakandaras et al., 2001). Thus, if the stirrups are too thin in an existing element, failure will probably result from the premature bending of the vertical reinforcement. In this case, local stress concentrations from the distressed bars will build up between the stirrups and will lead to a local failure of the jacket. Consequently, if bending of the vertical reinforcement has been evaluated as the most likely cause of column failure, the preferable choice for strengthening of the element would be to place a steel cage,

(6) In areas where the overlapping of reinforcement bars has been found to be inadequate (short lap lengths), confining the element with FRPs, steel cages or steel jackets will improve the strength and the ductility of the region considerably. However, even if it improved the behaviour, it is eventually unfeasible to deter the slipping of bars (Harris et al., 2003). Consequently, when the lap length of bars has been found to

be smaller than 30% of code requirements, the solution of welding of bars must be selected. Moreover, it must be pointed out that confinement cannot offer anything to longitudinal bars that are not in the corners of the cross section and

(7) Experimentally, the procedure of placing FRP sheets to strengthen weak beam-column joints has proved to be particularly effective (Antonopoulos, 2001). In practice, however, this technique has been found to be difficult to apply due to the presence of slabs and transverse beams. The same problem arises when placing steel plates. Other techniques, such as the construction of reinforced concrete jackets or the reconstruction of joints with additional interior reinforcement, appear to be more beneficial (Tsonos, 2001; Tsonos and Papanikolaou, 2003)). In cases where only a light damage to the joints has been found, repairing with an epoxy resin appears to be a particularly effective solution.

(f) The incorporation of energy absorption systems

The attractive idea of isolating a structure from seismic activity and to dissipate part of the seismic activity is very old. However, from the construction point of view, the incorporation of isolation and dissipation devices in existing buildings is a very difficult and complex problem. To date, the application of the method, in order to retrofit a building in Greece, has not been reported. It is the author's opinion that the widespread use of dissipation devices in other applications in the country will very soon extend to retrofitting works.

4. MATERIALS AND TECHNOLOGY – PRACTICAL ASPECTS

The choice of a suitable solution for the repair or the strengthening of a reinforced concrete structure presupposes that the engineer knows the materials and the techniques that are available for such interventions. An engineer that has not dealt with the issues of intervention will approach the subject with difficulty. Knowledge of the more traditional construction materials of concrete and steel will be insufficient to give a solution, even if they continue to play a major role in the process. Often, the use of new materials and technologies in combination with modified traditional materials are required. A variety of materials and techniques (special types of concrete, polymer glues, repair mortars, steel or fibre reinforced polymer plates, other fibre reinforced materials, dowels and anchors, special techniques for anchoring new reinforcement, connecting or welding new reinforcement and the use of steel elements or tendons) are at the disposal of the engineer for selective use depending on the planned solution. Concrete, resin injections, steel strips or plates and FRPs constitute daily application in the field of intervention. The engineer should know what each technique can offer, where it can or cannot be applied and be able to completely decide the required work. Unfortunately, a lack of sufficient subject knowledge, in combination with the social pressure for a fast restoration of damaged buildings, has often led to bad choices and failed interventions. It is not intended in this paper to extensively describe the materials and the technologies that are available, as further information can be found elsewhere (UNIDO/UNDP, 1985; Dritsos, 2001; OASP, 2001). The object here is to point out certain aspects of essential practical importance for the effectiveness of the intervention that the engineer may often find puzzling or may solve erroneously. Certain practical aspects of the subject are as follows:

(1) The placing of new concrete in contact with an existing element (by shotcreting and especially by pouring) will require prior aggravation of the old surface to a depth of at least 6 mm. This should be performed by sandblasting or by using suitable mechanical equipment (for example, a scabbler and not just simply a hammer and a chisel). This is to remove the exterior weak skin of the concrete and to expose the aggregate,

(2) When placing a new concrete jacket around an existing column, it is not always possible to follow code requirements and place internal rectangular stirrups to enclose the middle longitudinal bars, as show in figure 11a. In this case, it is proposed to place two middle bars in each side of the jacket, so that octagonal stirrups can be easily placed, as demonstrated in figure 11b. In the case where columns have a cross section with large a aspect ratio, the middle longitudinal bars can be connected by drilling holes through the section in order to place a S-shaped stirrup, as shown in figure 12. After placing the stirrup, the remaining void can be filled with epoxy resin. In order to ease placement, the S-shaped stirrup can be prefabricated with one hook and, after placing, the second hook can be formed by hand.

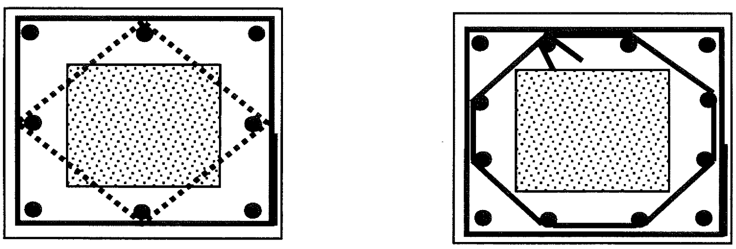


Figure 11. Placement of internal stirrups in rectangular cross sections.

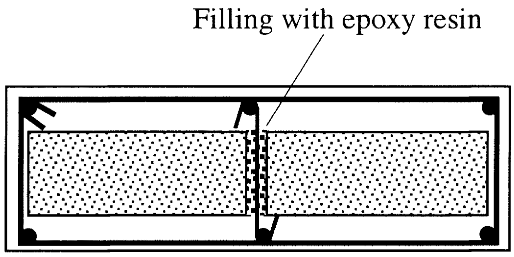


Figure 12. Placement of an internal stirrup in a rectangular cross section with a large aspect ratio.

(3) If a thin concrete jacket is to be placed around a vertical element and the 135° hooks at the ends of the stirrups are impeded by the old column, it would be acceptable to decrease the hook anchorage from 10 times the bar diameter to 5 or 6 times the bar diameter, as shown in figure 13a. Otherwise, the ends the stirrups should be welded together or connected with special contacts (clamps), as presented in figure 13b, that have now appeared on the market,

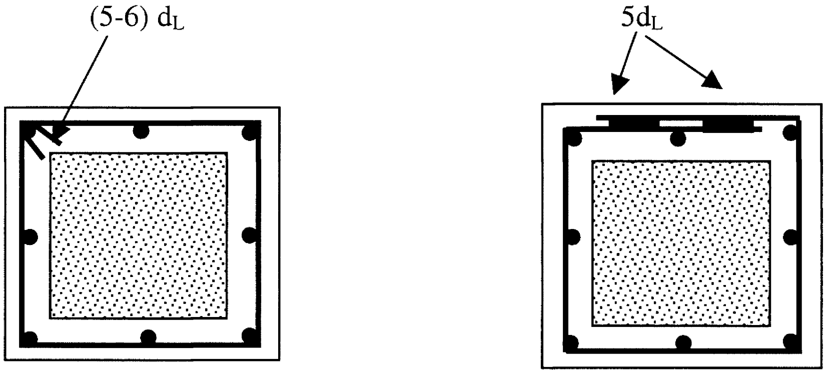


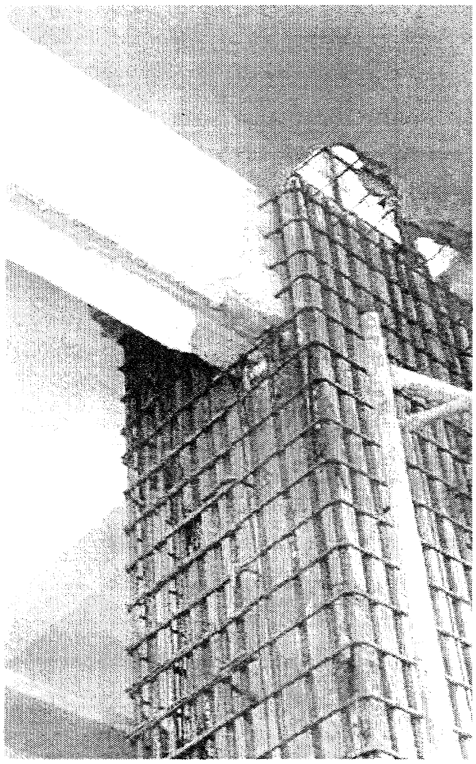
Figure 13. Reducing hook lengths and welding the ends of stirrups.

(4) When constructing a jacket around a column, it is important to also strengthen the column joint. As shown in figure 14, this can be accomplished by, where possible, extending the longitudinal reinforcement bars around the joint. In addition, as also shown in figure 14, stirrups must be placed in order to confine the concrete of the jacket around the joint.

In the case where the joint has been found to be particularly weak, a steel diagonal collar can be placed around the joint before placing the reinforcement, as shown in figure 15.

(5) It is preferable that a new concrete jacket is placed continuously from the foundation to the top of the building. If this is not possible (due to maintaining the functioning of the building), it is usual to stop the jacket at the top of the ground floor level. In this case, there is a need to anchor the jacket's longitudinal bars to the existing column. This can be achieved by anchoring a steel plate to the base of the column of the floor level above and then welding the longitudinal bars to the anchor plate, as shown in figure 16.

(6) In the case where there is a need to reconstruct a heavily damaged column, after first shoring up the column, all the defective concrete must be removed so that only good concrete remains, as shown in figure 17. Any buckled reinforcement bars must be cut and removed and new reinforcement bars must be welded to the existing bars. Finally, the column can be recast by placing a special non-shrink concrete.



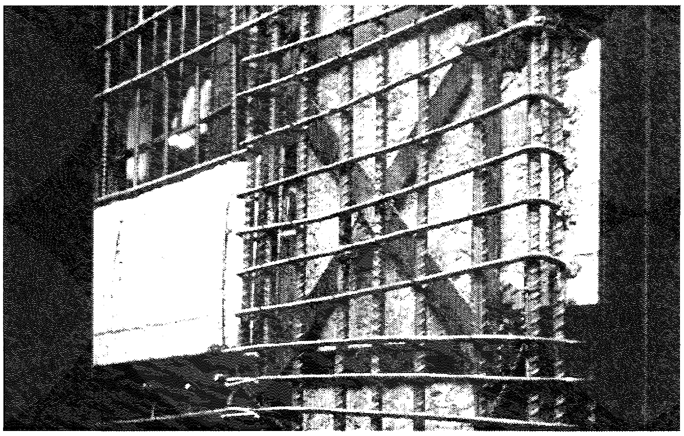


Figure 15. Placing a steel diagonal collar around a weak column joint.

Figure 14. Strengthening the column joint.

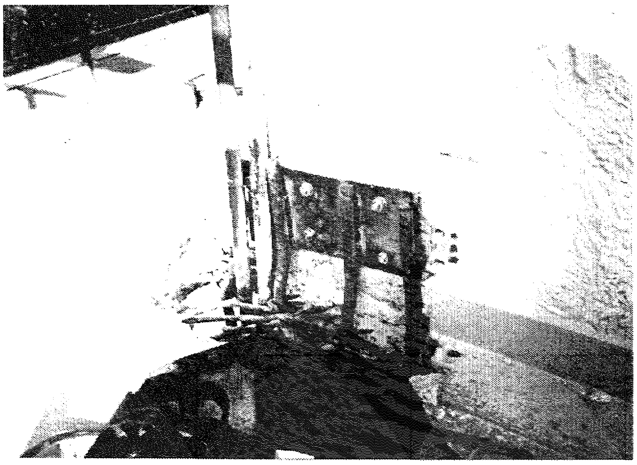


Figure 16. Welding longitudinal bars to an anchor plate.

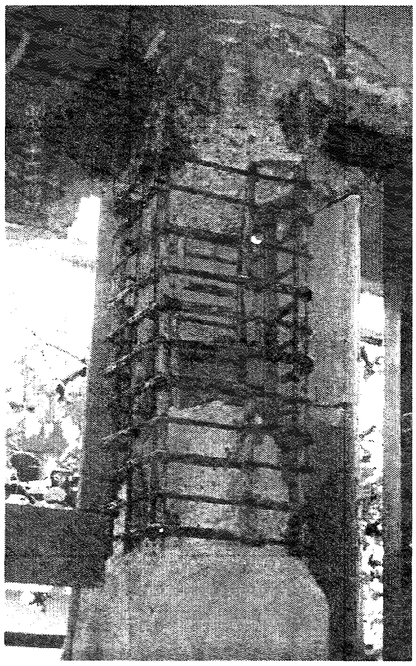


Figure 17. Removal of defective concrete from a heavily damaged column.

(7) In Greece, concrete structures built before 1990 were constructed using S220 (old StI) or S400 (old StIII) reinforcement bars. After 1990, S500s reinforcement bars were used. Welding of S220 and S500s reinforcement bars can be performed without any specific conditions, as they fulfil the requirements for weldable steel according to Greek Standards (ELOT 971, 1994). However, S400 reinforcement bars cannot be considered as weldable, as the carbon equivalent content exceeds the limit of 0.53%. This kind of steel is considered to be "weldable under special conditions" (ELOT 971, 1994). Normal welding will considerably decrease the plastic deformation characteristics of the steel because of the creation of hard martensite components. Preheating the bars to temperatures greater than 200 to 250 °C prevents the creation of the above hard components and preserves the plastic deformation characteristics of the bar (Nikolaou et al., 2003),

(8) In order to anchor new reinforcement bars, dowels or anchors with the use of epoxy glue, the diameter of holes drilled into the existing concrete should be roughly 4 mm larger than the diameter of the bar. The best way to remove dust from drilled holes would be to spray water at the back of the hole. The best results (higher adhesion forces) are achieved when the walls of the hole have been roughened slightly with a small wire brush,

(9) Care is required when shotcreting in the presence of reinforcement. There is a danger of an accumulation of material building up behind the bars. This is usually accredited to material sticking to the face of bars and may be due to either a low velocity, a large firing distance or insufficient pressure from the compressor,

(10) The placing of steel plates and especially FRP sheets or fabrics requires special preparation of the concrete surface to which they will be stuck. The rounding of corners and the removal of surface abnormalities constitute minimal conditions for the application of this technique and

(11) Two constructional issues that concern the connection of new walls to the old frame require particular attention. The first problem is due to the shrinkage of the new concrete and the appearance of cracks at the top of the new wall immediately below the old beam, in the region where a good contact between surfaces is essential. Here, the problem of shrinkage can be usually dealt with by placing concrete of a particular composition where special admixtures (for example, expansive cements) have been used. Alternatively, the new wall could be place to about 20 cm below the existing beam and after more than 7 days (taking into account temperature and how new concrete shrinks with time), the void can be filled with an epoxy or polyester mortar. In some cases, depending on site conditions (ease of access, dry conditions, etc.), the new wall can be placed to a height of 2 to 5 mm below the beam and the void filled with resin glue using the technique of resin injection. The second problem concerns the case of walls from ready-mix concrete and the difficulty of placing the higher part of the wall due to insufficient access. For this reason alone, the use of shotcrete should be the preferred option.

Finally, as interventions are applied under special and often difficult conditions, a quality control system, at a level considerably higher than that applied to new constructions, will be required. Specifically, special attention is needed for all techniques of strengthening to ensure that materials work together and that new materials are well anchored to the existing structure.

5. CONCLUSIONS AND MAIN OBSERVATIONS

The following conclusions and main observations can be drawn from the subject of the seismic retrofit of buildings in Greece:

(a) Many existing buildings (especially those that were constructed before the major revisions to the seismic code in 1985) are at least twice as susceptible to damage from an earthquake when compared to modern buildings. Therefore, a broad scale strategy for their redesign is required. This strategy should include all relevant aspects and should lead to the prioritisation of interventions,

(b) The technical-scientific side of the problem that concerns the design of obligatory interventions for the repair and/or strengthening is a more difficult and complex subject than that of designing new constructions. It constitutes a unique challenge for the engineering and requires a high degree of judgement and prudence since:

(i) Available knowledge on the subject is limited and is not sufficiently documented,
(ii) Regulations do not exist,
(iii) The design of existing buildings may be unacceptable but the buildings exist,
(iv) The basic data that has been collected during the initial phase of documentation of an existing situation for the development of the intervention may often prove to be erroneous,
(vi) New materials are promoted on the market but their behaviour may still be under investigation and
(vii) The specialisation and the experience of contractors for the implementation of the work may often be limited and, at times, may prove to be negative,

(c) The existing structure should be dealt as a whole and the procedure of redesign includes three stages. The first stage is the assessment of the seismic capacity of the existing structure. The second stage involves the process of decision making for the intervention. The third stage concerns the planning of the solution and the estimate of the cost,

(d) For economical and practical reasons, it would be reasonable to accept a lower performance level during an earthquake for a repaired and/or strengthened building when compared to a new construction. The owner of the building must be consulted and must accept the lower performance level. Obviously, after any intervention and with regard to national regulations, an absolute minimum capacity of the structure should be ensured,

(e) The method and the techniques that will be selected for the intervention should be included in the plan of the strategy and should aim to either increase the stiffness and the strength of the construction or to reverse local weaknesses and increase ductility. Alternatively, a combination of the above could be considered or the structure's susceptibility to seismic excitation could be reduced. The more notable methods of intervention are as follows:

(i) The addition of infill walls,
(ii) The addition of new external walls,
(iii) The addition of bracing systems,
(iv) The construction of wing walls,
(v) The strengthening of weak elements and
(vi) The incorporation of energy absorbing systems.

(f) The choice of a suitable solution for an intervention presupposes that the engineer knows the materials and the techniques that are available. In this paper, critical subjects of practical application have been pointed out that may often puzzle the engineer, or may be solved in an erroneous way. Also outlined are the various methods for the strengthening of the structure as a whole, various suitable techniques for local strengthening of critical regions in order to avoid a premature failure and the technology of interventions and

(g) The improvement in strength, stiffness and ductility of columns strengthened by concrete jacketing is significant even when the concrete jacket has been constructed without any specific construction detailing.

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