Assessment of the earthquake behavior of Hotel Ermionio in Kozani, Greece constructed in 1933 before and after its recent retrofit

G. C. Manos & E. Papanoum
Department of Civil Engineering, Aristotle University of Thessaloniki, Greece

Abstract

The earthquake behavior of a retrofitted four-story reinforced concrete (R/C) building, which was constructed in 1933 and belongs to the “Nouveau Monuments of Greece”, is investigated here. The predicted earthquake behavior of the old structure, under the provisions of the current Greek Seismic Code, revealed the low resistance of this building to the design seismic loads. The estimate of the strength of existing structural members was based on tests performed in-situ as well as at Aristotle University. The retrofitting scheme included a partial change of the structural system by cast-in-place R/C walls together with the strengthening of all columns and beams by R/C jackets as well as by the use of carbon fiber reinforced laminates (CFRP) for the improvement of the bearing capacity of the slabs. At the same time, all the architectural features of this building were preserved. The predicted earthquake behavior of the retrofitted structure demonstrated its sufficient earthquake resistance.

Keywords: retrofitting, heritage structure, reinforced concrete, CFRP laminate anchorage, flexural behavior.

1 Introduction

This particular building is a 4-story reinforced concrete (R/C) building with a basement located at the central plaza of the city of Kozani in Greece. Its construction commenced in 1931 under the design and supervision of the German engineer Max Ruthven; the owner was a local merchant (K. Vamvakas) and it became the best hotel of the region for a number of years (Hotel
Ermionio). It is representative of the first R/C structures of the region (Western Macedonia) and it has many common architectural features with a similar R/C building that was built at the same period in Nice, France by the same engineer. During the last decade, especially after the strong earthquake sequence of 13 May 1995 that affected the city of Kozani, the building was not in use as it was in need of structural repairs. Furthermore, it could not be demolished and rebuilt since it was protected under the cultural heritage legislation of Greece. The actual design for its structural repair, submitted to the local city planning authorities, is not presented here. Instead, this paper is concerned with a numerical investigation based on the actual geometry and structural details of the building prior to and after its repair. In assessing the expected earthquake performance of this building, use is made of the provisions of the current Greek Seismic Code [1] in finding estimates of the demands imposed on the structural members. The estimates of the corresponding strengths were based on the actual structural details prior to and after its repair as well as on the findings of a specific set of tests that were conducted either in-situ or at the Laboratory. More information is included in the dissertation of the second author [2] as well as in [3].

2 Assessing the performance prior to the retrofitting

2.1 Level A – quick assessment of the old existing building

This assessment was made according to the guidelines of a relevant assessing method adopted by the Organization of Antiseismic Planning and Protection of Greece [4]. This is a simple method based on the general structural features of a building as well as on particular information about its original design. The grading of this building, according to this Level A assessment was relatively low (total grade $= -1 < 0$); this is an indication that the building may have a non-satisfactory earthquake performance and one must proceed to level B assessment. The guidelines of the recent Greek Code for retrofitting of R/C buildings [5] include provisions relevant to the various aspects of assessing the expected seismic performance of old R/C buildings prior to retrofitting. These
provisions refer to material sampling as well as to the methodology of checking the basic inequality

\[ S_d < R_d \]  

(1)

between the strength \( R_d \) and the demand \( S_d \) placed upon the various members and the values of the safety factors of either the actions leading to the estimate of the demand or the various parameters leading to the estimate of the strength. This is done for assessing the performance prior to retrofitting as well as for designing and checking a given retrofitting scheme.

This process is broadly followed here although no particular reference is made to specific relevant provisions of this code.

2.2 Level B – assessment of the old existing building

For this assessment level, one needs to get information on the structural details and material properties, based both on the original design files as well as on in-situ information. Moreover, the design assumptions made on the mechanical properties of the materials must be supplemented by information obtained in-situ from the actual structure. To this end, concrete and steel specimens were taken from the slab of the ground floor ceiling and tested at the Laboratory. The average concrete strength found was 11.23 MPa. The corresponding characteristic strength that can be assumed today is C8/10 (\( f_{ck} = 8 \text{ MPa} \)). The reinforcing steel bars were of the smooth surface type with yield stress \( f_y = 361 \text{ MPa} \) for the 8 mm rebar (\( f_{ult} = 500 \text{ MPa} \)) and \( f_y = 317 \text{ MPa} \) for the 10 mm rebar (\( f_{ult} = 404 \text{ MPa} \)) and elongation at fracture larger than 20%. The corresponding yield stress that can be assumed today is \( f_y = 350 \text{ MPa} \).

\[ \min M = -23.4 \text{ KNm/m} \quad \max M = 7.2 \text{ KNm/m} \]

Figure 2: Flexural demands upon the ground floor ceiling.

2.2.1 Slabs

For assessing the expected performance of the slabs, a finite element (FE) analysis was performed for all the slabs with predominant loading the combination of dead and live loads as specified by the Greek Code. Fig. 2 shows the results of the ground floor ceiling that is one of the most-stressed slabs due to the relatively large exterior balconies and internal galleries as well as its live load requirements. The extreme values of some of the demands in terms of bending
moments are shown in the same figure. The structural details of this slab are shown in fig. 3.

One piece of the slab (95 mm thick) with dimensions 1.30 m × 0.96 m was removed from the building and was tested under controlled conditions in pure flexure at the Laboratory, resulting in a bending moment capacity $M_{Rd} = 9.18$ kNm/m. The corresponding estimate based on structural details (fig. 3) and relevant calculations is equal to $M_{Rd} = 11.78$ kNm/m. This non-conservative estimate must be attributed to the long term corrosion that reduced the effective cross-section of the reinforcing bars. Comparing the experimental bending slab strength with the numerical analysis maximum demand (fig. 2) for positive slab flexure, a safety factor equal to 1.27 is found; this indicates an acceptable limit-state performance. However, the demand for negative slab flexure yields a minimum bending moment that is approximately two and a half times larger ($M_{min} = 23.4$ kNm/m); this indicates a non-acceptable limit-state performance for the large exterior balconies. The retrofitting addressed this requirement as will be presented in section 3.1.3.

Figure 3: Structural detail of the R/C slab.

Figure 4: Site of earthquake recording and location of Hotel Ermionio.

Figure 5: Acceleration response spectrum for the Kozani earthquake of May 1995.
2.2.2 Multi-story structure

A dynamic numerical analysis was performed for assessing the expected earthquake performance of the multi-story structure according to the provisions of the Greek Seismic Code 2000 [1], using the SAP200 (version 10.1) software. Three different approaches were employed. First, the Simplified Spectral Method, then, the Dynamic Spectral Method based on the design spectrum specified by the Greek Seismic Code and finally, the Dynamic Spectral Method based on the response spectrum that resulted from the 1995 Kozani Earthquake [6-9]. In fig. 5, the design spectrum curve from the Greek Seismic Code (EAK) is included in addition to the acceleration response spectra curves for the two recorded ground acceleration components (East-West, North-South, 5% damping ratio). Moreover, the eigen-periods of the old existing structure as well as those of the retrofitted building are also shown. For the simplified spectral method the following parameters were applied in the numerical analysis:

- Eigen-periods $T_x = 0.47 \text{ s}$, $T_y = 0.483 \text{ s}$.
- Seismic Zone I, design ground acceleration 0.16g, soil category B, importance factor $I = 1$, response modification factor $q = 2.0$, damping ratio 5%, response amplification factor $\beta_o = 2.5$, influence of the foundation $\theta = 1$.

According to the Greek Seismic Code provisions, the original building is a torsionally sensitive structure with design eccentricities equal to:

$$e_{\text{max}} = 5.8 \text{ m}, e_{\text{min}} = 1.25 \text{ m}, e_{\text{min}} = -3.8 \text{ m}, e_{\text{min}} = -0.75 \text{ m}.$$  

According to the dynamic spectral 3-D method of analysis, the eigen-period values for the original building that are associated with more than 90% of the total mass are listed in the following table.

<table>
<thead>
<tr>
<th>$T_1$</th>
<th>$T_2$</th>
<th>$T_3$</th>
<th>$T_4$</th>
<th>$T_5$</th>
<th>$T_6$</th>
<th>$T_7$</th>
<th>$T_8$</th>
<th>$T_9$</th>
<th>$T_{10}$</th>
<th>$T_{11}$</th>
<th>$T_{12}$</th>
<th>$T_{13}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.51</td>
<td>0.96</td>
<td>0.94</td>
<td>0.54</td>
<td>0.33</td>
<td>0.33</td>
<td>0.27</td>
<td>0.16</td>
<td>0.16</td>
<td>0.12</td>
<td>0.09</td>
<td>0.09</td>
<td></td>
</tr>
</tbody>
</table>

Figure 6: $T_2 = 0.96 \text{ s} (70\% \text{ mass participation along } y-y)$.  

From the modal analysis it becomes apparent that there is a degree of coupling between mode 2 and mode 3 with modal participation factors that correspond to nearly 75% of the total mass (see figs. 6 and 7). Fig. 8 depicts the
flexural demands for the ground floor columns as they resulted from the various analysis approaches briefly described above.

- The Simplified Spectral Method.
- The Dynamic Spectral Method based on the design spectrum specified by the Greek Seismic Code.
- The Dynamic Spectral Method based on the response spectrum that resulted from the 1995 Kozani Earthquake (figs. 4 and 5).

Figure 7: \( T_3 = 0.94 \text{ s} \) (70\% mass participation along \( x-x \)).

Figure 8: Predictions of the flexural demands for the ground floor columns.

As can be seen from fig. 8, the flexural demands resulting from the Simplified Spectral Method are 2 to 6 times larger than the corresponding demands resulting from the Dynamic Spectral Method based on the design spectrum specified by the Greek Seismic Code. This must be attributed to the torsional sensitivity of the existing old structure, which is penalized by the provisions of the Greek Seismic Code when the Simplified Spectral Method is applied. When the results of the Dynamic Spectral Method based on the response spectrum of the 1995 earthquake are compared with the corresponding results based on the design spectrum included in the Greek Seismic Code it can be concluded that the
former are 1.25 to 1.75 times smaller than the latter. This must be attributed to the difference in the amplitude of the corresponding design spectral curves when combined with the dominant eigen-period values of the existing old building (see fig. 5). Table 2 lists the predicted inter-story drift values, in terms of angular inter-story response, for the various stories of the old existing structure as predicted by the Dynamic Spectral Method based on the design spectrum specified by the Greek Seismic Code. As can be seen these inter-story drift values are more than 4 times larger than the limit of 0.005 set by the Greek Seismic Code.

Table 2: Inter-story drift values for the frame K1-K9-K15-K22 in the South.

<table>
<thead>
<tr>
<th>Angular Inter-story Drift</th>
<th>Ground floor (h=4.09 m)</th>
<th>Mezzanine (h=2.60 m)</th>
<th>1st story (h=3.60 m)</th>
<th>2nd story (h=3.75 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>γ</td>
<td>0.0233</td>
<td>0.0226</td>
<td>0.0219</td>
<td>0.0166</td>
</tr>
</tbody>
</table>

When the flexural strength capacities of the columns of the old existing building are compared with the flexural demands, predicted as explained above, the demands by far exceed the capacities when these demands are based on the Simplified Spectral Method; when these demands are obtained from the Dynamic Spectral Method based on the design spectrum specified by the Greek Seismic Code they exceed the flexural capacities of almost all the Ground Floor columns. The flexural capacities satisfy the demands for the columns of the structure only when the predictions are by the Dynamic Spectral Method and the response spectrum of the 1995 Kozani Earthquake acceleration recording is used. As pointed out, this is due to the frequency content of this particular ground motion combined with the eigen-frequencies of the old existing structure (figs. 4 and 5).

3 Structural retrofitting

The structural retrofitting (fig. 9), aimed to upgrade the earthquake performance of the building, included jacketing of all the columns and beams (with a thickness of 75 mm for the columns and 50 mm for the beams) as well as the construction of shear-walls (with a thickness of 300 mm), within the bays of the old structure at the North and East perimeter of the building that would not obstruct its functioning which could be served from the other two sides (South and West) facing the main square of the city. The structural retrofitting scheme applied in this particular case combines the following two basic structural retrofitting principles:

a) To increase the strength and ductility of individual structural members by the construction of R/C jackets around the columns and beams in a way that the retrofitted members’ strength can successfully meet the seismic demands imposed on the retrofitted structural members.

b) To partially change the structural system by transforming some of the frames at the back sides of the building to R/C shear walls or by alternative changes
in the structural system. In this way, one aims at decreasing the seismic demands on the individual structural members.

In this way, one tries to vary the terms of both sides of inequality (1) so that a favourable final performance results for all the structural members of the retrofitted structure. The provisions of the Greek Code for design of the repair and strengthening of R/C structures [5] allows the cast-in-place R/C wall, when its thickness is less than the thickness of the surrounding members, to be encased within the bay of the surrounding frame either non-connected or partially connected to the top and bottom beam of the surrounding frame. It also allows, when the thickness of the cast-in-place R/C wall is larger than the thickness of the members of the surrounding frame, to construct a fully reinforced shear wall that encases all the frame’s members. For this type of cast-in-place R/C walls the following three distinct cases are provided for:

1. The cast-in-place R/C wall does not have any particular connection with the members of the surrounding frame apart from simple contact.
2. The cast-in-place R/C wall is connected only at the top and the bottom with the corresponding beams of the surrounding frame. This connection is done with steel anchors placed at certain intervals.
3. The cast-in-place R/C wall is connected at the top and the bottom with the corresponding beams as well as left and right with the corresponding columns of the surrounding frame. This connection is also done with steel anchors placed at certain intervals.

It should be mentioned here that one could utilize either one of the above a) and b) structural retrofitting principles on its own. Thus, one could try to form a structural retrofitting scheme that would rely only on upgrading the strength of the individual structural members of this building by jacketing or other means. Alternatively, one could introduce changes to the structural system, by adding shear walls or other means, in an effort to bring the seismic demands to acceptable levels with the existing strengths of the structural members. The structural retrofitting scheme, that is presented in this paper, combines both the structural retrofitting principles, described above as a) and b), in order to achieve the desired goal; despite this conservative design combination, this structural retrofitting in cost terms represents a relatively small part of all structural, non-structural and maintenance cost for works made on this building.

As this is a listed building, belonging to the Nouveaumonuments of Greece, all the non-structural elements of the façade (door and window openings, wooden shades, cast iron railings etc.) as well as selected non-structural elements in the interior had to be preserved.

As already mentioned, this structural retrofitting scheme resulted in both increasing the torsional stiffness of the structure and thus decreasing the seismic demands as well as upgrading the capacities of all the columns and beams; it extended all the way from the foundation to the top of the building. The specified retrofitting materials are C20/25 for the concrete and S500s for the steel. Moreover, the structural retrofitting included the upgrading of the flexural capacity of the slabs by applying CRFP laminates (see section 3.13).
3.1 Level B assessment of the retrofitted building

The expected earthquake performance of the retrofitted structure was assessed in the same way as described in section 2.2.2 employing the previously described methods of analysis and numerical tools. In what follows, a summary of this assessment will be presented by comparing critical response parameters of the retrofitted building with those of the existing old building.

3.1.1 Influence of the retrofitting on the eigen-modes and eigen-periods

In figs. 10a and 10b the dominant eigen-modes of the retrofitted structure are shown together with the corresponding eigen-periods, which are also listed in table 3. As can be seen, the retrofitting scheme resulted in a significant increase of the lateral stiffness of the structure.
Table 3: Comparison of eigen-period values between old and retrofitted structure

<table>
<thead>
<tr>
<th>Structural scheme</th>
<th>Eigen-period for dominant (y-y) response</th>
<th>Eigen-period for dominant (x-x) response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old Structure</td>
<td>(T_2 = 0.96 \text{ s})</td>
<td>(T_3 = 0.94 \text{ s})</td>
</tr>
<tr>
<td>Retrofitted Structure</td>
<td>(T_2 = 0.15 \text{ s})</td>
<td>(T_4 = 0.09 \text{ s})</td>
</tr>
</tbody>
</table>

3.1.2 Influence of the retrofitting scheme on the maximum displacement and stress response

The influence of the retrofitting scheme is studied by comparing the maximum response values, in terms of displacements and stress-resultant values relevant to flexural response of the columns, by applying the Dynamic Spectral Method based on the design spectrum specified by the Greek Seismic Code [1].

Table 4: Comparison of displacement response between old and retrofitted structure.

<table>
<thead>
<tr>
<th>Top Story Corner</th>
<th>Old Existing Building</th>
<th>Retrofitted Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(u_x) (mm) (u_y) (mm) (\theta_z) (rad) (u_x) (mm) (u_y) (mm) (\theta_z) (rad)</td>
<td></td>
</tr>
<tr>
<td>S-W</td>
<td>51.36 56.99 3.64 (\cdot 10^{-4})</td>
<td>0.62 5.79 1.33 (\cdot 10^{-4})</td>
</tr>
<tr>
<td>N-W</td>
<td>51.10 47.37 5.87 (\cdot 10^{-4})</td>
<td>0.95 1.50 2.47 (\cdot 10^{-4})</td>
</tr>
<tr>
<td>N-E</td>
<td>50.34 47.37 7.08 (\cdot 10^{-4})</td>
<td>2.51 1.50 3.57 (\cdot 10^{-4})</td>
</tr>
<tr>
<td>S-E</td>
<td>50.30 56.51 8.28 (\cdot 10^{-4})</td>
<td>2.64 5.54 3.45 (\cdot 10^{-4})</td>
</tr>
</tbody>
</table>

As can be seen in table 4, the increase in the lateral stiffness of the structure resulted in a drastic decrease of the horizontal displacement response at the ceiling of the third story (top of the building). This decrease occurred despite a relative increase in the lateral seismic forces resulting from the increase of the spectral values corresponding to the dominant eigen-modes of the retrofitted structure when compared with those of the old existing structure (fig. 4). This is also true for the stress-resultant response values relevant to flexure of the columns. These maximum response values are listed in table 5 for all the columns of the Ground Floor before and after retrofitting. As can be seen this maximum response is reduced by 2 to 4 times as a consequence of the applied retrofitting scheme.

A critical element in the applied retrofitting is the strengthening of the existing foundation. The positive aspects regarding the retrofitting of the existing foundation were first, that it was above the water level and second, that it was accessible all around the external perimeter of the building; thus it could be easily retrofitted by new external R/C jackets. Thus, new external R/C jackets were constructed at the top of all the existing footings together with new cast-in-place R/C beams springing from each footing and connecting all the adjacent columns. These new cast-in-place R/C foundation beams were also connected to the reinforcing cages that formed the jacketing of all the corresponding columns.
Table 5: Comparison of stress–response between old and retrofitted structure.

<table>
<thead>
<tr>
<th>Ground Floor Column No</th>
<th>Old Existing Building</th>
<th>Retrofitted Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( N ) (kN)</td>
<td>( M_2 ) (kN·m)</td>
</tr>
<tr>
<td>266</td>
<td>89.92</td>
<td>108.91</td>
</tr>
<tr>
<td>267</td>
<td>246.90</td>
<td>104.47</td>
</tr>
<tr>
<td>279</td>
<td>180.76</td>
<td>103.21</td>
</tr>
<tr>
<td>287</td>
<td>269.94</td>
<td>114.20</td>
</tr>
<tr>
<td>291</td>
<td>197.96</td>
<td>76.08</td>
</tr>
<tr>
<td>299</td>
<td>311.10</td>
<td>150.70</td>
</tr>
<tr>
<td>323</td>
<td>74.65</td>
<td>141.96</td>
</tr>
<tr>
<td>327</td>
<td>317.95</td>
<td>166.11</td>
</tr>
<tr>
<td>347</td>
<td>81.33</td>
<td>182.72</td>
</tr>
<tr>
<td>351</td>
<td>205.96</td>
<td>107.81</td>
</tr>
<tr>
<td>266</td>
<td>89.92</td>
<td>108.91</td>
</tr>
</tbody>
</table>

starting from the foundation level and extending all the way to the top of the building. The reduced flexural demands for the columns, resulting from the cast-in-place R/C walls at the back sides of the building all along its height, combined with the increased capacities of the structural elements due to the construction of new R/C jackets for all the structural elements (columns and beams) lead to a satisfactory earthquake performance of this building after retrofitting.
3.1.3 Upgrading the flexural capacity of the slabs

As described in section 2.2.1, the assessment of the expected performance of the slabs revealed that they were in need of being upgraded regarding their flexural capacity. Towards this objective, a retrofitting scheme needed to be applied to these slabs. This scheme was planned to make use of a certain type of CFRP laminate with a cross-section of 50 mm × 1.2 mm. These laminates could be applied in-situ either on the upper or lower surface of the concrete slabs with the use of a special epoxy paste. An extensive experimental parametric study was performed on this type of attachment by testing a series of specimens prior to applying the best attachment detail to prototype slab specimens. The tests were performed at the laboratory of Strength of Materials of Aristotle University and utilized twin prismatic concrete specimens with dimensions 100 mm × 100 mm × 150 mm each. These twin concrete prisms were joined with the CFRP laminates; they were attached at the two opposite sides of each twin concrete prism. These specimens were then subjected to such a loading that would force the laminates to be detached from the concrete surface. This is shown in figs. 11a to 11d. A variety of attachments between the CFRP laminates and the twin concrete prisms were tried. The simplest form of attachment was the one employing only epoxy paste between the CFRP laminates and the concrete surface (figs. 11a and 11b). Next, a variety of bolting arrangements were utilized with or without the epoxy paste. Figs. 11c and 11d depict such an attachment of the laminates whereby the epoxy paste is combined with one bolt on each side of the twin concrete prism. The bond strength in the first case was found equal to 2.70 MPa whereas in the second case, it was equal to 4.40 MPa, which represents an increase of 63%. The maximum bond strength that was achieved throughout these tests reached the value of 7.20 MPa, which represents an increase of 167% from the simple attachment of the laminates only with epoxy paste. These findings were utilized with slab specimens that were taken from the actual mezzanine slab of the structure; they were cut from parts of the slab where an opening would be formed for a new staircase.

Figure 12 depicts the loading arrangement that was used to subject these slab specimens to four-point flexure. Initially, this was done for a specimen without any retrofitting that reached a maximum bending moment value equal to 8.84 kNm. Then two laminate strips, each having a cross section of 50 mm × 1.2 mm, were attached on the bottom surface of this slab specimen, which was reloaded to flexure as before. Fig. 13 depicts the obtained flexural behavior of this retrofitted slab specimen. At this stage the CFRP laminates were attached to the specimen only with the use of epoxy paste. This time the specimen reached a maximum bending moment value equal to 20.89 kNm, which represents an increase equal to 136% when compared to the maximum bending moment value of the un-retrofitted slab. This retrofitted slab failed in the form of debonding CFRP laminates (fig. 14). The debonding of the two CFRP laminates was followed by a sharp decrease in the flexural bearing capacity together with the formation of a plastic hinge at mid-span in the form of excessive plastic deformations of the old steel reinforcing bars at the bottom of the slab as well as crashing of the concrete at the top of the slab.
Figure 12: Loading arrangement of the slab specimens in flexure.

Next, two new CFRP laminates were reattached to the same slab specimen; this time epoxy paste was used again together with bolts penetrating through the slab and securing the attachment of these CFRP laminates. The flexural bearing capacity of the slab this time reached the same maximum bending moment value as the one observed when only epoxy paste was used for the attachment of the laminates; they exhibited satisfactory performance without any signs of failure.
either in the form of debonding or fracture. This is shown by the bending moment versus mid-span deflection curve plotted for this test in fig. 13. Despite the satisfactory performance of the CFRP laminates, no increase in the flexural bending capacity could be achieved due to the slab damage from its previous loading history; however; the performance of the anchors for the attachment of the CFRP laminates was satisfactory and the flexural behavior of the slab exhibited a much larger range of deformability (fig. 13).

![Debonded CFRP laminates](image)

Figure 14: Flexural failure of the Ermionio slab specimen retrofitted with two CFRP laminates bonded only with epoxy paste. The CFRP laminates were debonded from the concrete slab in this case.

4 Conclusions

The detailed assessment of the old existing building revealed that an unsafe earthquake performance could be expected, especially for the Ground Floor columns as well as for parts of the slab of the Ground Floor ceiling.

It is demonstrated that the retrofitting scheme actually applied to this structure, which combines a partial change of the structural system by cast-in-place R/C walls together with new R/C jackets for all the columns and beams, is successful in meeting the demands specified by the current Seismic Code of Greece. At the same time, all the architectural features of this building, which belongs to the Nouveau Monuments of Greece, were preserved.

This retrofitting scheme was supplemented with a scheme to upgrade the flexural capacity of the slabs by applying a certain type of CFRP laminates; the increase in the flexural capability of the slabs of this building resulting from such retrofitting scheme was validated by a special laboratory investigation.

The successfully retrofitted building is already in use again as a luxury hotel.
References


