Design of seismic retrofitting of reinforced concrete structures

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Abstract

As knowledge increases and buildings in seismic regions are designed with higher degrees of safety, it becomes clear that older structures may require upgrading. In general, codes or regulations to aid the engineer in this field do not exist. At the present moment in Greece and with regard to reinforced concrete structures, there is a considerable effort being placed into making up this shortfall. As part of this effort, this paper concentrates on the latest design procedures for strengthened reinforced concrete composite elements.

Modern seismic design of existing structures is moving towards a displacement capacity assessment of the structure. For monolithic elements, curves and formulas exist to evaluate the values for the yield strength and for the displacements at both the yield stage and at the failure stage. No information exists for the corresponding values for a strengthened element. By applying suitable corrective factors to a monolithic element the capacity curve of a strengthened element can be converted to the corresponding field of monolithic design.

This paper defines the required correction factors and presents experimental results from different strengthening methods used to construct concrete jackets. The experimental results are then used to propose preliminary design values for the required correction factors. Equations for use with the correction factors are also described. Finally, the correction factors are compared with those provided in the relevant codes. Although based on a limited database of experimental results, it has been found that the monolithic behaviour coefficients do not always correspond to the relevant values proposed in design codes.

Keywords: repair, strengthening, retrofitting, design, reinforced concrete, jacket and column.
1 Introduction

It is clear that, as the years pass, knowledge of the seismic behaviour of structures increases, the Aseismic Codes are revised and each new construction can be designed with a higher level of safety.

In Greece, the old Greek Aseismic Code was revised in 1985 (Greek Ministry of Environmental Planning and Public Works, [1]) and finally a New Code was applied for the design of new constructions in 1995 (NEAK [2]). Although new buildings are stronger in resisting earthquakes, an anxiety has arisen with regard to the safety of older constructions.

It can be easily accepted that upgrading appears to be the solution. However, tools to manipulate the subject are rare and the absence of a code or a special regulation for the design of interventions for existing structures makes the problem complex and difficult.

In Greece, actions towards clearing the field with regard to the upgrade of old buildings has already resulted in the publication of the “Guidelines for Pre-Seismic and Post-Seismic Interventions in Buildings” by OASP [3], the publication of the “Provisional Technical Specification” by IOK (Dritsos et al. [4]) and the effort in now concentrating on preparing a Greek Retrofitting Code (GRECO, [5]). A 17 member Scientific Committee, working since 1999, has already present the first draft of the code and it is to be hoped that the final version of the code, which should be published in 2 years time, should help to clarify the subject of retrofit design.

Sharing from the above effort, the present paper focuses on the latest design of strengthened reinforced concrete elements.

2 Structural design concept of retrofitted elements

Repaired and strengthened elements can be considered as multi-phased elements. They consist of the initial element of the load bearing structure and a new element (made from similar or other materials) that has been connected in order to limit shear slippage between the contact surfaces and to avoid detachment. Consequently, for the structural design of the above elements, the process of designing composite elements should be followed and the transfer mechanism of the forces at the interface between the old and the new element should be taken into consideration. Alternatively, an approximate process could be used based on an equivalent monolithic cross section modified by correction factors, providing that reliable correction factors are available.

The connecting procedure between the contact surfaces would have an important effect on the behaviour of a composite element. Figure 1(a) schematically depicts a strengthened section subjected to bending. Since slippage between the contact surfaces cannot be avoid, the strain distribution with height of cross section of the strengthened section will have a discontinuity, as shown in figures 1(b) and 1(c).

Obviously, the size of the discontinuity in the linear strain distribution would depend on the relative slippage between the contact surfaces. Figure 1(c) shows
a greater degree of slippage than that of figure 1(b). Consequently, in order to not only estimate the strength and the deformation capacity of the composite element but also the amount of stress that would be activated or deformation that would develop between the contact surfaces, a model to simulate the shear stress and the shear strain that would occur at the interface due to slippage would be required. This can be achieved by using a diagram of interface shear stress against slippage and would depend on the conditions of the connection between the two parts of the element. Simplified diagrams, concerning different connection conditions, can be taken from the available literature (CEB Bul. N° 162, [6]).

![Diagram](image_url)

Figure 1: Distribution of strain with height of cross section for a strengthened section.

The process of designing the repair and/or strengthening of elements described above can be extremely laborious and may involve a large calculation time. It is probably unfeasible in interventions of a large scale unless suitable software has been developed. Moreover, in order to follow the above procedure, accurate and reliable models for the simulation of the contact surface behaviour are required and these models must take into account the actual working practices on site. Since it is very difficult to simulate what occurs on site, an approximate process could be applied by first considering monolithic conditions in order to use well known structural design methods for reinforced concrete elements. The results could then be correcting using special correction factors (monolithic behaviour coefficients).

Modern seismic design of existing structures is moving towards a displacement capacity assessment of the structure. For that reason, the yield strength or the failure strength of an element are not enough to express the elements capacity. In order to express the whole behaviour of an element, a capacity curve in terms of action effect against displacement is required. For
monolithic members, guidelines for design and recent draft codes (FEMA 356, [7], GRECO, [5]) provide an idealized curve for any member, as shown in curve (a) of figure 2. Moreover, formulas are also provided to evaluate the values for the yield strength, $F_y$, and for the displacements at both the yield stage, $\delta_y$, and at the failure stage, $\delta_u$. However, no information has yet been made available for the relevant values for a strengthened element.

![Diagram of capacity curves](image)

Figure 2: Capacity curves (a) for monolithic elements and (b) for strengthened elements.

The transfer of the actual characteristics of response for the composite element may be considered to be equivalent to applying suitable corrective factors of simulation, $k$, to a monolithic element that has comparable characteristics and cross section. Since the critical member characteristics are the strength and the displacement at the yield stage and the displacement capacity at the failure stage, the required correction factors can be defined as follows:

- For the resistance:
  \[
  k_{F_y} = \frac{F_y, M}{F_y, M}.
  \]

- For the displacement:
  \[
  k_{\delta_y} = \frac{\delta_y, M}{\delta_y, M} \quad \text{and} \quad k_{\delta_u} = \frac{\delta_u, M}{\delta_u, M}.
  \]
With the above given monolithic factors, the capacity curve of the strengthened elements, presented in curve (b) of figure 2 above, can be converted to the corresponding field of monolithic design (that is familiar to the designer).

The determination of reliable monolithic correction factor values for use by the engineer is one of the critical areas in the sector of redesign. Extensive analytic investigations and experimental trials are required in order to determine the real characteristics of stiffness, strength and displacement capacity of repaired and/or strengthened elements. These characteristics must then be compared with the characteristics of corresponding monolithic elements.

3 Experimental data for reinforced concrete columns strengthened with concrete jackets

Values for monolithic correction factors can be drawn from recent experimental results from a number of research projects undertaken at the University of Patras (Fardis and Bousias, [8], Vandoros [9], Vandoros and Dritsos, [10]) and elsewhere (Tsonos [11]). The present paper presents results from the above research. Seven different strengthening procedures used for the construction of concrete jackets are compared with data from monolithic columns that had the same cross sectional dimensions and detailing.

Figure 3: Ends of jacket stirrups welded together.

The seven different strengthening procedures were as follows:
Procedure R involved the roughening of the column surface and the use of shotcrete for the construction of the jacket,
Procedure D involved the use of dowels as connectors at the interface between the column and the jacket and the use of shotcrete for the jacket,
Procedure RD involved the roughening of the column surface, the use of dowels at the interface between the column and the jacket and the use of shotcrete for the jacket,
Procedure W involved the welding of the jacket longitudinal bars to the longitudinal bars of the column and the use of shotcrete for the jacket,
Procedure NT₁ involved the construction of the jacket with shotcrete but no other special connecting measure, such as roughening of the column surface or the use of dowels, was used,
Procedure NT₂ was the same as procedure NT₁ but the jacket was constructed with conventional poured concrete and the ends of the jacket stirrups were welded together, as shown in figure 3 and
Procedure NTP was the same as procedure NT₁ but the concrete jacket was constructed with conventional poured concrete while the original column was subjected to 75% of the total axial load.

For the cases where the procedure involved the roughening of the column surface, a scabbler was used to remove the exterior weak skin of the concrete and to expose the aggregate, as shown in figure 4.

Figure 4: Use of a scabbler to remove the exterior weak skin of the concrete and to expose the aggregate.

The range of the values for the monolithic coefficients $k_{Fy}$, $k_{\delta y}$ and $k_{\delta u}$ that resulted from the above mentioned research are presented in Table 1 for the seven different procedures of constructing the concrete jackets.
Table 1: Experimental values of $k_{Fy}$, $k_{\delta y}$ and $k_{\delta u}$ for columns strengthened with concrete jackets.

<table>
<thead>
<tr>
<th>Type of jacket</th>
<th>$k_{Fy}$</th>
<th>$k_{\delta y}$</th>
<th>$k_{\delta u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>0.85-0.90</td>
<td>1.70-1.94</td>
<td>0.70-0.84</td>
</tr>
<tr>
<td>D</td>
<td>0.80-0.83</td>
<td>1.63-1.89</td>
<td>0.79-0.85</td>
</tr>
<tr>
<td>RD</td>
<td>0.90-0.94</td>
<td>1.15-1.85</td>
<td>0.71-0.83</td>
</tr>
<tr>
<td>W</td>
<td>0.85-0.87</td>
<td>1.16-1.57</td>
<td>1.09-1.39</td>
</tr>
<tr>
<td>NT$_1$</td>
<td>0.80-0.81</td>
<td>1.09-1.26</td>
<td>0.63-0.70</td>
</tr>
<tr>
<td>NT$_2$</td>
<td>0.71-0.73</td>
<td>0.84-1.11</td>
<td>0.82-0.89</td>
</tr>
<tr>
<td>NTP</td>
<td>0.97-0.99</td>
<td>2.69-3.10</td>
<td>0.74-0.80</td>
</tr>
</tbody>
</table>

As it can be seen from table 1, the values of $k_{Fy}$ are always lower than unity and range from 0.71 to 0.99, values for $k_{\delta y}$ are generally far higher than unity and range from 0.84 to 3.10 while for $k_{\delta u}$ are generally lower than unity and range from 0.63 to 1.39.

4 Design with the use of monolithic factors

For practical reasons, it is not acceptable to construct jackets without taking any special connecting measure at the interface between the old element and the jacket. Therefore, from the above results, only cases R, D, RD and W should be considered for design purposes and procedures NT$_1$, NT$_2$ and NTP should be considered for interest purposes only.

In design, when considering strength, it is conservative to accept the lowest possible values. However, this is not the case when deformation is considered. For this situation, using the average value for deformation at the yield and at the failure stage appears to be more conservative.

Table 2: Proposed design values of $k_{Fy}$, $k_{\delta y}$ and $k_{\delta u}$, for columns strengthened with concrete jackets.

<table>
<thead>
<tr>
<th>Type of Jacket</th>
<th>$k_{Fy}$</th>
<th>$k_{\delta y}$</th>
<th>$k_{\delta u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>R</td>
<td>0.85</td>
<td>1.80</td>
<td>0.80</td>
</tr>
<tr>
<td>D</td>
<td>0.80</td>
<td>1.75</td>
<td>0.80</td>
</tr>
<tr>
<td>RD</td>
<td>0.90</td>
<td>1.50</td>
<td>0.80</td>
</tr>
<tr>
<td>W</td>
<td>0.85</td>
<td>1.35</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Following on from the above considerations, the values for the monolithic coefficients $k_{Fy}$, $k_{\delta y}$ and $k_{\delta u}$, that can be provisionally suggested for design are presented in Table 2. It must be stressed that the proposed values in table 2 have been drawn from a limited database of experimental results. Therefore, many more experimental results are required, based on different section properties and
dimensions, before accurate values for monolithic coefficients can be proposed for design and code use.

Therefore, for a flexure-controlled reinforced concrete element strengthened with a concrete jacket, the element’s capacity curve could be determined in terms of bending moment, $M$, and chord rotation, $\theta$, by using the values of the monolithic factors $k_{FY}$, $k_{\delta y}$ and $k_{\delta u}$, that are proposed in table 2, as follows:

$$M_y = k_{FY} M_{y, M} ,$$

$$\theta_y = k_{\delta y} \theta_{y, M} ,$$

and

$$\theta_u = k_{\delta u} \theta_{u, M} .$$

where: $M_y$ and $M_{y, M}$ are the bending moments at the yield stage for the strengthened and the monolithic elements respectively, $\theta_y$ and $\theta_{y, M}$ are the chord rotations at the yield stage for the strengthened and monolithic element’s section respectively and $\theta_u$ and $\theta_{u, M}$ are the respective failure stage chord rotations for the and monolithic strengthened element’s section.

Chord rotations are defined as the angle between the tangent to the axis of the element and the chord connecting the maximum stressed section (where $M_y$ develops) and the end of the shear span. Bending moment $M_{y, M}$ can be easily calculate through either conventional design equations (CEB/fib [12]) or other more accurate expressions are provided by fib [13]. Chord rotation $\theta_{ym}$ can be computed from semi-empirical expressions provided in many references in the literature. However, it is the author’s opinion that the expressions provided by fib [13] are the most accurate.

Finally, the secant to yield stiffness of the strengthened element can be calculated from $M_y$ and $\theta_y$, as follows:

$$E I = \frac{M_y}{3 \theta_y} L_s$$

where $L_s$ equals $M/V$ and is the shear span of the strengthened element.

The relevant values proposed for design in the draft version of part 3 of EC8 [14], are in the range of 0.9 to 1.0 for $k_{FY}$, 1.05 to 1.25 for $k_{\delta y}$ and equal to 1.0 for $k_{\delta u}$. By comparing these values with the relevant ones presented in table 2 above, it is obvious that the proposed code values are in reasonable agreement for the case of the strength coefficient, $k_{FY}$. This is not the case for the displacement coefficients, as values for $k_{\delta y}$ are underestimated while values of $k_{\delta u}$ are overestimated.

## 5 Conclusions

The following conclusions can be drawn from this paper:

(a) A method has been outlined to seismically design strengthened reinforced concrete elements within the environment of displacement capacity assessment design,
(b) The method involves the practical use of monolithic behaviour coefficients,
(c) Existing experimental results have been evaluated and displacement and
strength design values for monolithic behaviour coefficients have been proposed and
(d) The proposed design values have been drawn from a limited database of
experimental results and much more experimental research is needed to confirm
the proposed values.

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References

[5] GRECO, Greek Retrofitting Code, draft version by the Greek Organization for Seismic Planning and Protection, Greek Ministry for Environmental Planning and Public Works, Athens, (in Greek), 2004,


