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FLEXURAL BEHAVIOUR OF RETROFITTED RC BEAMS

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ABSTRACT

A practical and cost-effective technique for increasing the flexural capacity of reinforced concrete beams involves the casting of new plain concrete or cement based layers on the compressive side of beams. In this paper, the results from theoretical and experimental investigations, justify the effectiveness of the technique. The flexural capacities of all strengthened beams were almost the same as the capacities expected from monolithic beams. Moreover, enhanced deformation characteristics were observed in most of the examined cases. The flexural capacity of the strengthened beams was influenced neither by the bonding mechanism employed at the interface between the initial beam and the new layer nor by the material type of the new layer. However, the deformation characteristics were considerably affected. In specimens in which interface preparation involved just roughening of the existing surface, the ultimate deflections recorded were very low, even lower than those in the corresponding unstrengthened beams. In the case of strengthening damaged beams without any previous repair, the stiffness was found to be very low. Moreover, the cracking load was very low, even lower than the unstrengthened control beams.

KEYWORDS

Retrofitting; concrete strengthening; flexural capacity; concrete layer; cement grout.

INTRODUCTION

Many existing reinforced concrete (RC) structures in high seismic risk areas, designed prior to the advent of advanced seismic design codes, require extensive retrofitting to render them safe. Two different techniques, commonly used in practice, for RC beam flexural capacity enhancement, are chosen for discussion. The first technique involves bonding of steel plates on the tensile side of the element, while the second technique involves a cement based layer being added to the tensile or compressive side of the member. In both techniques the crucial point is the bond between the old and the new element. In practice, the first technique seems to be more popular than the second, since it is less time consuming and can be performed with a minimum interruption of use of the structure during the phase of retrofitting. However, two major shortcomings of the technique can be identified. One is the considerable stress concentration at the ends of the plates, which may lead to a brittle failure due to peeling of the steel plate along with the member concrete cover. The second shortcoming relates to the sensitivity of steel to corrosion, which leads to a considerable shortening of the structure's life. Finally, it must be pointed out that this technique is not being used extensively for strengthening the compressive side of a flexural element. On the other hand, adding a new layer of concrete could be an ideal strengthening technique, when parameters other than time consumption and the structure's unavailability during intervention are concerned. This can be the case when taking into account the available relevant experience of the local contractors, the durability of the retrofitted element, the reliability of analytical predictions and, finally, the cost of intervention. Although this intervention technique is being considerably used in practice, the design procedure has been left to engineering judgment. A limited experimental information is available (CEB GTG 12, 1983; Vassiliou, 1975; Dritsos and Pilakoutas, 1995; Dritsos, 1996) and analytical tools are very scarce (Saiidi *et al.*, 1990; Dritsos, 1994).

At the Department of Civil Engineering, University of Patras, a research program which has been in progress since 1993, investigates the behaviour of RC beams, columns and frames strengthened by the addition of new concrete layers. Results concerning strengthening of beams on the tensile side, have already been presented elsewhere (Dritsos, 1996). In this paper, the experimental results obtained from tests on beams strengthened by new compressive overlays are presented. An analytical procedure based on a recently developed model (Dritsos, 1994) is used to predict the response of the strengthened beams. Then, the specimen behaviour is compared with the analytical predictions and with the behaviour of unstrengthened beams.

EXPERIMENTAL WORK

Eleven prismatic RC specimens, of 1000 mm length, were constructed horizontally in steel moulds. The beam cross-section used was 70 mm x 130 mm with two steel bars of 5.8 mm diameter as tensile reinforcement. Three specimens were used as control specimens from a previous study (Dritsos, 1996). Stirrups of 5.8 mm diameter steel bars, at a spacing of 60 mm, were used in all the specimens, except in the central portion of 150 mm length, as shown in Fig. 1. The yield and ultimate strength of the bars were found to be 345 MPa and 435 MPa, respectively. Twelve (100 mm) cube specimens were taken at different casting stages for testing the concrete strength. The concrete strength was found on the day of testing to be between 25 and 35 MPa (see Table 1 for details).

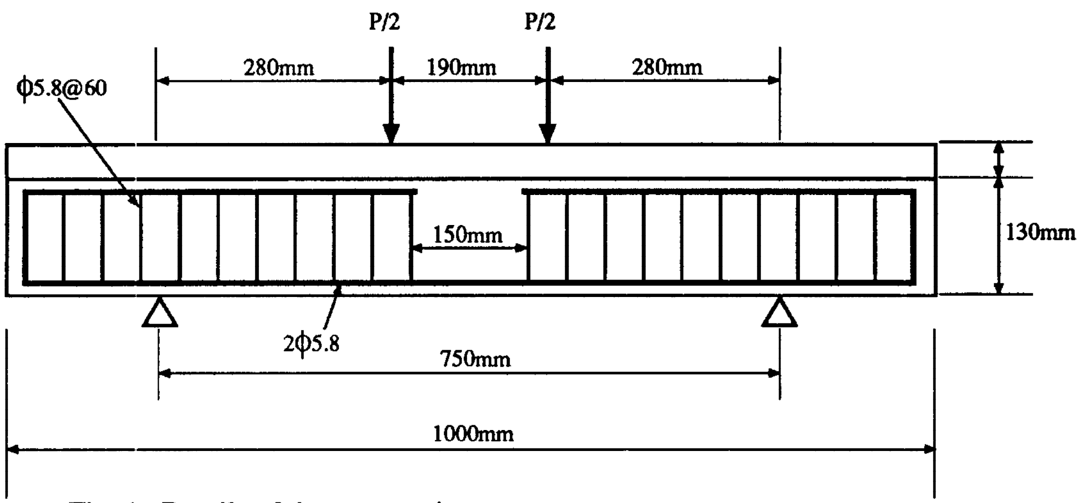


Fig. 1 Details of the test specimens

All the specimens were strengthened on the compressive side, by casting 20 mm unreinforced concrete or cement grout overlayers. The compressive strength of the concrete and non-shrinking cement grout overlayers was found, on the day of testing, to be around 35 MPa 60 MPa, respectively (see Table 1 for details). Two beams were strengthened after being tested to failure in bending, whilst seven others were tested without any previous damage. Casting of the new layers was done five moths after casting the original specimens. The original beam specimens were kept under water for six hours before the application of the new layers, but were thoroughly dried on the surface before casting the new layer.

Three different bonding procedures were used at the interface between the original beam and the new overlayer as explained below:

1. The first procedure involved roughening of the connecting surface of the beam specimen, as recommended in paragraph 11.7.9 of the ACI code (1989). However, steel dowels of yield strength 520 MPa were used at the interface as an additional bond mechanism. The length and diameter of the dowels were 30 mm and 3 mm, respectively.
2. The second procedure involved the use of an epoxy resin adhesive as a bonding agent.
3. The third procedure involved only roughening at the connecting surface, in the same way as the first procedure.

Two different types of cement based materials were used, as new casting layers. One was a non-shrinking cement grout material commonly used in practice for other applications, as for example to fill existing holes in concrete elements. The second type was conventional concrete, with a 10 mm maximum aggregate size.

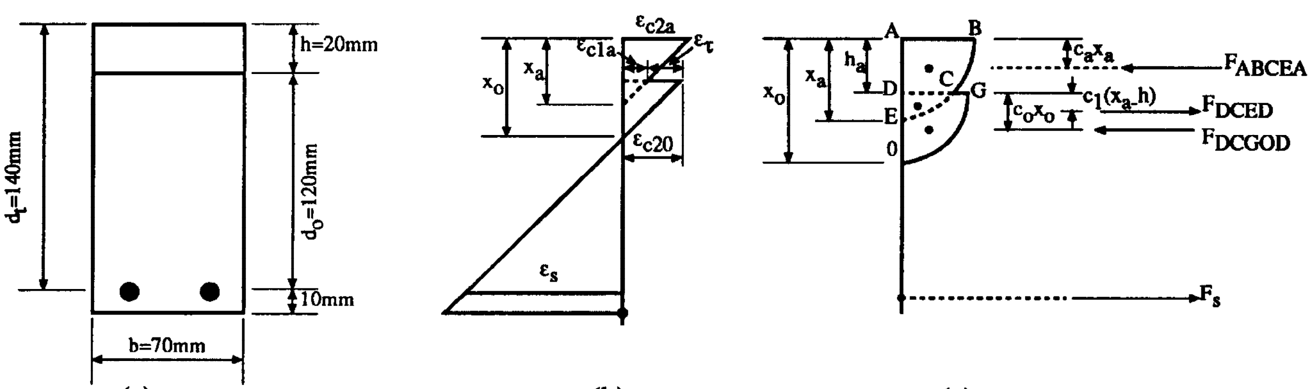
Table 1 Details of specimen tested

Type	Number	Condition of Original Beam	Bonding Procedure	Original Beam Concrete Strength (MPa)	Material of Layer Strength , f_{ca} (MPa)
CA1	2	Undamaged	Roughening and Dowels	25	Concrete 35
CA2	2	Undamaged	Roughening and Dowels	32	Non-shrinking grout 55
CB1	2	Undamaged	Epoxy Resin	30	Concrete 35
CB2	2	Undamaged	Epoxy Resin	33	Non-shrinking grout 65
CC1	1	Undamaged	Roughening	25	Concrete 35
DA1	1	Damaged	Roughening and Dowels	32	Concrete 35
DA2	1	Damaged	Epoxy Resin	32	Concrete 35
O	3	Undamaged	-	35	-

The tested specimens are classified into eight groups, as shown in Table 1. Specimens in which roughening and steel dowels were used in the bonding procedure are denoted as type A. Specimens in which an epoxy adhesive was used as a bonding agent are denoted as type B. Specimens in which only roughening was used as bonding procedure are denoted as type C. Letter D is used when strengthening was undertaken on previously damaged (failure in flexure) specimens. Control specimens (unstrengthened) denoted by the letter O are included from a previous study (Dritsos, 1996). The two different types of materials used for the new layer are denoted by the numbers 1 and 2. Specimens were tested in bending as simply supported beams of a 750 mm span, two months after casting the layers. Two equal point loads were applied 280 mm away from each support, in increments of 5 kN, at a steady rate. Before loading to failure, a loading-unloading test was carried out, up to a load not exceeding 40% of the expected failure load. Measurements included the total applied load, P, and the mid-span deflection.

ANALYSIS OF TEST SPECIMENS

Initially, it is assumed that the slip strain at the interface is lower than the top fibre concrete strain of the original beam. Therefore, the strain profile shown in Fig 2b is adopted. Furthermore, by considering a parabolic-rectangular stress-strain relationship for the concrete, the stress block profile that is shown in Fig. 2c, is obtained. However, if subsequent calculations result in different profiles, an iterative procedure may be necessary, considering a different strain profile in which the lower part of the overlayer is in tension.
The geometrical dimensions of the cross-section b, h, d_o, d_t, etc., are defined in Fig. 2, together with the concrete and steel strains and forces.



(a) (b) (c)

Fig. 2 Strain and Stress Profiles

By considering the whole "composite" cross-section, the following equations are obtained from the equilibrium between the internal forces and the applied moments (M_s) and forces (action effects).

$$F_{ABCEA} - F_{DCED} + F_{DCGOD} - F_S = 0 \tag{1}$$
$$M_S = F_{ABCEA} (d_t - c_a x_a) - F_{DCED} [d_o - c_1 (x_a - h)] + F_{DCGOD} (d_o - c_o x_o) \tag{2}$$

Where x_a, x_o are the neutral axis depths of the overlayer and the original beam, respectively, and c_a, c_1 and c_o are coefficients specifying the centre of the compressive blocks ABCEA, CDEC and DCGO, respectively.

The interface interaction is usually assumed such as to allow only longitudinal slip and vertical separation (Saidi *et al.*, 1990; Dritsos, 1994). As a result, during bending, the curvature (k) of the two elements can be considered the same. Therefore, the following expressions are obtained:

$$\varepsilon_{c2a} = k\, x_a \tag{3a}$$
$$\varepsilon_{c1a} = k\, (x_a - h) \tag{3b}$$
$$\varepsilon_{c2o} = k\, x_o \tag{3c}$$
$$\varepsilon_s = k\, (d_o - x_o) \tag{3d}$$

Furthermore, the following geometrical relations can be obtained from Fig. 2b

$$\varepsilon_\tau = \varepsilon_{c2o} - \varepsilon_{c1a} \tag{4}$$
$$k\, h = \varepsilon_{c2a} - \varepsilon_{c1a} \tag{5a}$$
$$k\, d_o = \varepsilon_s - \varepsilon_{c2o} \tag{5b}$$

Where, ε_τ is the slip strain at the interface.

The internal forces can be expressed as functions of strains, using conventional RC analysis techniques. Moreover, taking into account equation (3), the following expressions are obtained:

$$F_{ABCEA} = b\, x_a\, f_{ca}\, a_a = b\, a_a\, \varepsilon_{c2a}\, f_{ca} / k \tag{6}$$
$$F_{DCED} = b\, (x_a - h)\, f_{ca}\, a_1 = b\, a_1\, \varepsilon_{c1a}\, f_{ca} / k \tag{7}$$
$$F_{DCGOD} = b\, x_o\, f_{co}\, a = b\, a_o\, \varepsilon_{c2o}\, f_{co} / k \tag{8}$$
$$F_S = p.\, b\, d_o\, \sigma_s \tag{9}$$

where, f_{ca} and f_{co} are the compressive strengths of the overlayer and the original beam, respectively, $p = As / bd_o$, is the steel ratio of the original beam, σ_s is the steel stress and $a_a = a_a(\varepsilon_{c2a})$, $a_1 = a_1(\varepsilon_{c1a})$, $a_o = a_o(\varepsilon_{c2o})$ are coefficients specifying the mean value of the compressive blocks ABCEA, DCED and DCGOD, respectively.

By taking into account equations (3), (6), (7), (8) and (9), equation (1) can be rewritten as following:

$$f_{ca} (a_a\, \varepsilon_{c2a} - a_1\, \varepsilon_{c1a}) + f_{co}\, \varepsilon_{c2o}\, a_o - p\, (k\, d_o)\, \sigma_s = 0 \tag{10}$$

Furthermore, by considering the equilibrium of the internal forces on the cross-section of the original beam only, the following expression is obtained:

$$T = F_s - F_{DCGOD} \tag{11}$$

where, T, the shear force at the interface, is expressed as:

$$T = \tau\, b\, (\lambda d_o) \tag{12}$$

where $\lambda = l_x / d_o$ and l_x is the distance of the two point loads from the supports of the specimens (l_x = 280 mm), $\tau = \tau(\varepsilon_\tau)$ is the shear stress at the interface, which can be expressed as a function of the slip strain (ε_τ). This relationship has not been well established yet, since the available experimental data shows a considerable scatter. However, the up to date results indicate that an elastoplastic relationship is reasonable (CEB GTG 12, 1983; Saiidi *et al.*, 1990; Dritsos and Pilakoutas, 1995). Therefore, the following equations are adopted:

$$\tau = K_\tau\, \varepsilon_\tau \quad \text{if} \quad \varepsilon_\tau < \tau_u / K_\tau \tag{13a}$$
$$\tau = \tau_u \quad \text{if} \quad \tau_u / K_\tau < \varepsilon_\tau < \varepsilon_{\tau u} \tag{13b}$$
$$\tau = 0 \quad \text{if} \quad \varepsilon_\tau > \varepsilon\tau_u \tag{13c}$$

where, K_τ expresses the interface bond stiffness, $\varepsilon_{\tau u}$ is the ultimate slip strain and τ_u is the ultimate shear stress.

In the present study, the following values for ultimate slip strain and shear strength of the interface are assumed, according to recent experimental data obtained from pure shear tests (Dritsos, 1996):

$\varepsilon_{\tau u} = 3\ 10^{-3},\ \tau_u = 2$ MPa,

when concrete is cast against roughened concrete without any additional bonding mechanism;

$\varepsilon_{\tau u} = 2\ 10^{-4},\ \tau_u = 3$ MPa,

when an epoxy adhesive is used as a bonding agent at the interface and

$\varepsilon_{\tau u} = 8\ 10^{-3},\ \tau_u = 4$ MPa,

when concrete is cast against roughened concrete and steel dowels are used as an additional bonding mechanism.

The interface shear stiffness (K_τ) is considered, for any bonding procedure, in a range of values between 2000 MPa and 80000 MPa.

In the present study, the maximum value of the tensile force F_s is 23 kN, for all the strengthened specimens. Therefore, the upper limit of the magnitude of the interface shear stress can be calculated to be equal to 1.17 MPa. As a result, the slip strain cannot exceed the yield slip strain, in any of the tested specimens. Therefore equation (13a) is the only one valid among equations (13). Consequently, taking into account equations (4), (8), (9), (12) and (13a), equation (11) can be rewritten as:

$$\lambda\ K_\tau\ (kd_o)\ (\varepsilon_{c2o} - \varepsilon_{c1s})\ f_{cs} + a_o\ \varepsilon_{c2o}\ f_{co} = p\ (kd_o)\ \sigma_s \qquad (14)$$

Equations (5a), (5b), (10) and (14) involve five unknown quantities, e.g. the strain magnitudes ε_s, ε_{c2s}, ε_{c2o}, ε_{c1s}, (kd_o). By definition, at a limit state, one of the four strains ε_τ, ε_{c2o}, ε_s and ε_{c2s} is controlling the member behaviour. Usually, the steel strain ε_s or the concrete strain ε_{c2s} is critical. In the present case, the steel strain ε_s was found to be critical for every case. This could be expected since none of the specimens was overeinforced. Therefore, the system of the four equations can be solved by considering strain ε_s to be equal to 0.001.

It is important to note that the interface slip strain, ε_τ, obtained from the above analysis was very low and was never found to be higher than 0.0005 in the whole range of the values of K_τ considered in the analysis. As a result, the flexural capacity of the "composite" beam was found from equation (2) to be almost the same for all the specimens. The same result was also obtained in the case of strengthening RC beams with new tensile layers (Dritsos and Pilakoutas, 1995; Dritsos, 1996). Even though this result cannot be generalised since it was obtained under specific conditions, it seems reasonable to be valid for beams that are not over-reinforced.

The average value of the flexural capacity of the strengthened beams was found to be 3.02 kNm. This value is 15% higher than the theoretical flexural capacity of the unstrengthened control specimens and almost the same as the one obtained for monolithic beams.

EXPERIMENTAL RESULTS AND DISCUSSION

The cracking load, Pr, together with the failure load ,Pmax, and the ultimate deflection ,Δu, for the specimens, are shown in Table 2. The force versus mid-span deflection curves are shown in Fig. 3. All types of specimens failed in a flexural mode, as predicted analytically. In some cases (specimens CA1-1, CA1-2, CB1-1, CB2-1, CB2-2), partial interface separation occurred which, however, did not seem to affect the flexural capacity of the specimens significantly. This observation was not in line with previous analytical predictions (see previous paragraph) and should be attributed to the conditions of preparation of the connecting surface and curing of the overlayers. It should be noted that similar observations were reported in the case of strengthening with tensile layers (Dritsos 1996). Therefore, the sensitivity of the operation to details of construction is very important.

From Table 2 and Fig. 3, it can be concluded that the applied strengthening technique is very effective, since the observed enhancement of flexural capacities was almost the same as would be expected if the beams were monolithic. It can also be noticed that a considerable enhancement of the deformation characteristics was achieved in many cases. The highest ultimate deflections were observed when a non-shrinking grout was used as an additional layer. In this case, the ultimate deflections were found to be almost twice that of the unstrengthened control beams. Moreover, in most of the cases, the stiffness was increased considerably.

Table 2 Table of results

Specimen	P_r(kN)	P_{max} (kN)	Δu(mm)	Failure mode
CA1-1	10-15	23	11.5	Flexural and Interface Separation @ P = 20 kN
CA1-2	10-15	27	23.5	Flexural and Interface Separation @ P = 22.5 kN
CA2-1	10-15	24.5	39.5	Flexural
CA2-2	10-15	24	40	Flexural
CB1-1	10-15	23	25.5	Flexural and Interface Separation @ P = 17 kN
CB1-2	15-20	25.5	37	Flexural
CB2-1	10-15	21.5	40	Flexural and Interface Separation @ P = 21 kN
CB2-2	10-15	23	34.5	Flexural and Interface Separation @ P = 22 kN
CC1	15-20	23	13.5	Flexural
DA1	5-10	22.5	20	Flexural
DB1	5-10	24	21.5	Flexural
O-1	10-15	19	20.5	Flexural
O-2	10-15	20	21	Flexural
O-3	10-15	18	20	Flexural

In the following, some of the most important observations are pointed out and discussed:

(a) For all strengthened specimens tested, the flexural capacity was found to be at least 13% higher than the capacity of the control beams. The average flexural capacity was found to be 6.5% higher than the analytical predictions. A similar observation has been reported in the case of strengthening beams on the tensile side (Dritsos, 1996). An explanation on the above discrepancy, could be that the actual diameter of the steel bar is higher than the value of ϕ=5.8 mm (lowest diameter along the bars) considered in the calculations. The low shear ratio can also affect the flexural capacity, due to the spreading effect of the load.

(b) The flexural capacity of the strengthened beams was not much influenced either from the bonding mechanism employed at the interface between the initial beam and the new layer, or from the type of material of the new layer. The highest flexural capacity was found when the bonding mechanism consited of roughening of the connecting interface together with steel dowels. The same result has been reported elsewhere (Dritsos, 1996).

(c) The behaviour of the strengthened beams was not at all influenced by the concrete strength of the original beams. This seems reasonable since the neutral axis depths of the "composite" cross-sections were found to be very small. As a result, compressive stresses were developed only in the additional layer.

(d) Less variation in strength values was observed in type 2 specimens (non-shrinking grout as additional layer), than in type 1 specimens (additional layer made of conventional concrete). Therefore, it can be stated that the behaviour of a beam strengthened by using non-shrinking grout can be predicted better than when conventional concrete is used. The same observation has been made in the case of strengthening with tensile layers (Dritsos 1996).

(e) In the case of specimens C (just roughening of the connecting surface), the ultimate deflections recorded were very low, even lower than the corresponding ones of the unstrengthened beams.

(f) In the case of strengthening damaged beams without any previous repair, the stiffness was found to be very low. The cracking load obtained was the lowest among all the cases studied, even lower than the one for the unstrengthened control beams. However, the ultimate deflections recorded were of the same level as for the control beams.

(g) In Eurocode 8 (1995), the degree of monolithic behaviour of a strengthened "multifacic-composite" element, can be expressed by a factor, denoted by "k_r", which is defined as the ratio of the capacity of the strengthened element to the corresponding capacity of the equivalent monolithic element. In reality, " k_r " is a correction factor which is used in order to simplify the analysis and the design of retrofitted RC structures in which "multifacic" RC elements are involved. From the above results, it is obvious that the value of factor, kr, is almost equal to one. Therefore, the correction factor $k_r = 0.90$, proposed by the Eurocode 8 can be conservatively accepted for any material used in the new layer and for all bonding mechanisms used at the interface.

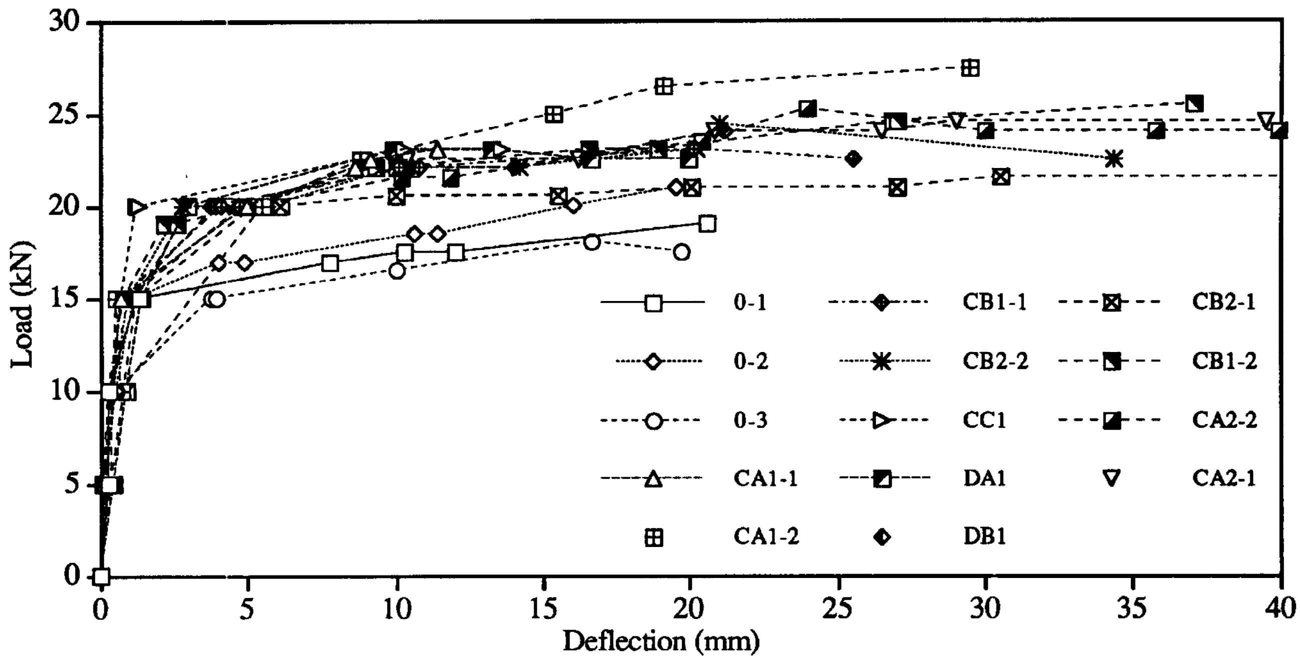


Fig. 3 Load deflection curves

The above observations and results cannot be considered valid for all strengthening situations, since they were obtained under the specific conditions of the project. However, it seems reasonable that they are valid for under-reinforced beams.

CONCLUSIONS

The effectiveness of a strengthening technique which involves the increase in the depth of an RC beam by casting a new concrete or cement based layer on the compressive side of the beam, has been verified from the theoretical and experimental results presented in this study. The flexural capacities obtained for all the strengthened beams were almost the same as the ones expected for monolithic beams. Moreover, enhanced deformation characteristics were obtained in most of the examined cases.

Although the flexural capacity of the strengthened beams was not much influenced either from the bonding mechanism employed at the interface between the initial beam and the new layer, or from the type of the material of the new layer, the deformation characteristics were considerably affected. The same conclusion was obtained when tensile underlayers where examined (Dritsos, 1996). The highest ultimate deflections were observed when a non-shrinking grout was used as an additional layer. In this case, the ultimate deflection was found to be almost twice that of the unstrengthened control beams. Moreover, in most cases, the stiffness was considerably increased.

As far as the moment resistance of the strengthened beams is concerned, the value for the correction factor "k_r" equal to 0.90, as proposed in Eurocode 8 (1995), can be conservatively accepted for any material used in the new layer and for all bonding mechanisms employed at the interface.

In the case of specimens in which the bonding mechanism at the interface was just roughening of the existing surface, the ultimate deflections recorded were very low, even lower than the ones corresponding to the unstrengthened beams. In the case of strengthening damaged beams without any previous repair, the stiffness was found to be very low. The cracking load obtained was the lowest among all the cases studied, even lower than for the unstrengthened control beams, while the ultimate deflections recorded were at the same level as those obtained from the control beams.

The structural behaviour of the strengthened beams was found to be influenced very much by construction details and errors.

All the above observations and results cannot be considered valid for any strengthening situation, since these were obtained under the specific conditions of the project. However, it seems reasonable that they are valid for under-reinforced beams.

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