Towards a European code for seismic assessment and strengthening of existing buildings

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Abstract

In Europe, actions towards preparing a code document regarding assessment and strengthening of existing reinforced concrete structures are already present in the draft document Part 3 of EC8. However, for the case of strengthening by the addition of new reinforced concrete, specific provisions, to check the capacity of the connections between contact surfaces, are missing.

Structural design of strengthened concrete elements can be placed into the framework of the currently known processes of design, which are used for new constructions, supplemented by a crucial investigation at the interface between the contact surfaces, to ensure that failure in each strengthened element precedes failure at the interface, between the old and the new material. For that reason, shear forces and shear resistances at the interfaces, between the old and the new element, must be examined. An evaluation of the shear force that develops between the contact surfaces can be obtained in a similar way as for steel and concrete composite structural elements. The main mechanisms that contribute to the shear resistance at the interface are: (a) concrete-to-concrete adhesion, (b) concrete-to-concrete friction, (c) connecting action from steel bars placed across the interface between the old and the new concrete and (d) bent steel bars welded between the bars of the old and the new concrete. The total shear resistance between contact surfaces can be found by summing the individual shear resistances that are mobilised by each individual mechanism for a common interface slip. To prevent a brittle failure at the interface, a minimum amount of steel shear connectors in the form of dowels or bent steel bars must be provided.
Keywords: aseismic code, assessment, buildings, design, reinforced concrete, repair, retrofitting, strengthening.

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1 Introduction

Various methods and techniques are used in practice to enhance the seismic capacity of reinforced concrete (R.C.) structures (fib [9]; Dritsos [5, 6]; Tsonos [16]; CEB Bul. N$^{o.}$ 162 [3]). However, analytical tools to manipulate the subject are rare and the absence of a specific design code, regarding retrofitting of existing old structures makes a complex and difficult problem (Tassios [12]; Tsonos [15]; Apostolopoulos [1, 2]).

In Europe, actions towards preparing a code document regarding assessment and strengthening of existing R.C. structures, has already present a draft document of EC8-Part 3 [7] revising the existing Part 1.4 of EC8 [8]. However, for the case of strengthening by the addition of new reinforced concrete, specific provisions, to check the capacity of the connections between contact surfaces, are missing.

In the following, supplemental relevant material, regarding the above issue, is provided for possible use in the final code edition.

2 Control of a sufficient connection between contact surfaces

Structural design of strengthened concrete elements can be placed into the framework of the presently known processes of design that are used for new constructions, supplemented by a crucial investigation at the interface between the contact surfaces, to ensure that, failure in each strengthened element precedes failure at the interface, between the old and the new material. (Tassios [13]; Chronopoulos [4]; EC 8 [8]; GRECO [10]; Dritsos [6]).

Load transfer mechanisms between the old and new materials must be capable of transferring the tensile, compressive and shear stresses that develop at the interface.

As far as interface tensile stresses are concerned, the transfer can be guaranteed if the developed stresses are lower than the tensile strength of the weakest concrete. If not, an appropriate quantity of reinforcement or anchor bars crossing to the contact surface should be provided, as specified later in this paper.

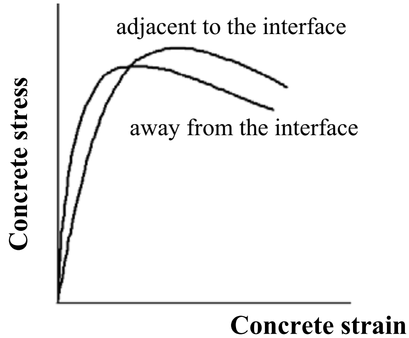


Figure 1: Stress against strain diagram for interface concrete.

Regarding concrete-to-concrete direct compression, a full continuity compression transfer can be expected at the interface if adequate treatment measures have been performed on the old concrete surface (such as roughening). However, as shown in figure 1 (CEB Bul. N$^{o.}$ 162 [3]), a lower modulus of elasticity should be considered for concrete adjacent the interface, as higher deformations develop due to the mechanical treatment of the existing concrete and contact and compaction imperfections. Obviously, the interface compressive strength can be considered to equal the lowest compressive strength of the contact materials.

In order to guarantee a sufficient connection between contact surfaces, the check for safety at the ultimate limit state can be expressed symbolically by the following equation of safety:

$$S_d \leq R_d \tag{1}$$

where: S_d is the design action effect and R_d is the design resistance. This control will include checking the shear force and the shear resistance at the interface between the old and the new element. That is to say, the following relationship must be satisfied:

$$V_{Sd}^{interface} \leq V_{Rd}^{interface} \tag{2}$$

where: $V_{Sd}^{interface}$ is the shear force acting at the interface and $V_{Rd}^{interface}$ is the shear resistance at the interface.

Obviously, a guaranteed connection that avoids premature failure would be desirable. This would be because it represents the critical factor for the effectiveness of the intervention and would ensure an acceptable degree of reliability for calculations.

If failure between the contact surfaces precedes failure of the strengthened element, the load bearing capacity of the connection will determine the load bearing capacity of the strengthened element. In addition, the load bearing capacity of the strengthened element cannot be considered smaller than that of the original unstrengthened element.

The control between contact surfaces along the whole length of the strengthening structural element should be based on average values of $V_{Sd}^{interface}$ and $V_{Rd}^{interface}$ corresponding to various segments of length l_{i-j} (i and j for successive segments) into which the element has been divided. That is to say:

$$V_{Sd(i-j)}^{interface} \leq V_{Rd(i-j)}^{interface} \tag{3}$$

The length of each successive segment should not be greater than twice the height of the cross section of the element. However, the process can be facilitated if the lengths of segments are also fixed at characteristic cross sections. As such, sections dividing an element should be placed at the following locations: (a) at the largest positive or negative bending moment, (b) at the supports, (c) at positions of point loads, (d) where there are abrupt changes in cross section and (e) at the ends of cantilevers.

2.1 Shear forces acting at the interface

An evaluation of the shear force that develops between the contact surfaces can be obtained by analysing each segment of the strengthened element assuming monolithic behaviour (by approximately calculating the shear stress at the interface using mechanics theory). Alternatively, the more accurate calculation method that is applied for steel and concrete composite structural elements could be used. Figure 2 schematically illustrates the shear force that develops between contact surfaces.

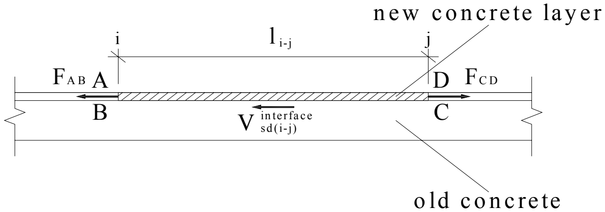


Figure 2: Shear force at the interface.

If a structural element has been strengthened with the new layer of concrete, the size of the shear resistance between the contact surfaces, for a segment length of l_{i-j}, can be determined by considering the equilibrium of forces in the new concrete segment ABCDA of figure 2. That is:

$$V_{Sd(i-j)}^{interface} = V_{Sd}^{BC} = F_{AB} - F_{CD} \tag{4}$$

A process of section analysis can be used to determine the magnitudes of the forces F_{AB} and F_{CD}. That is, by taking sections through the whole element at positions i and j respectively and determining the internal tensile or compressive forces corresponding to layer sections AB or CD.

2.2 Interface shear resistance

Four mechanisms contribute to the shear resistance at the interface. These are concrete-to-concrete adhesion, concrete-to-concrete friction, the connecting action from either steel bars placed across the interface between the old and the new concrete or bent steel bars welded between the bars of the old and the new concrete. These four mechanisms can be subdivided into the two groups of unreinforced and reinforced interfaces, depending on whether or not additional steel is placed across the interface or welded between the bars of the old and the new concrete. In general, the shear resistance developed at the interface depends on the amount of slippage at the interface.

2.2.1 Unreinforced interface shear resistance

The two mechanisms acting at an unreinforced interface are adhesion and friction. Figure 3 (CEB Bul. N$^{o.}$ 162 [3]), presents a plot of the mobilised shear resistance (τ) against interface slip (s) and it can be seen that the maximum adhesion values are achieved for low interface slip values (in the region of

0.02 mm), while friction becomes important for much higher slippages. Therefore, the maximum resistances from adhesion and friction do no coincide and cannot be considered to act together.

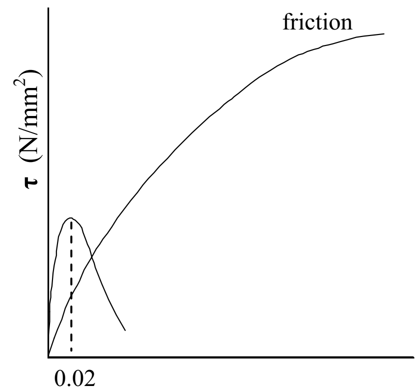


Figure 3: Mobilised shear against slip.

The main parameters affecting the adhesion at the interface are the roughness and treatment of the joint surface and the tensile strength of the weaker concrete. The Greek Retrofitting Code (GRECO [10]) accept the following design values for the magnitude of adhesion at concrete interfaces:

0.25 f_{ck} for smooth interface conditions
0.75 f_{ck} for rough interface conditions
1.00 f_{ck} if a resin bonding agent is used at the interface
1.00 f_{ck} if the additional concrete is shotcrete

where: f_{ctk} is the characteristic value of the tensile strength of the weaker concrete.

The parameters that affect concrete-to-concrete friction are the size and shape of the aggregates if exposed (large angular aggregates are better) and the surface roughness of the original column (rougher surfaces have greater areas of surface contact). Additional parameters include the concrete compressive strength, the external normal compressive stress (a higher normal stress gives a higher shear stiffness) and if the loading is cyclic or not (cyclic loading quickly deteriorates the contact surfaces giving a larger slip or a lower shear response). Representations that model concrete-to-concrete friction (τ_f) can be found in the literature. In an analytical work presented by Tsoukantas and Tassios [17] the following formula was proposed:

$$\left(\tau_f / \tau_{fu}\right)^4 - \left(\tau_f / \tau_{fu}\right)^3 = 0.3s - 0.03 \tag{5}$$

Using the above equation it can be found that the shear resistance due to friction (τ_f) reaches a maximum when the relative slip is in the region of 1.75 mm. Moreover the maximum value of the design concrete-to-concrete shear resistance due to friction (τ_{fu}) can be calculated from the following equation:

$$\tau_{fu} = 0.4(f_c^2 * \sigma_c)^{1/3} \tag{6}$$

where: f_c is the compressive strength of the weaker concrete and σ_c is the interface compressive stress.

2.3 Reinforced interfaces

2.3.1 Clamping action of transversal reinforcement

When a steel bar crosses the interface between old and new concrete, an additional action that may occur is clamping action. This action would take place when the surface of the old concrete has been roughened, or shotcrete has been placed and if the steel bar is adequately anchored. As it is demonstrated in figure 4 (Tassios and Vintzeleou [14]) when a shear stress is applied, a slip is produced and the contact surface between the old and the new concrete must open as one surface rides up over the other due to the roughness.

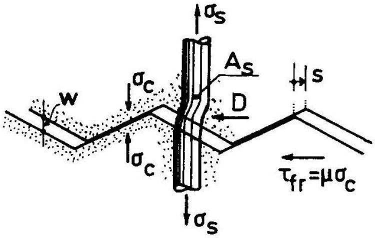


Figure 4: A qualitative description of friction resistance τ_f due to clamping action.

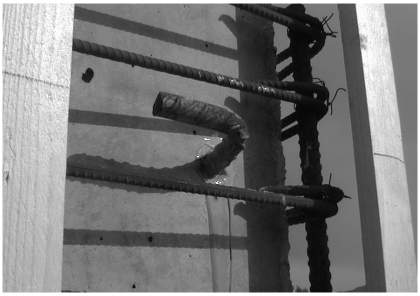
Therefore, a tensile stress is activated in the steel bar, which in turn produces a corresponding compressive stress, or clamping action, and a frictional resistance is mobilised. Equation 6 can be modified in order to take into account the additional frictional resistance mobilised by clamping action, as follows:

$$\tau_{fu} = 0.4(f_c^2 * (\sigma_c + \rho_d f_y))^{1/3} \tag{7}$$

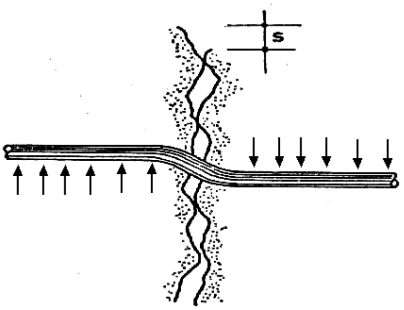
where: ρ_d is the total cross sectional area of the shear connectors divided by the cross sectional area between the contact surfaces and f_y is the yield stress of the transversal bars.

2.3.2 Dowel action of interface reinforcement

Tranversal resistance of steel bars crossing the contact interface (fig. 5), is commonly referred as dowel action. Parameters that affect dowel action include the concrete strengths of the new and the old concrete, the yield stress of the dowel (f_y), the diameter of the dowel (d_b) and the amount of dowels placed. The maximum interface resistance is obtained only if the dowels are adequately embedded in the old and the new concrete at depths of at least 8 times the dowel diameter. In addition, measures should be taken to avoid failure due to placing dowels too close to the edge of the concrete.



(a)



(b)

Figure 5: (a) Use of dowels in concrete jackets, (b) Dowel action.

At least 3, 5 or 6 times the dowel diameter are respectively required from the edge of the original element or the top or base of the original element or jacket if a partial jacket is placed. In an analytical work presented by Vintzeleou and Tassios [18] the following model was proposed concerning one dowel bar:

For small slip values: $\left(V/V_u\right) = 200 * \sqrt{f_c/f_y} * (s/d_b) \le 0.4$ (8)

For higher slip values: $\left(V/V_u\right) = \frac{4}{3} * \sqrt[4]{s/d_b} \le 1.0$ (9)

where: V is the magnitude of the mobilized dowel resistance due to a slippage s, V_u is the maximum value of V and d_b is the dowel diameter.

The maximum value of the design shear resistance from dowel action (V_u) can be calculated from the following equation (Rasmussen [11]; Vintzeleou and Tassios [18]):

$$V_u = 1.3 * d_b^2 * \sqrt{f_c * f_y} \tag{10}$$

If earthquake action is expected, it would be conservative to remove the value of 1.3 from equation (10) (GRECO [10]).

2.3.3 Action of welded bent steel bars

A practice that is commonly used and has a good reputation, is to weld bent steel bars between the reinforcement of the old concrete and the new concrete (fig.6(a)). When there is relative slip between the old and the new concrete, a part of the force in the old bar is transferred to the new bar via the bent bar. Figure 6b conservatively demonstrates the mechanism (CEB Bul. N$^{o.}$ 162 [3]; Tassios [12]). When there is slippage (s) at the interface, one of the angled legs of the bent bar is elongated by a length of s/(√2) while the other angled leg is shortened by the same length. Therefore, the respective tensile or compressive strains (ε_{sb}) and stresses (σ_{sb}) are:

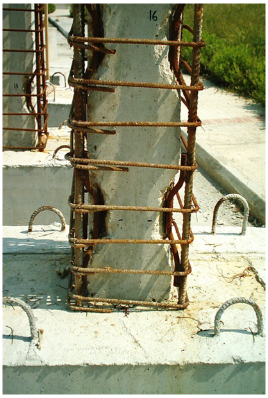
$$\varepsilon_{sb} = \frac{s/\sqrt{2}}{\sqrt{2}h_s} = \frac{s}{2h_s} \text{ and } \sigma_{sb} = E_s \frac{s}{2h_s} \le f_{yb} \tag{11}$$

where: h_s is the distance between the centrelines of the outer arms of the bent steel bar (fig.6b), E_s is the modulus of elasticity for the steel bar and f_{yb} is the characteristic value of the yield strength of the bent steel bar.

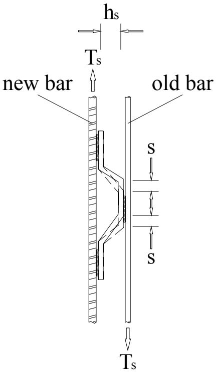
By considering the equilibrium of forces, the force that can be transferred to the new reinforcement (T_s), expressing in other words the shear capacity of the interface, can be derived:

$$T_s = A_{sb} * E_s (s/\sqrt{2}h_s) \leq T_{sy} = \sqrt{2}A_{sb}f_{yb} \tag{12}$$

where: A_{sb} is the cross sectional area of the bent bar and T_{sy} is the force required to yield the weaker longitudinal bar.



(a)



(b)

Figure 6: (a) Use of bent bars in concrete jackets, (b) bent bar model.

3 Conclusions and proposal for design

The total shear resistance between contact surfaces can be found by summing the individual shear resistances that are mobilised by each individual mechanism for a common interface slip. Figure 7a presents a plot of the superposition of slippage from all the mechanisms discussed above for the transfer of shear stress at the interface obtained from available literature (CEB Bul. N$^{o.}$ 162 [3]; Tsoukantas and Tassios [17]; GRECO [10]) and represents typical experimental results. In order to simplify calculations, bilinear diagrams of the type OAB of figure 7(a) could be applied. Elastic simplifications, as in curves OA_1 or OB of figure 7(b), could be used to facilitate the analysis. More precise results could be obtained by using elasto-plastic diagrams such as curve OA_1BB_1 of figure 7(b). In general, the remaining shear resistance (τ_{res}) could be considered as insignificant.

For structural elements that resist seismic actions, it may be useful (and it would simplify calculations) if the mechanisms of adhesion and friction were ignored and only the shear resistance from dowels or other shear connectors is taken into consideration. In other elements that do not resist seismic action (for example concrete slabs), it could be considered that shear connectors are required only when, in some region of the structural element, the shear stress between the contact surfaces exceeds the shear strength from adhesion or friction.

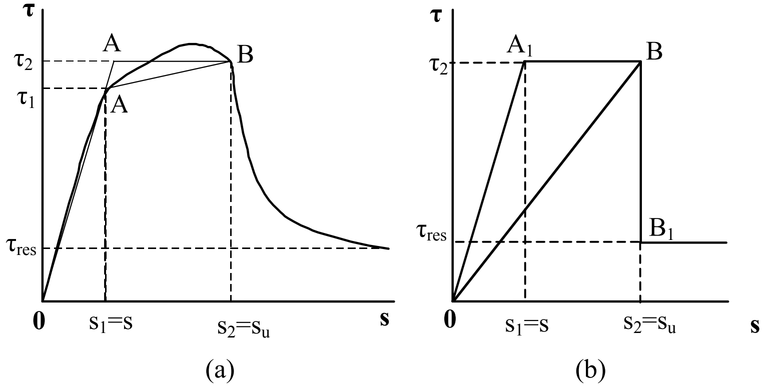


Figure 7: Representations of the longitudinal transfer of shear stress: a) typical experimental result and bi-linear simplification and b) elastic and elasto-plastic simplifications.

In order to prevent a brittle failure at the interface, a minimum amount of steel shear connectors in the form of dowels or bent steel bars are required for concrete-to-concrete connections. The required percentage can be calculated in a similar way to that of determining the minimum shear reinforcement in monolithic elements and the following relationship has been proposed (Dritsos [6]; GRECO [10]):

$$\rho_d \geq \max\,(0.18\ f_{ctm}/f_{yk},\ 0.12\%) \tag{13}$$

where: f_{ctm} is the average tensile strength of stronger concrete and f_{yk} is the characteristic yield strength of the steel shear connectors or bent down bars.

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