Restoration of existing historic concrete structures with PE fiber strain hardening cementitious composites.

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Abstract. Important historic buildings of the 20th century located in the seismogenic zones of the Mediterranean have been designed without the provisions of Eurocode 8; thus, they require strengthening to account for seismic excitations and to be rendered safe for use by future generations. A common disadvantage in historic concrete buildings is the insufficient or even null reinforcement in shear, which usually results in brittle shear failures. This research focuses on the use of a self-compacting, fiber reinforced cementitious composite, which can be applied as cover replacement, or in a thin layer around the perimeter of an existing column, thus providing confinement to the original core and increasing the shear strength of the structural member. Different mix designs prepared in various countries have been examined, all incorporating the use of industrial by-products, such as silica fume or fly ash. The aforementioned mixtures were replicated with the use of locally available raw materials at the laboratories of the University of Cyprus. Polyethylene (PE) fibers, with or without coating, measuring 12 and 18 mm in length were also included in the various mix designs. The mechanical properties of the hardened specimens were compared through compression, tension and flexure tests at 28 and 90 days of curing. Based on the results, a self-compacting mixture, presenting particularly high strengths in direct tension and bending, was eventually chosen to investigate the strengthening of existing historic concrete members.

1 Introduction

The term heritage has only recently been introduced as a concept in human societies, while its scope has even more recently been extrapolated, not only to physical objects, but also to intangible aspects. Until some decades ago, monuments belonging to the architectural heritage were predominantly masonry buildings, constructed sometime between the prehistoric period and the period of modernization. Yet, the inclusion of additional values of interest attributed to cultural and architectural heritage, such as artistic, scientific, technical and social values, has widened the range of structures that may be listed to include those of the modern era, built with contemporary materials, i.e., reinforced concrete, or even structures of the post-modern movement built in the early 1980s.

Even though some structures built in the era of modernity are now considered by experts in the heritage sector an integral part of a country's cultural and architectural heritage, this perception is not widely shared by the community of practitioners, architects and engineers. This is highlighted not only by the loss of multiple important historic concrete structures that were demolished to be replaced, but also by the abandonment of such structures that ought to have been repaired, or even by the extensive alterations carried out on historic reinforced concrete structures that have inevitably changed their original features. Additionally, in many countries there are no specific guidelines for the protection of historic concrete structures, or even systematic listing of such structures by the local authorities, as opposed to the case of their pre-modern counterparts.

1.1 Historic Concrete Repair

The repair of historic structures is lately starting to take legislative form, with the introduction of regulations defining structural rehabilitation procedures. For example, by using tax incentives, the U.S. Secretary of the Interior's Standards for Rehabilitation [1] have provided guidelines for the overall strategies that may be applied to historic structures, recommending ***retaining and repairing, rather than replacing***, existing materials wherever possible. This is also the new tendency proposed by researchers in the field of historic concrete, who have replaced the previously used term reversibility, which originated from art conservation, as it was found unsuitable for building conservation, and have replaced it with the term retreatability [2], which may be defined as ***the procedure signifying that the conservation treatment / repair material should not preclude or impede further treatment in the future*** [2,3].

It is worth noting that materials used in repairs, as in the case of the patch-repairs, where the volume of the deteriorated concrete to be replaced is usually extensive, must be carefully selected, since certain e.g., polymer-modified mortar coatings can reduce moisture penetration to minimum and may trap moisture within the original concrete [4]. The ideal repair material should have a low water absorption, in order to reduce ingress

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of chlorides and carbon dioxide to prevent corrosion, and a high water vapor transmission, in order to evaporate moisture and prevent damage and debonding from freeze-thaw cycles [5]. Additionally, if the repair material will not carry loads, it should have a lower modulus of elasticity than the substrate, while if it will carry loads, its modulus of elasticity should be similar to the parent concrete [6]. When the modulus of elasticity of the repair material is higher than that of the original concrete, there is risk of damage to the latter [6].

Methods for the repair of structural members, especially those of historic character, should be less invasive than e.g., concrete jacketing. At the same time, however, they should generally be able to assist the structural members sustain vertical or lateral loads in a ductile manner, especially in seismic zones where the original design did not include seismic detailing.

1.2 Strain Hardening Fiber Reinforced Cementitious Repair Composites

One of the most prominent techniques that may be used towards this goal, is the confinement of structural members with the use of cementitious composite materials, such as fiber reinforced cementitious composites. The confinement provided by these means increases the compressive strength of concrete, as well as the shear capacity and bond between the longitudinal reinforcement and the surrounding concrete, especially in cases where lack of adequate stirrups may result in brittle failures. Most of these composites offer solutions that do not alter the geometry of the original structural members, which, in the case of historic concrete structures, due to aesthetic reasons, is of outmost importance. Additionally, such repair materials may be self-compacting; this is important for practical application reasons where compaction may not be feasible due to the geometry of the reinforcement or the thickness of the layer [7,8].

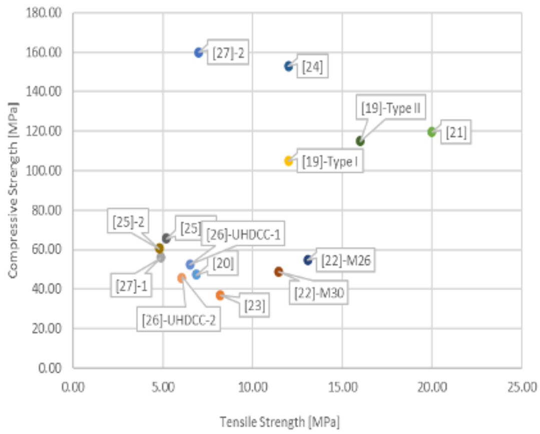
Novel mix designs with the incorporation of short plastic discontinuous fibers in their matrix, that have been investigated since the 1980s, have exhibited strain hardening capacity in tension and may well be the future of cement-based construction. These types of materials have been widely investigated for new large scale constructions, such as bridges, whilst some preliminary research has also shown advantages in the case of crack repairs [9], overlays on bridge decks [10], substrates at the bottom of beams [11–14] or layers around beams [15].

For the repair of existing concrete columns, very limited research has been conducted so far. In most cases, the original material to be repaired had almost the same compressive strength as the repair material, ranging from 30-40 MPa [16–18].

This research focuses on the use of a high performance, self-compacting, fiber reinforced cementitious composite, that may be applied as a repair material, in thin layers, on historic concrete members. The admixtures that are usually incorporated in high performance materials are pozzolans, such as fly ash or silica fume. The ultra-high performance material under investigation in this study additionally has tensile ductility that is provided by the use of short discontinuous (Polyethylene (PE)) fibers. The specific gravity of this material is lower than that of normal concrete, while its high compressive strength allows for the application of smaller covers around existing members. Additionally, its dense matrix results in low permeability and, thus, protection of the steel reinforcement.

2 Selection of repair mix

Through an extensive literature research, a series of strain hardening fiber reinforced cementitious composites (SHFRCC), prepared with the use of PE fibers by different research groups around the world, have been identified, and their performance in terms of tensile strength, compressive strength and ductility was compared, as shown in Fig. 1. Minimum requirements in terms of performance and local availability of materials were set to select some of these mixes to be prepared at the laboratory facilities of the Department of Civil and Environmental Engineering at the University of Cyprus. The various mix designs selected were prepared with different types of cement, sand, fly ash or silica fume and PE fibers. The same aggregates were used throughout, thus facilitating a performance comparison.



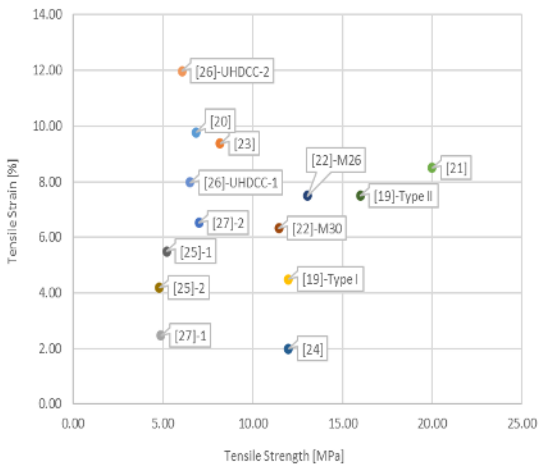


Fig. 1 Performance of SHFRCC from the literature in terms of compressive-tensile strength and ductility in tension [19],[20],[21],[22],[23],[24],[25],[26],[27]

2.1 Experimental program

2.1.1 Raw materials and mix proportions

The mixtures were comprised of cementitious binders, water, silica sand, high range water reducer (HRWR) and high strength, lightweight filament Polyethylene fibers (PE). The cementitious binders included Portland cement (52.5) and industrial by-products, such as fly ash (Type F) and silica fume. Table 1 lists the mix proportions that were selected from the literature and reproduced for this experimental research. The amount of each constituent material is recorded in comparison to the quantity of cement in the mix.

The fibers that were used to produce the mixes were High Density Polyethylene fibers (PE), with (C) and without (U) coating, measuring 12 and 18 mm in length, provided by Minifibers Co. Two fiber volume fractions, 1.5% and 2%, were used in the mixtures. The physical and mechanical properties of fibers are listed in Table 2.

Table 1. Mix proportioning.

	C	W	S.S.	SF or FA*	HRWR	V_f (%)	L_f [mm]
E_1 [19]	1	0.33	0.71	0.21	0.036	2.0	12C
E_2 [27]	1	0.32	0.60	0.67	0.083	2.0	12C
E_3 [23]	1	0.36	0.43	0.43 *	0.004	1.5	18C
E_4 [28]	1	0.44	0.56	0.11	0.022	1.5	12C
E_5	1	0.45	0.50	0.10	0.010	2.0	18C
E_6	1	0.40	0.50	0.10	0.015	2.0	12C
E_7	1	0.45	0.50	0.10	0.010	2.0	18U
E_8	1	0.40	0.50	0.10	0.010	2.0	12U

Table 2. Properties of 12- and 18-mm length PE fibers.

Density [kg/m^3]	970	970
Length L_f [mm]	12	18
Diameter d_f [μm]	17.9	17.9
E [GPa]	114	114
Elongation	0.026	0.026
Aspect ratio (L_f/d_f)	670	1005

The fiber length, surface properties and volumetric percentage influence the final properties of the composite material in terms of strength and ductility. PE fibers have a hydrophobic surface; thus, when crossing a crack, their behavior is mainly dependant on the friction between the fibers and the surrounding cementitious matrix. In that case, pull-out failure is observed, that is relevant to the anchorage length of the fiber bridging the crack. Researchers have demonstrated that PE fibers produce more ductile composites when their surface is coated in order to increase the bond between the fiber surface and the cementitious matrix. In this sense, the fibers engage within the opening of the crack and are able to develop stresses as high as their rupture strength, thus increasing the subsequent generation of multiple cracking, the ductility and the ultimate tensile strength.

2.1.2. Mixture preparation

Fig. 2 Molds for compression/flexure and tension specimens

The experimental procedure had to be carefully designed to ensure proper fiber dispersion within the mixture. Initially, the dry materials, such as silica sand, fly ash and silica fume, were mixed with the cement for 5 min. Then, as the dry materials were mixing, water and HRWR were added to the mix. After the mix obtained self-compacting fluidity, the polyethylene fibers were slowly added to disperse in the entire mix. Mixing of the materials continued until a cohesive and self-compacting composite was achieved. Each mixture was cast into dogbone and small beam shape molds, as shown in Figure 2, and was kept for 24 hours at room temperature. After demolding, all specimens were cured in water for 28 days, until testing.

2.1.3. Testing Procedure

In order to compare the mechanical properties of the mixtures, uniaxial compression strength, direct tensile and 3-point bending tests were performed; three specimens from each mix design were tested. The tensile tests were performed at the age of 28 days, while the compression and flexure tests were performed both at 28 and 90 days.

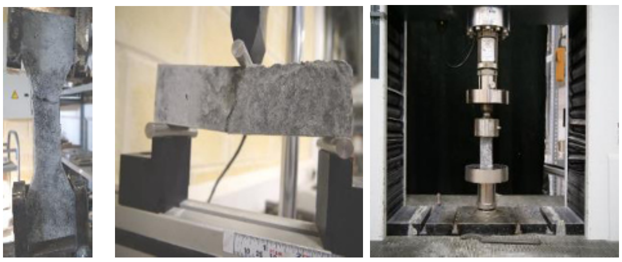


Fig. 3 Experimental setup for uniaxial tension, 3-point bending and uniaxial compression

2.1.3.1. Compressive Strength

Uniaxial compression tests were performed on 40x40x80 mm specimens, as shown in Fig. 3. The load was applied by displacement control with a speed of 0.024 mm/s, for static conditions and in order to obtain the post-peak descending branch of the behaviour of the materials.

2.1.3.2. Tensile Strength

Dogbone specimens were tested under uniaxial tension. Two digital elongation meters (LVDTs) were placed in the center of each specimen, on either side, recording the elongation of the mid 100 mm region that develops multiple cracking. Detailed information about the dimensions of the specimens may be found in [29]. Loading was applied at a displacement control rate of 0.01 mm/s.

2.1.3.3. Three-point bending

The strength under flexure was obtained by bending prismatic specimens, measuring 40x40x160 mm, through loading at the center of their span length (3-point bending). Load control was applied up until the appearance of the first crack, followed by displacement control to specimen failure.

2.2. Experimental results and analysis

The density of the various mixes at 28 and 90 days is recorded in Table 3, as an average resulting from the specimens prior to testing. It is important to note that the density observed is much lower than that of normal concrete currently used in practice, mainly due to the lack of coarse aggregates in the mixes. Yet, this low density is related to a lower modulus of elasticity, that is usually found in old historic concrete, and is thus more appropriate for repair purposes.

Table 3. Density of the various mixes.

Mixture	Density (kg/m^3)	
	28 days	90 days
E_1	1890.85	1900.45
E_2	1782.14	1824.53
E_3	2121.71	2129.69
E_4	2033.76	2028.01
E_5	2009.74	2009.51
E_6	2033.66	2043.93
E_7	2043.56	2012.40
E_8	2048.20	2059.31

2.2.1. Uniaxial Tension

Polyethylene fibers used in the mixes play an important role in avoiding abrupt tensile failure and maintaining a relatively constant strength as the deformation increases. Specimens hereby tested exhibited strain hardening in tension after the first cracking, that is attributed to the tensile strength of the matrix, with the formation of multiple cracking along the critical 100 mm length. After a critical tensile strength, deformation was localized in a single crack (Fig. 4), while the tensile strength gradually reduced, with fibers pulling-out from the critical cross-section.

Fig. 5 shows the tensile stress - tensile strain results of all the mixes at the age of 28 days. All the specimens exhibited a strain hardening behavior of at least 5%, while, in some mixes, strain hardening developed up until a strain of 15%, a result close to the failure strain of steel reinforcement (see for example mix design E3). Additionally, the tensile strength varied from 2 to 6 MPa. The highest stress was obtained in the case of the 18 mm fibers at a ratio of 1.5% by volume, with the use of fly ash as cement replacement (mix design E3). Moreover, the tensile stress-strain curves showed a high energy dissipation even in the descending branch, where crack localization was initiated.

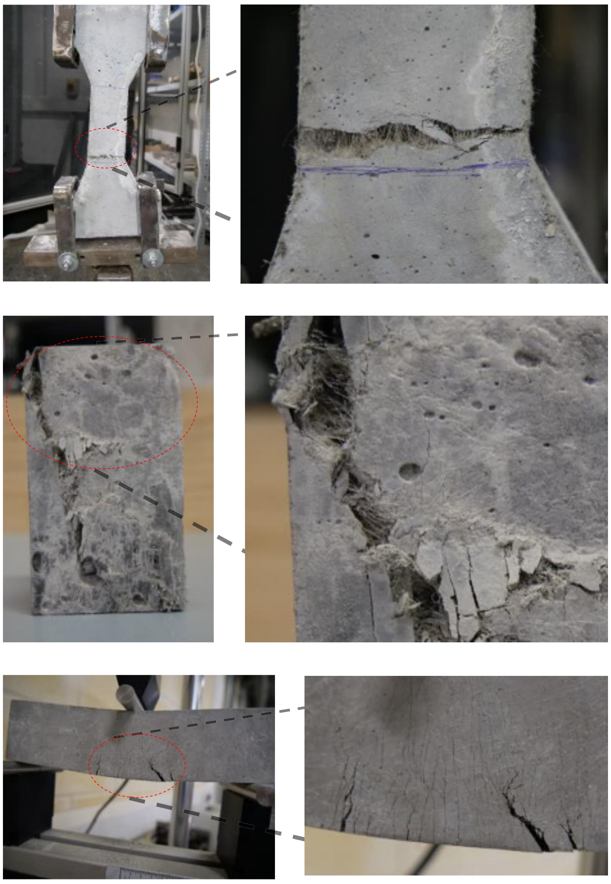


Fig. 4 (Top) Dogbone specimen after failure, (Middle) PE fibers bridging a crack, (Bottom) Multiple cracking during bending test

2.2.2. Compressive Strength

The compressive strength of the mixes was derived from the testing of specimens with a width to height ratio of 1:2 (40:80 mm). The compressive behavior of all the tested specimens at 28 days, in terms of stress-strain curves, is depicted in Fig. 6. As a result of the compressive stresses, failure manifested in a diagonal crack initiating from the top of the specimen and reaching its bottom edge. Within this crack, the significant contribution of the PE fibers was evident, bridging the cracks, as shown in Fig. 4, and reducing the possibility of an abrupt failure. In all cases, the fibers provided a confining effect, by bridging the cracks and transferring lateral stresses. The formation of the usual bell shape failure was avoided, due to the confining effect of the fibers, and the crack pattern resembled a shear type failure.

The peak compressive strength varied between 40 to 86.4 MPa, with the highest compressive strength obtained for mix design E_3. In addition, mix designs E_1 and E_2 had the lowest compressive strengths, 39.60 MPa and 45.90 MPa, respectively. The graphs present systematic ductile behavior in compression, with a slow strength diminishing/descending branch and a residual strength in the order of 10-20 MPa in all cases. The slope of the descending branch is increased with an increase of the compressive strength.

2.2.3. Flexural strength

Figure 4 shows an example of the crack pattern of a specimen in the 3-point bending test, with multiple cracks visible at the tensile zone of the specimen. This pattern developed in all the specimens tested, albeit to a different extent in terms of width of the cracking zone, number of cracks and distance between the cracks. After a certain hardening effect with increase of the load capacity of the beams, localization of deformation was exhibited, leading evidently to failure of the specimen. Equation 1 was used to calculate the flexural stress corresponding to the load.

$$\sigma=3PL/(2bd^2) \qquad (1)$$

where P is the total load applied, L is the span length between the supports, b and d are the width and depth of the beam, respectively. It is important to note that stress calculated from Eq. 1 (engineer value) is not the actual stress developed at the extreme tensile zone of specimen, since this equation is derived from elastic analysis, where the neutral axis of the cross-section is located at the center of the beam depth. These equivalent flexural stress-center point deflection curves for each specimen, at 28 days are depicted in the graphs of Fig. 7, for comparison reasons. The results show that specimens from mix E_3 presented the highest equivalent flexural strength i.e., 23.12 MPa.

2.2.4. Strength increase with age

The use of industrial by-products, such as fly ash and silica fume, in concrete mix design decreases the rate of end-product strength development with age, in comparison to normal concrete. Composites with the aforementioned by-products develop their final strength after 70 days; therefore, in this study, tests were also performed at the age of 90 days for comparison reasons. Figure 8 shows the mean compressive and flexural strengths for all mixes at the age of 28 and 90 days. After 90 days, all specimens showed an increase in compressive strength, with the most extreme increase of 34% recorded for mix design E_6. The same applied in the case of flexural strength, which is related to the compressive (and tensile) strength of the material.

3 Repair of low strength concrete

One of the most common deficiencies in old substandard historic concrete structures is their lack of ductility under seismic loading, especially in the case of columns, due to the increased axial load. When the axial load to the compressive load ratio, $\nu=N/(A_c \cdot f_c)$, is >0.4, then premature compressive zone failure is exhibited, prior to the initiation of yielding of the flexural reinforcement. This is attributed both to the low compressive strength of historic concrete that is usually in the order of 12 MPa, due to the low quality of cement of that era, but also to the design practices that took under consideration only the vertical gravity loads, as well as to the lack (or very sparse distribution) of stirrups for confinement.

In order to change this deficiency of old substandard columns, a usual practice is to increase the members cross-section, by reinforced concrete jacketing, resulting in alterations of the architectural characteristics of the heritage structure (Fig. 9). Reinforced concrete jacketing requires the addition of new longitudinal and transverse reinforcement, as well as proper cover depth, that is usually >70 mm. In cases of old substandard columns measuring 200x200 mm in cross-section, this practice results in a jacketed member with dimensions 350x350 mm.

In order to investigate the possibility of strengthening historic concrete columns with the use of strain hardening cementitious composites, a preliminary set of experiments was performed, as a first step towards adopting this method. The repair composite is hereby proposed to be used only as cover replacement, thus preserving the original dimensions of the member to be repaired. This technique is expected to increase the compressive strength of the old concrete, through confinement provided by the fibers. Additionally, an increase of the compressive load capacity should result in lower axial load ratios, v, thus changing the mode of failure from brittle compressive to ductile yielding of the steel reinforcement. Furthermore, the strain hardening composite at the perimeter of the member will act as stirrups, increasing the shear capacity of the column.

3.1. Confinement of low strength concrete cores with strain hardening composites

A series of low strength concrete cores (HC=mean compressive strength of 10.22 MPa), with diameters of 100 and 150 mm, were confined with the use of the E6 mix design material (Fig. 10). The confinement thickness was in the order of 25 mm, equal to the depth of the uniaxial tension test coupon. The concrete cores were loaded in two different ways: compression only in the internal low strength core, and (b) compression in the full cross section of the repaired member. The first loading condition was used to derive the confinement effect of the repair material on the low strength concrete properties, while the second loading condition was used to calculate the change in the axial load that a repaired member may be able to sustain.

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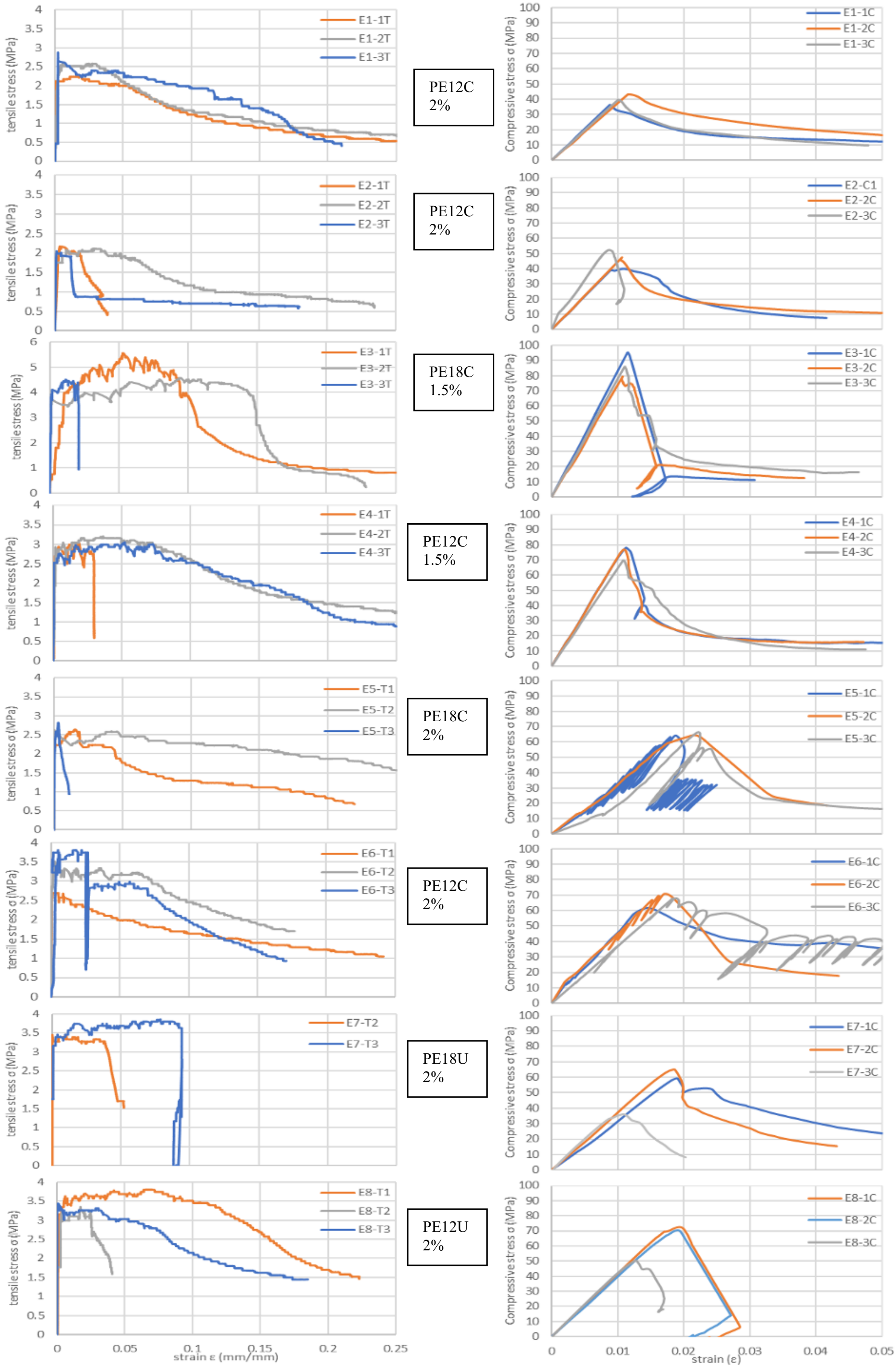

Fig. 5 Tensile stress-strain curves (28 days)

Fig. 6 Compression stress-strain curves (28 days)

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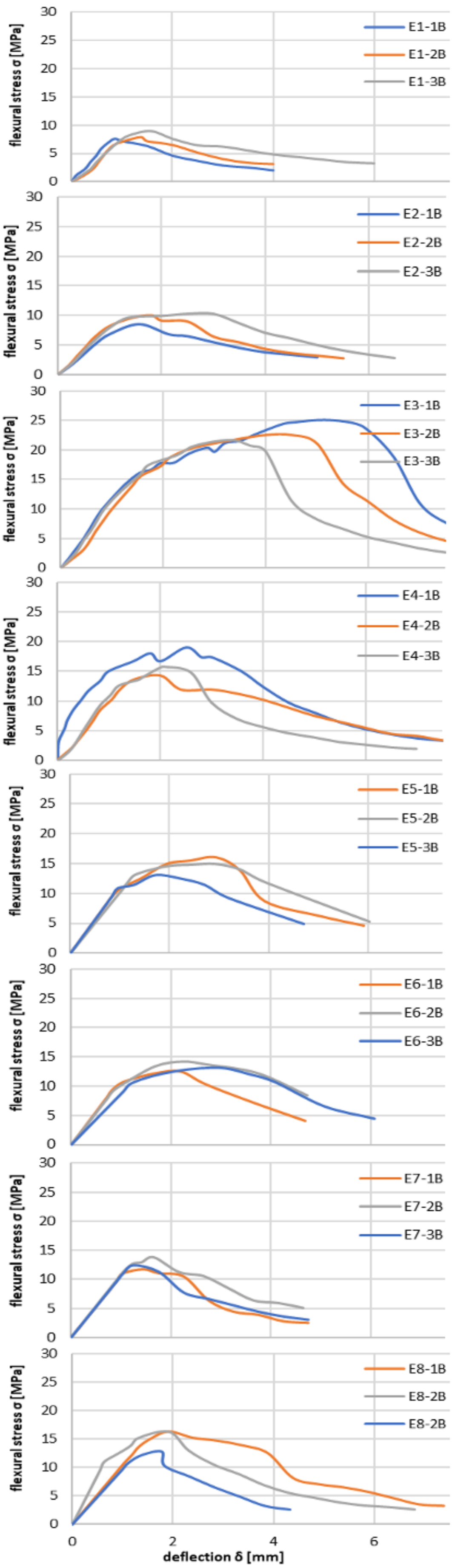

Fig. 7 3-Point Bending stress-deflection (28 days)

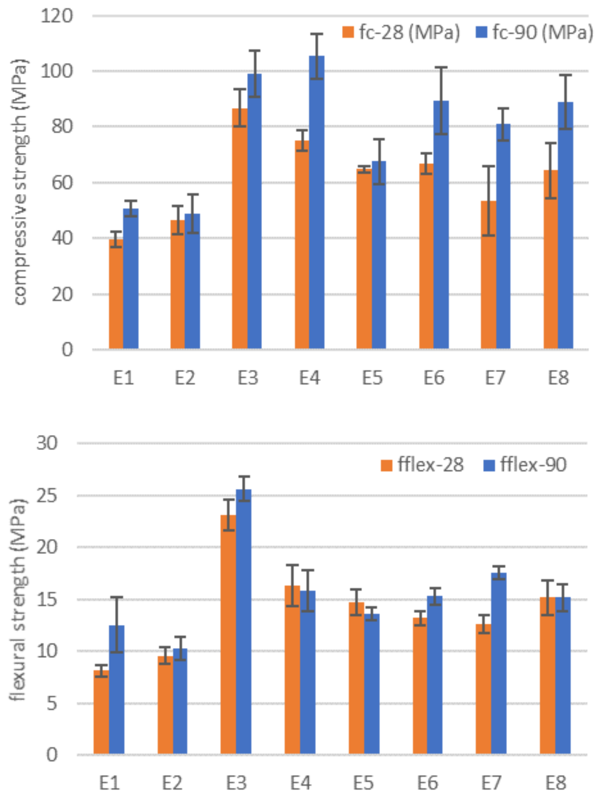

Fig. 8 Comparison of mean strength at the age of 28 and 90 days in compression (top) and flexure (bottom)

Fig. 9 Jacketing of historic concrete columns, Cyprus

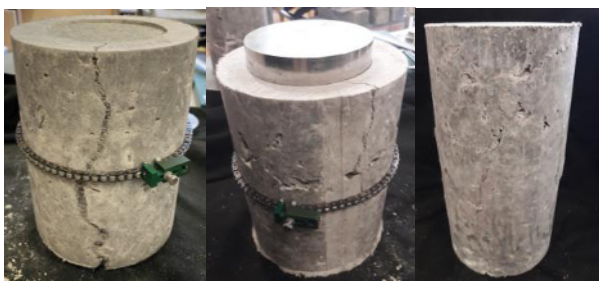

Fig. 10 HC cores confined with strain hardening composite failure patterns

3.2. Failure load results for repair

Table 4 lists the properties of the four types of specimens and the failure strength under compression that was exhibited during testing. Specimens HC-100 and HC-150 are the reference specimens with the low strength concrete, corresponding to 100 and 150 mm in diameter. Ra-100 and Rb-100 are cylinders with diameter 100 mm, that were repaired with a 25 mm strain hardening composite; these were loaded wither only internally, or throughout the full cross section of the repair respectively. L is the height of each sample, f_{cmax}, the maximum compressive strength, and N_{fail}, the failure axial load. Three specimens were tested for each different setup.

Table 4. Reference and repaired specimen results in compression

Specimen	Internal Core d (mm)	D_{final} (mm)	L (mm)	f_{cmax} (MPa)	N_{fail} (kN)
HC-100-1	0	100.02	202.27	9.69	76.12
HC-100-2	0	100.09	201.9	9.32	73.36
HC-100-3	0	99.81	202	11.64	91.03
HC-100				**10.22**	**80.17**
HC-150-1	0	146.14	308.23	8.79	147.38
HC-150-2	0	144.17	301.87	8.28	135.23
HC-150-3	0	146.04	300.76	9.33	156.22
HC-150				**8.80**	**146.28**
Ra-100-1	99.85	146	195.28	19.81	331.64
Ra-100-2	99.83	147.55	195	20.38	348.51
Ra-100-3	100.15	148.37	198.62	18.29	316.20
Ra-100				**19.49**	**332.12**
Rb-100-1	101.07	150.3	291.77	31.01	550.11
Rb-100-2	101.3	145.72	291.69	32.59	543.48
Rb-100-3	102	144.65	290.19	34.21	562.26
Rb-100				**32.60**	**551.95**

The results indicate that the strain hardening composite acts in a two-fold way. Primarily, the repair material acts as confinement reinforcement, increasing the compressive strength of the low strength concrete of the internal core. In the case of historic concrete with low strength in the order of 10 MPa, and with a mere 25 mm repair cover, the strength is doubled to 19.5 MPa. Figure 11 shows in a graphical way this increase in strength. In the case where the full repaired cross section was loaded under compression, the repair material also contributed to the load transfer, increasing even more the compressive strength of the repaired member. This indicates that a low strength historic column measuring 150 mm diameter with an axial load capacity of 146 kN, when repaired by replacement of the 25 mm cover with the strain hardening composite hereby used, will be able to sustain three times its original load, reaching 550 kN. Assuming that the original axial load ratio, v, was 0.4, resulting in brittle compressive zone failure, the new axial load ratio after the repair, will become, v_{rep}=0.1, transforming the member to ductile, with prevailing yielding of the flexural reinforcement.

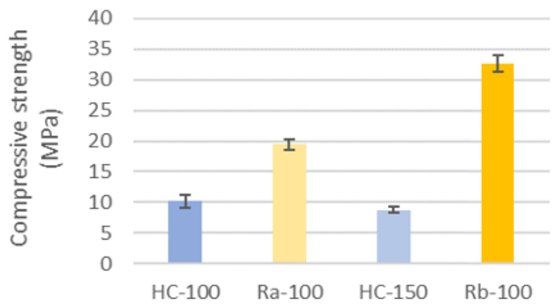

Fig. 11 Compressive strength of HC cores confined with strain hardening composite

4 Discussion and Conclusions

4.1. Indirect determination of tensile strength of strain hardening composites for design purposes

In order to design the suitable repair of a historic concrete column, based on the fiber reinforced material that will be used, the tensile strength of the strain hardening composite must be derived. This property of the material can be assessed using uniaxial tension coupons; yet this type of testing is not available in all laboratories, while it is also very difficult to conduct. Many attempts have been made until now to provide an indirect method of determination of the tensile strength of strain hardening composites; however, the procedures used are still complex and hard to follow. The multiple parameters that affect the tensile and compressive behavior of these composites, such as type of fibers, length, percentage, surface properties, matrix characteristics, specimen dimensions, render the mechanics of their behavior very complicate. Usually, the indirect methodology applied uses bending tests (4-point bending), along with inverse analysis of the load-deflection curve.

The 3-point bending tests that were performed in this study on the various mixes with PE fiber are hereby used to derive an equation that may be used directly, along with compression test results, for the determination of the tensile strength of a PE strain hardening composite. A regression analysis performed with the results, with the dependent being the engineering value of the 3-point bending flexural strength, f_{flex}, and two independent variables, i.e., the compressive strength, f_c, and the tensile strength, f_t, gave a regression curve with a statistical significance of 7.65E-06<0.05 and an R^2 value of 0.837, thus reliable results. The P-values of the f_t and f_c parameters were 0.001 and 0.032, respectively.

The curve fit given by the regression analysis is:

$$f_{flex}=3.45f_t+0.08f_c-2.1 \quad (2)$$

with the line fit plots for the two independent variables in relation to the experimental results presented in Fig. 12. Thus, the uniaxial tensile strength of a PE fiber reinforced cementitious composite may be derived directly from the engineered flexural strength of a three-point bending test and the compressive strength of the composites as:

$$f_t=0.29f_{flex}-0.024f_c+0.61 \qquad (3)$$

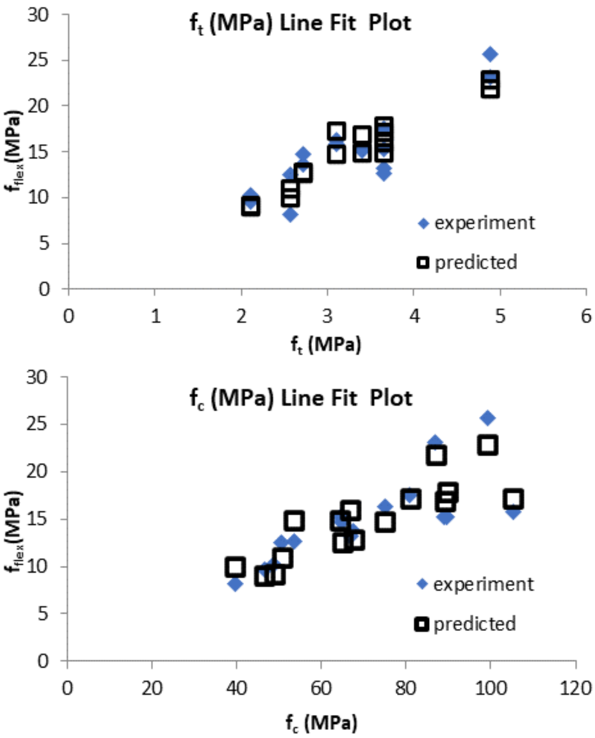

Fig. 12 Line fit plots from regression analysis

4.2. Design of the repair

Determination of the compressive strength of a repaired member can be performed, if the axial load sustained by the internal confined core is added to the axial load that may be caried by the repair cover. The confined concrete strength depends directly on the confinement provided by the strain hardening material lateral stress (σ_{lat}), based on the Richart model for confined concrete [30]. In this case, σ_{lat}, is equal to the tensile strength of the repair material, f_t.

$$N_{rep}=N_{cov}+N_{con}$$

$$N_{rep}=f_{c,R}\cdot A_{cov}+ f_{cc,HC}\cdot A_{cor} \qquad (4)$$

For the set of experimental data produced within this research, N_{cov} =440 kN, N_{con}=155 kN, and a calculated N_{rep}=595 kN. The average experimental axial strength of the repaired cores is 552 kN.

5 Conclusions and future research

In this paper, a series of experiments were carried out, in order to explore the beneficial effect of strain hardening cementitious composites for the repair of historic concrete structural members. Initially, multiple strain hardening mixes with PE fibers were selected from the literature and reproduced in the lab, in order to compare their tensile, compressive and flexural performance. One of the mixes was thereafter selected and used for the confinement and repair of low strength concrete core samples. The results indicate that with a minimum of 25 mm cover, the compressive strength of the member can become twice or three times that of the original.

These experimental findings suggest the use of strain hardening composites in the case of historic concrete members, where preservation of the architectural form is mandatory. Ongoing research is expected to provide further insides for this repair practice, through varying also the selected repair mix, the geometry and dimensions of the specimens, and the cover thickness in relation to the cross-section dimensions.

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