Service life of concrete structures rehabilitated with polymers

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ABSTRACT: The estimation of the service life of concrete rehabilitation works is more and more important. The rehabilitation techniques appeared as a need to solve problems posed by the degradation of concrete structures. Some years ago the rehabilitation techniques were not developed and it was important to find solutions for the problems. Now, there is more preoccupation with the service life of concrete rehabilitation techniques, like external strengthening with FRP, increase of concrete sections or reinforcement of cracked sections. The increase of the service life started with the quality of the concrete rehabilitation works. This includes the quality of the design, the products and the execution. Some standards are now available and establish specifications for concrete rehabilitation works. This paper presents the main questions related with this subject. The use of polymers in concrete rehabilitation imposes a different analysis related with durability.

1 INTRODUCTION

During their service life concrete structures must present good levels of security and functionality. Nevertheless, several problems on design, construction and use can put some of these requirements at risk. In situ rehabilitation of concrete elements can be made using different techniques like external strengthening with FRP (Aguiar et al. 2008) or steel plates (Tadeu and Branco 2000), increase of concrete sections (CEN 2008) or reinforcement of cracked sections (ACI 2001).

In order to increase the service life it is important the quality of concrete rehabilitation works. The design of the concrete structures rehabilitation should be made with the same care as for new concrete structures. The selection of the rehabilitation systems and products needs to take in account the expected service life. The quality of products is considered for example in ACI (1997). The quality of the products is essential to obtain long service life. Control quality and conformity evaluation systems as the CE marking established in Europe (CEN 2004) are essential. The last aspect is the quality control of the works. There are several documents that can be followed (CEN 2003 and ACI 1994 and 2002) to establish for each case an adequate quality control.

2 SERVICE LIFE

2.1 Service life prediction

Based on the measured material properties, a service life prediction can be performed using a time dependent reliability analysis (Aguiar et al. 2007). This provides a means to evaluate the probability of failure of a component. The term component describes a structure or a structural element whose limit state function is defined in terms of a single function known as the limit state function (O'Connor 1991).

The planned service period of a structure is determined by the owner of the structure based on social factors, such as future changes in the type or level of performance required of the structure, and economic factors. This is determined at the time of designing together with the design service life (JSCE 2005). The design service life may normally be assumed longer than the planned service period (Fig. 1). It is possible to assume a design service life shorter than the planned service period while maintaining the structure by repair and strengthening (JSCE 2005).

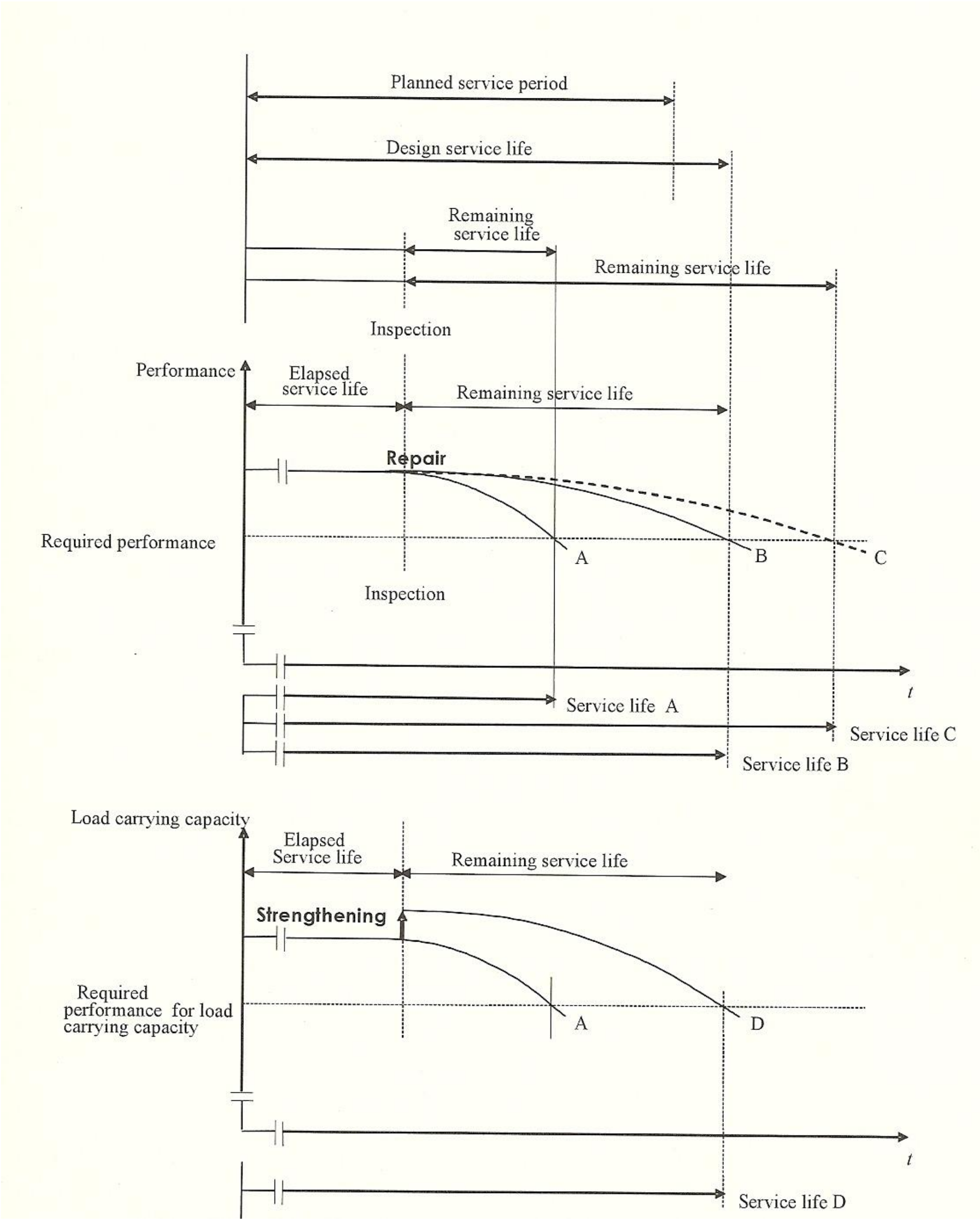

Figure 1: Definition of service life and service period.

The prediction of the service life of concrete rehabilitated with polymers needs to take in account the deterioration progress of all the materials involved: concrete, polymers and others. It is important the use of adequate methods to predict the deterioration of these materials. The deterioration progress can be easily estimated for some mechanisms of degradation of concrete. Talking about polymers and rehabilitated concrete structures the deterioration progress is not easy to determine because the mechanisms of degradation are not sufficiently studied.

The prediction of carbonation progress in concrete should be made by using the square root relationship between time and carbonation depth. For the prediction of chloride ion diffusion the use of the diffusion equation under appropriate boundary conditions is recommended. The prediction of progress of steel reinforcement corrosion can be made taking into account the results of the carbonation progress and the chloride ion diffusion. The corrosion rate and the induced cracks should be also taken in consideration in the predictions to be made (JSCE 2005).

The European standard EN 1990 (CEN 2002a) gives guidelines in order to establish the desirable service life for a concrete structure. The service life should be 10 years for temporary structures, 50 years for buildings and 100 years for bridges.

2.2 Concrete structures externally strengthened with FRP

To assure the desirable service life in the case of concrete structures strengthened with FRP there is a need to consider either the degradation of the reinforced concrete and the externally bonded FRP system. Related with the last aspect, it is important the consideration of the materials degradation and also the interfaces degradation. The factors that could affect the FRP systems are: water and moisture, salt solutions, alkaline ambiance, temperature, solar radiation, mechanical factors and fire.

The systems that use FRP to externally reinforce concrete structures have polymers in two parts, the saturating resin and the adhesive. The glass transition temperature (T_g) is the temperature above which polymers change from relatively hard and elastic to viscous, rubbery materials. Moreover, when the polymer is exposed to high humidity, this temperature (T_g) decreases (Malvar et al 2003). Because of this fact, some recommendations have suggested that FRP systems should not be used at temperatures above their T_g and further that the selected materials should have a T_g of at least 20ºC above the maximum expected service temperature (Aguiar et al 2008).

However, in most of the technical literature, temperature is not considered as a variable. This is not consistent with the importance of the temperature variation on the bond behaviour. In fact, the bonding agent deteriorates quicker than concrete, steel or CFRP reinforcement as the temperature increase, and the characteristics of the adhesive affect the strength of the bond (Tadeu and Branco 2000).

According to Gamage et al (2006) both experimental and finite element results show that the epoxy adhesive temperature should not exceed 70 ºC in order to maintain the integrity between the CFRP and concrete at high temperatures. These authors also indicate the need for a sound insulation system for CFRP strengthened concrete elements in order to promote higher fire resistance.

3 EXPERIMENTAL PROGRAM

3.1 Materials

To verify the influence of temperature on externally bonded CFRP strengthening for reinforced concrete structures, an experimental research program was defined in order to give simple and comparative results.

Two types of concrete were used: conventional (CC) and high-performance (HPC). The compositions are presented in Table 1. The concretes were produced using two Portland cements classified as CEM I 42.5 R (CC) and CEM I 52.5 R (HPC), natural river sand (maximum aggregate size of 4.76 mm and fineness modulus of 3.21), crushed granite coarse aggregate (maximum aggregate size of 9.53 mm and fineness modulus of 5.82), and a new generation co-polymer based superplasticizer.

Conventional concrete was made with a water-cement ratio of 0.60 and slump varied between 80 and 100 mm. High-performance concrete had a water-cement ratio of 0.30 and slump varied between 150 and 180 mm. At the age of 28 days, the conventional concrete had an average compressive strength of 30.0 N/mm^2 and the high-performance concrete achieved 90.0 N/mm^2.

Table 1: Compositions of the concretes.

Constituents		CC	HPC
Cement	(kg/m^3)	340 (CEM I 42.5 R)	550 (CEM I 52.5 R)
Sand	(kg/m^3)	869	469
Gravel	(kg/m^3)	865	1158
Water	(l/m^3)	206,4	165
Superplasticizer	(kg/m^3)	-	13,75

To evaluate the flexural behaviour, 650x150x100 mm^3 reinforced concrete beams were produced. The amount of steel reinforcement was the same in HPC and in CC beams and it was designed to avoid shear failures. The flexural reinforcement steel (f_{syd} = 400 N/mm^2) was 6.0 mm in diameter and the shear reinforcement steel (f_{syd} = 500 N/mm^2) was 3.0 mm in diameter (Fig. 1).

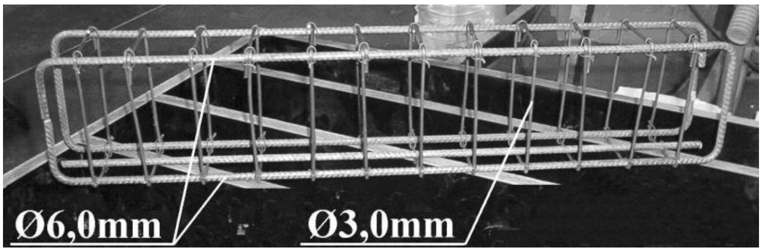

Figure 1: Steel rebars of the flexural beam specimens.

The beams were kept in the moulds during the first 24 hours. Afterwards, the specimens were removed and maintained for 20 days in water at a temperature of 20ºC. The CFRP reinforcement was applied to the beams when they were 28 days old. Before the CFRP application, the beams remained for 7 days in the laboratory at a temperature of 20ºC. The CFRP plates have a tensile strength of 2800 N/mm^2 and an elastic modulus of 165000 N/mm^2.

The adhesive used was an epoxy mortar (Table 2). It was mixed immediately before the application. Resin and hardener were mixed with a ratio of 3:1, respectively. They had different colours, so complete mixing could be evaluated after uniform colour had been achieved (ACI 1998). This adhesive contained calcareous filler.

Table 2: Epoxy mortar properties.

Specific Weight (kg/m^3)	1770
Pot-life – 35 ºC (min.)	40
Shrinkage (%)	0.04
Glass Transition Temperature, T_g (ºC)	62
Static Young Modulus (N/mm^2)	12800
Thermal Expansion Coefficient – from -10 ºC to 40 ºC (ºC^{-1})	9x10^{-5}

3.2 *Bond procedures*

After 28 days, the bonding was carried out. To prepare the surface of the hardened concrete, a diamond disc, an abrasive disc, air spurts and a soft brush were used. These resources were important in order to remove laitance, oils and dust. At the same time they gave roughness to the extremely smooth surface. The CFRP was cleaned immediately before the application of epoxy adhesive, with the volatile product indicated by the supplier.

It is important to spread the adhesive immediately after mixing, to dissipate the heat and extend its usable life. The adhesive was applied both on the concrete and the CFRP surfaces (ACI 2002). This procedure reduced the risk of forming voids when pressing the CFRP plate against the concrete surface. The producer recommends a joint of 0.5 to 2 mm thickness.

The specimens subjected to flexural tests were reinforced concrete beams strengthened with CFRP (Fig. 2). They were maintained in laboratory air (20ºC) for 7 days after the bonding process. Afterwards, they were exposed to the degradation process.

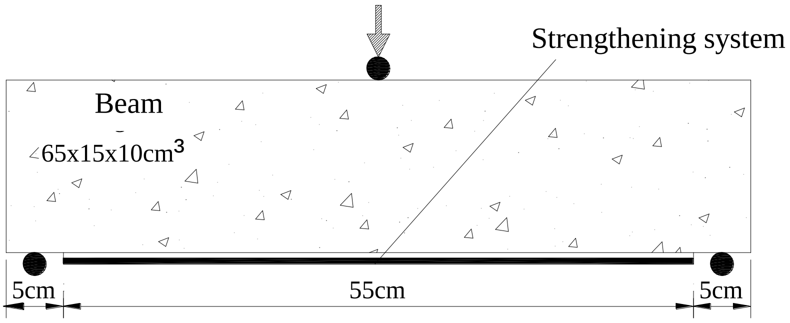

Figure 2: Reinforced concrete beam strengthened with CFRP.

3.3 *Degradation*

A degradation program was established. The glass transition temperature of the epoxy mortar was of 62ºC, so, one of the temperatures of the thermal exposure was 60ºC. It would be equally important to exceed significantly that temperature and know the behaviour at lower temperatures, such as in the laboratory environment and between that and the glass transition temperature.

The thermal exposures were based on previous work and on an European standard (CEN 2002b). The program of degradation is presented in Figure 3. The time of each cycle was 6 hours at each temperature. The number of cycles was 50.

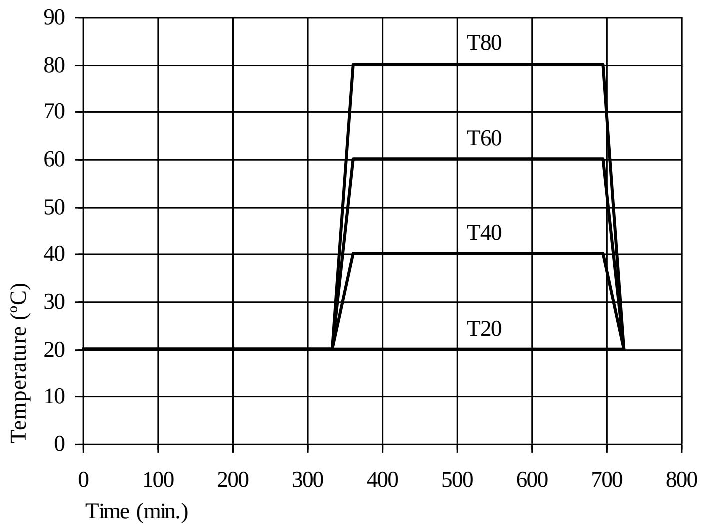

Figure 3: Variations of temperature during the thermal degradation.

To verify the real temperature due to solar exposure on the surface of the adhesive, some measurements were made. A thermocouple was installed in a CFRP strengthened beam into the epoxy adhesive layer, and the temperatures were recorded during a spring day in May. Beams were subjected to two different kinds of exposure conditions: protected and unprotected from wind action. In Figure 4 one can see the results obtained.

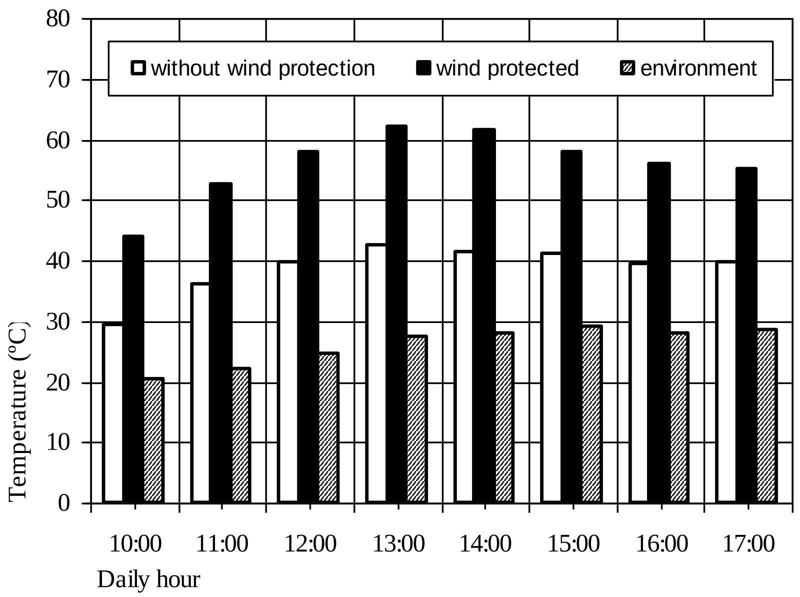

Figure 4: Variations of temperatures in the adhesive during a spring day in May.

As can be seen in Figure 4, solar exposure can imply adhesive temperatures that can attain high values, higher than 60 º C during a warm and windy spring day of May. Moreover, this shows that the chosen thermal temperatures up to 60ºC, reflect real solar exposure conditions.

3.4 Three-points bending tests

The strengthened beams were subjected to three-point bending tests, after the thermal cycle exposure. The load test was carried out using a servo-controlled system guaranteeing a mid-span deflection increase at a constant rate of 10 μm/s (Fig. 5). The test was carried out with the beams at the maximum temperature of the thermal cycle.

Figure 5: Three-point bending test.

4 RESULTS

The evaluation involved the numerical results and the visual analysis of the behaviour of the beams during and after the final test. Figures 6 to 9 represent the average curves of bending moment vs. mid-span deflection after each type of thermal exposure. In Figure 10 one can see the evolution of the maximum resisting bending moment for the different degradations.

Beams without CFRP strengthening made with conventional and high-performance concrete are referred as CC and HPC respectively. The abbreviation CC/CFRP and HPC/CFRP represents correspondingly the CFRP strengthened conventional and high-performance reinforced concrete beams.

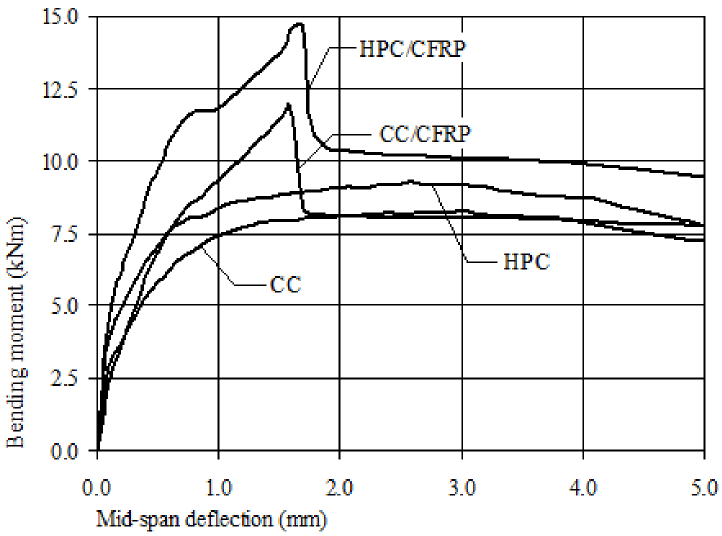

Figure 6: Variation of the bending moments with mid-span deflection, type of concrete and CFRP reinforcement (T20).

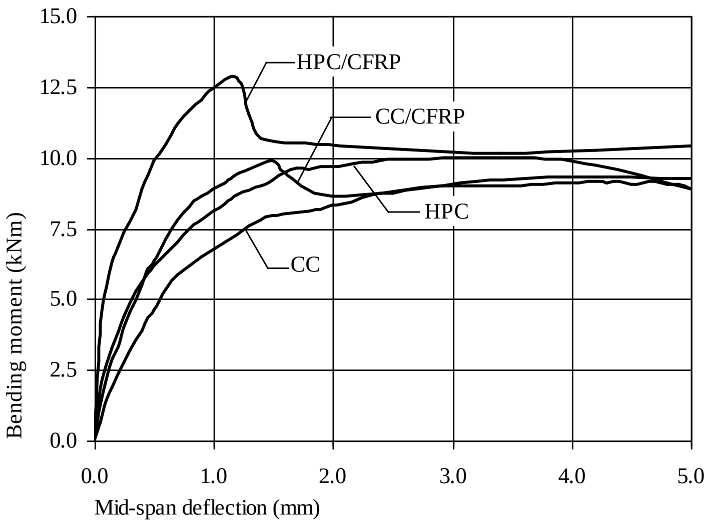

Figure 7: Variation of the bending moments with mid-span deflection, type of concrete and CFRP reinforcement (T40).

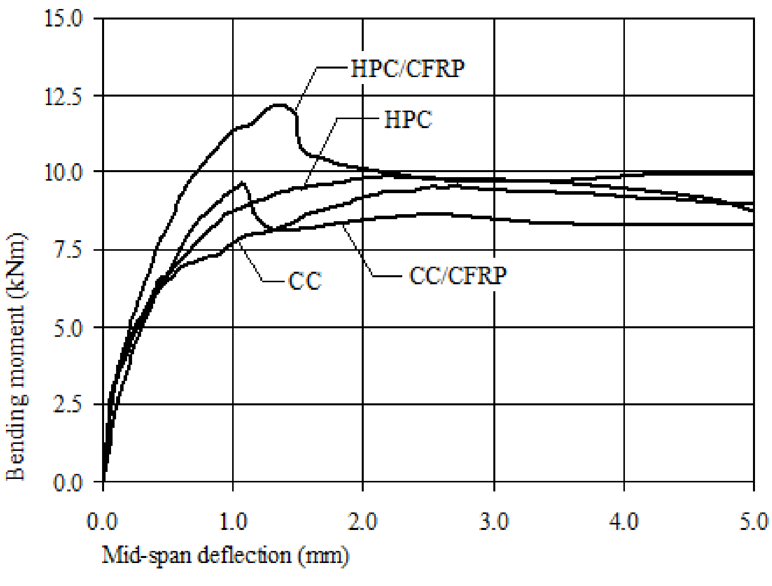

Figure 8: Variation of the bending moments with mid-span deflection, type of concrete and CFRP reinforcement (T60).

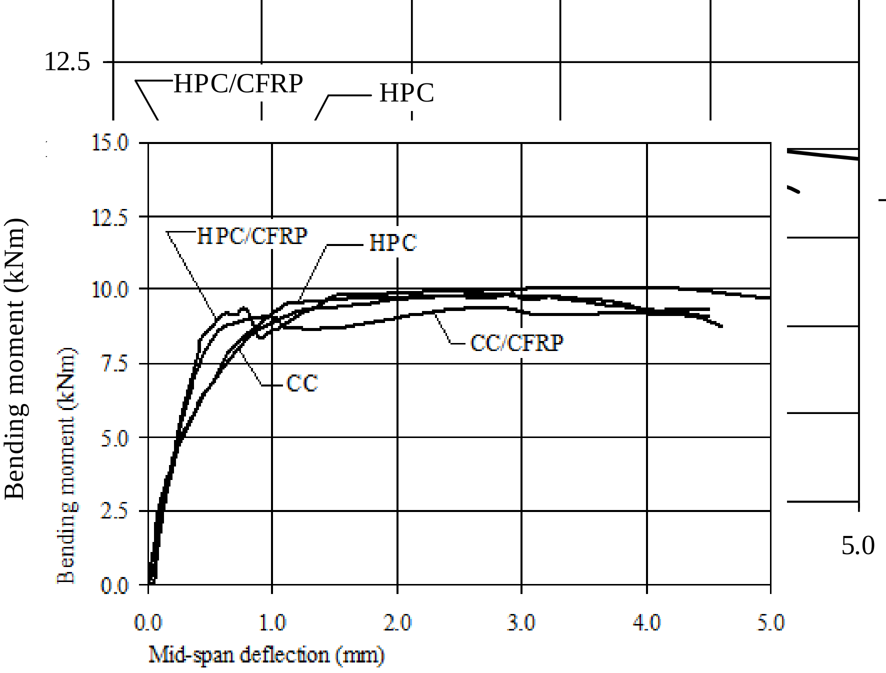

Figure 9: Variation of the bending moments with mid-span deflection, type of concrete and CFRP reinforcement (T80).

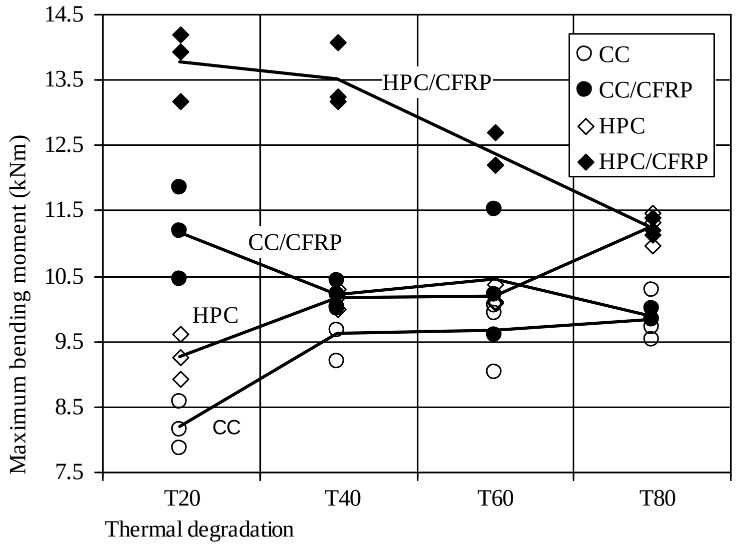

Figure 10: Evolution of maximum bending moments with temperature, type of concrete and CFRP reinforcement.

The failure types observed in the flexural tests are presented in Table 4. From Figures 11 to 13, one can see the different failure type that occurred: flexural failure (flexural), delamination of the concrete cover (delamination) and CFRP debonding (debonding).

Table 4: Flexural strength failure types.

Thermal Degradation	Failure type			
	CC	CC/CFRP	HPC	HPC/CFRP
T20	flexural	Delamination	flexural	delamination
T40	flexural	Delamination	flexural	delamination
T60	flexural	delamination and debonding	flexural	delamination and debonding
T80	flexural	Debonding	flexural	Debonding

Figure 11: Flexural failure.

Figure 12: Delamination of the concrete cover.

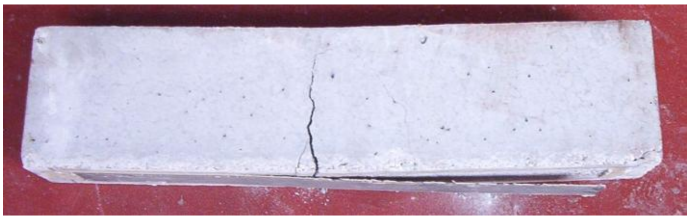

Figure 13: CFRP debonding.

5 ANALYSIS OF RESULTS

As expected, the increase in the severity of the thermal exposure decreased the CFRP reinforcement efficiency. When the glass transition temperature of the adhesive was nearly attained (exposition T60) or exceeded (expositions T80), CFRP started to debond.

With the increase in temperature, the bending moment vs. deformation curves (Fig. 6 to 9) of the strengthened beams became closer to the curves for the beams without reinforcement. The maximum bending moment increases associated with the presence of CFRP diminished significantly when the temperature increased. For T20, the CFRP strengthening gains (measured by maximum bending moments) were about 35% and 50% for CC and HPC, respectively. For T60 this was reduced to only about 10% (CC) and 20% (HPC) and for T80 exposure there was no apparent advantage in using CFRP reinforcement because the maximum bending moments of concrete beams with or without CFRP laminates were similar both for CC and for HPC.

In the series without degradation (T20) and degradation T40, the beams without reinforcement displayed flexural failure (Figure 11), while the CFRP strengthened beams exhibited delaminations caused by failure of the cover concrete (Fig. 12).

When the aggressiveness of the thermal exposition was near the adhesive T_g (T60), some debonding in the extremities of the CFRP reinforcement was noted. In these situations, particularly for the HPC/CFRP beams, debonding occurred at the concrete/adhesive interfaces

(Fig. 13). In the most severe exposuse (T80) and with HPC/CFRP beams, complete debonding of the CFRP reinforcement was observed.

6 CONCLUSIONS

The prediction of the service life of concrete structures rehabilitated with polymers needs to take in account the deterioration progress of all the materials involved: concrete, polymers and others. The mechanisms of deterioration of concrete are relatively well known. However, for polymers and for rehabilitation systems the mechanisms of deterioration need to be more studied.

CFRP laminates are currently used for reinforced concrete structural applications. In particular, repair and upgrading using CFRP bonded plates have gained acceptability all over the world in the construction field. Adhesive bonding represents the natural method of joining together different materials such as concrete and polymer composites. Consequently, assuring the durability of the externally bonded reinforcement system is crucial for the success of this technique. Among others, temperature is one of the aggressive actions that must be considered.

Based on the results obtained, it is possible to conclude that the epoxy adhesive bond properties deteriorate rapidly with exposure to high temperatures. This seems to be highly relevant, because even in solar exposure of a concrete element, it is possible to have temperatures high enough to cause some problems. Therefore, the use of reinforced systems bonded with epoxies in warm locations needs to be carried out in a very careful way. It is recommended to select epoxies with an elevated T_g at least 20ºC above the maximum environmental temperature or to considerer the application of protective insulation systems.

It is important to note that this study involved only the effect of temperature and load acting simultaneously, but there are other degradation agents to consider. At the same time then, one must also take into account the effects of relative humidity, substrate moisture, substrate surface contamination by chlorides in marine location, or chemically aggressive environments.

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