SOLVING THE BRIDGE MAINTENANCE PROBLEM IN BULGARIAN ANALYSIS OF THE OPPORTUNITIES OFFERED BY COMPOSITE MATERIALS

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Abstract

Many bridges all around the world experience very hard climatic conditions, and due to the irregular maintenance effort, show different problems of cracking, concrete cover detachment and steel reinforcement wear and corrosion.

In general, as a consequence of the polluted rain wetting, the problem is more evident in vertical structures than in the horizontal ones, and serious problems can arise when the piers and columns are very tall.

In recent times, the solutions offered by composite materials have shown their effectiveness in the field of bridge maintenance. As a consequence of lightness and ease of installation they are particularly intended to the rehabilitation of tall slender structures. Furthermore, the externally bonded reinforcement offers a valid protection to chloride diffusion and rusting of steel rebars.

In the paper the situation of the Bulgarian highways is briefly discussed in order to point out the key factors that promote the deterioration of the concrete structures.

Afterwards, the recent knowledge gained in the retrofit of building and bridges gained in Italy in ten years of external application of FRP as a solving problem tool is summarized.

Finally, the potentiality of these techniques in the solution of the maintenance problem of Bulgarian bridges is reviewed, with the aim to point out the possible cost reduction and life extension characteristic of these solutions, when translated in the Bulgarian environment.

Some application proposals close the presentation.

Introduction

All materials show deterioration or consumption with time as a consequence of physical and chemical processes [1]. The life extension of civil engineering materials like steel and concrete is of great concern in obtaining a safe and reliable infrastructural system for transportation. Furthermore, the interaction of artificial elements like roads and bridges with the environment is very complex and presents many feedback effects that could reduce the service life and generate unsustainable maintenance costs.

It is important to point out the different vulnerability sources which can affect the safety of a bridge:

1) **Environmental and geotechnical effects :** the nature and stability of the ground surface and the hydrologic phenomena are critical for the foundation stability
2) **Geometrical effects:** the geometry of some elements like barriers, joints, water drainage and other secondary items could affect the durability of the work,
3) **Mechanical and static properties:** the nature and quality of the materials as well as the maintenance ordinarily done influence the life of bridges; eventual errors in initial design or construction and overloads can generate irreversible damages;
4) **Seismic vulnerability:** in case of construction in a seismic area the bridge could experience strong motions of various levels capable to generate damages and failures.
5) **Chemical and physical attacks:** the deterioration of metallic elements due to corrosion is initiated by many types of chemical reactions or wearing of the protection materials;
6) **Operation defects:** mechanical devices like hinges and rollers or dampers can be blocked by rusting and change the static scheme of the bridge bays.

The main problem of the corrosion of the steel rebars can be graded by using a scale defining the extent of the suffered attack:

Table I : *corrosion level description for the principal reinforcement*

Level	Grade	Contamination	Diffusion	ΔR_d
1	Optimal	Less than 0.2% in volume of Cl^-	none	0 %
2	good	Less than 50 μm of bar thickness reduction	Localised in non critical zones	5 %
3	Sufficient	Less than 10% of area reduction in the bars	Diffused in non critical zones	10%
4	bad	Less than 25% of area reduction in the bars	Localised in critical zones	25%
5	worst	More than 25% of area reduction in the bars	Diffused in the whole structure	50%

The definition of the relation between the salt contamination of the concrete and the section reduction of the steel reinforcement is not easy and many alternative proposals are available.

Synthetic description of the concrete corrosion factors

The atmospheric corrosive components are:

- oxygen as an oxidant agent and promoter of the cathodic reactions;
- water moisture can dilute the electrolytic agents after condensation on the concrete surfaces
- gases as carbon oxide and dioxide and sulphuric anhydrite can penetrate into concrete pores and activate reactions.

The presence of these contaminants can dramatically reduce the service life of concrete elements, and eventually produce sudden collapses. Besides the aggressive agents of the atmosphere (CO_2, SO_2, Cl^- , etc.), other factors affect the concrete durability [1, 4]:

- the nature of the concrete constituents as cement, sand, gravel and water ion content;
- the grouting technique;

- the climatic conditions..

The many alterations that are possible in concrete are all mediated by water which is the main vector of the attack by means of dilution, capillary suction, sorption, wetting and others.

Physical deterioration of concrete

The artificial material named concrete is a composite material and up to some decades ago it was retained not degradable during normal service lives.

Actually, we know that irreversible phenomena like decohesion, cracking and cover detachment can expose the reinforcement to corrosive agents and accelerate the structural degradation.

Freeze – Thaw degradation

The necessary conditions for the freeze thaw degradation of the concrete are the temperature oscillation around 0°C along with the presence of moisture in the concrete pores and the absence of entrained air bubbles in the cement paste

The damage is produced by the volume increase of ice forming inside pores. The worst conditions are due to melting salts that produce differential ice content across thickness which can be the cause of surface ploughing and spilling.

Micro air bubbles inserted in the cement paste can help the concrete to dissipate the pressures caused by water migration during icing. Aerating agents have been introduced in recent times in order to obtain this effect during grouting.

Sulphate Attack

The sulphate attack in concrete is a consequence of the huge expansion of salts containing Calcium or Magnesium during hydration. The typical reaction involves ettringite (hydrate sulphoalluminate $Ca_3Al_2O_6$-$3CaSO_4$-$32H_2O$) and thaumasite ($CaSO_4$-$CaCO_3$-$CaSiO_3$-$15H_2O$), which are highly water sensitive. The most important cases are related to wetting and drying of sea sprays, and to contamination of the concrete gravel content. The attack is initiated when the sulphate content is over 0.4 – 0.6%.

Pure water dissolvement

Pure water typical of mountain rivers can mobilise the calcium hydroxide by hydrolysis. Furthermore, if the carbon dioxide content is over the equilibrium limit with the carbonate, it forms bicarbonate that is readily dissolved by water. If the Ph is going to be low, silicate and alluminate too can be dissolved by pure water.

After attack by pure water the concrete surface shows characteristic carbonate concretions.

Alkali aggregates reaction

If the cement is highly basic as the Portland one, the silica aggregates produce a silica gel that during expansion can exert huge pressures on the cement paste. After crack formation the gel is exposed to air and carbonization, and recovers a solid phase.

Electro chemical attack of the concrete

The steel reinforcement inside concrete in protected in optimal conditions by the concrete alkalinity which normally presents pH values in the range 12.5 to 13.8.

When the pH is higher than 11.5 the chloride content necessary to initiate the steel corrosion is unevenly high and the concrete protects the rebars. In these conditions only the diffusion of the carbon dioxide or the chloride ion can lower the pH value in the bar surfaces till the corrosion process can begin [1].

When the ferrous oxide velvet flakes are formed, the volume is rapidly increasing producing soon the spilling of the concrete cover.

Carbonization of the concrete

The carbon dioxide can penetrate in the concrete pores neutralising the alkaline content. The concrete pH reduces from 13 to 9 allowing for the ferrite – ferrous phase transition in the steel where the bars stay inside the diffusion zone.

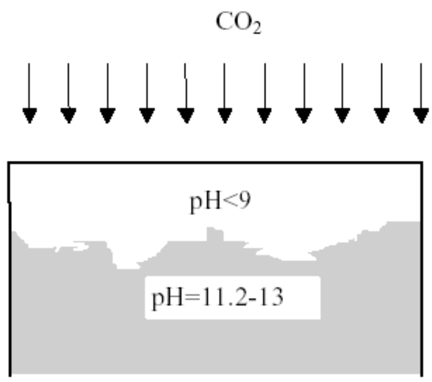

Fig. 1: *Schematic representation of the carbonatation process*

The carbonization is thus linked to the carbon dioxide transport across the concrete pores with the following approximate reaction:

H2O, CaOH $\quad Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$

When the steel surface is in contact with water and oxygen in a depassivated environment, the corrosion is initiated with the rust formation. Only in presence of water and oxygen the reaction is possible.

By example, in a deactivated zone the corrosion can be neglected if the relative humidity of the air is less than 70% (lack of water), or when the concrete element is submerged in water (lack of oxygen). Therefore, in a carbonated element the worst conditions derive from the cyclic wetting and drying

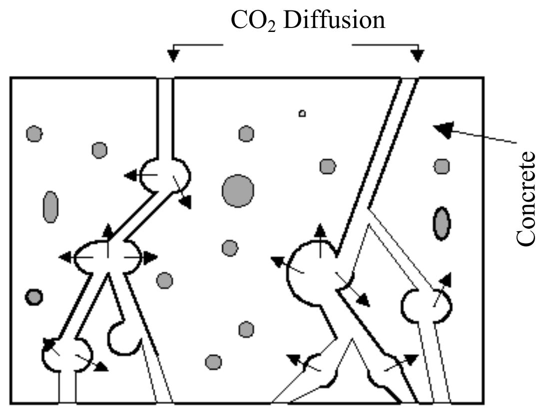

Fig. 2. *Schematic representation of the carbon dioxide diffusion into the concrete pores.*

Carbonization initiates from the outer surface and progresses to the core with decreasing speed. The law defining the penetration depth with time is a Flick diffusion like law:

$$s = K\, t^{1/n}$$

where s is the carbonated depth and t is the time parameter.

In porous concretes the law is parabolic with an exponent $n = 2$; the exponent is increasing inversely with the density of the concrete; the diffusion coefficient K is strongly dependent on the relative humidity of the air.

It is not easy to define the dependence of K from the various parameters; as a general consideration, for compact concretes the K value is around 1.0, with a penetration depth of 10 mm in 50 years, while for porous concretes it grows up to 10.0 with a penetration of 40 mm in 15 years. In figure 3 the limit trends are shown graphically.

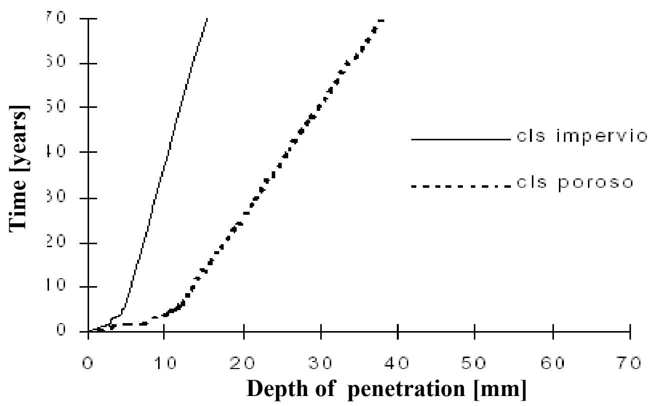

Fig.3. *Advancement of the carbonated front*

Fig. 4. *Effects of the carbonization and pitting corrosion on the reinforcement*

It is important to point out that in the conditions that give the largest penetration speed, the corrosion speed is not significant. For these reasons structures inside buildings have a so limited humidity content that the corrosive attack is not apparent. In structures wet usually by rain the factors are inverted: the speed of penetration of the carbonization front is very low but once the steel reinforcement is raised up the corrosion proceeds very fast.

The experimental detection of the carbonization depth is done by using a phenolphthalein alcoholic solution sprayed onto the concrete surface. If the concrete is basic with a pH larger that 9.0 the solution assumes a red colour typical of basic ambient.

Fig. 5: *Collapse of the Saint Stephan bridge due to pitting corrosion of the main bars*

Chloride Attack

The presence of diffused chloride aggressive ions produces the destruction of the basic protective layer around the steel bars. The saline attack is typical of sea environments where sprayed water is transported by wind or in mountain bridges, where the ice melting is obtained by using Sodium or Calcium salts.

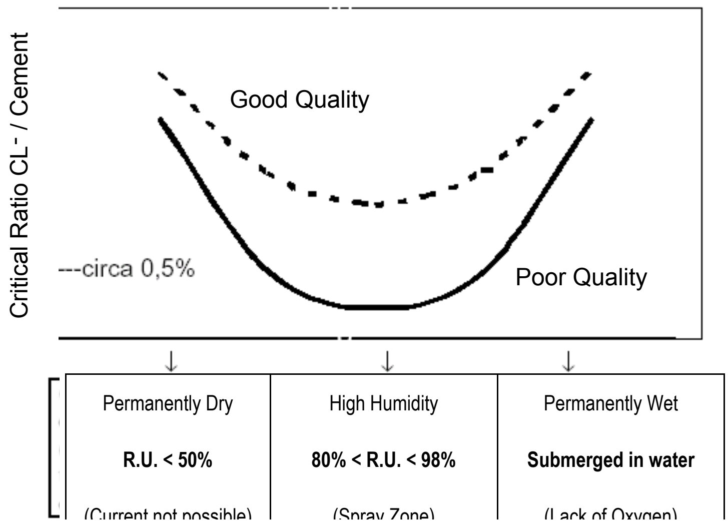

Fig. 6: *Influence of the ambient conditions on the critical chloride content*

The minimum concentration of the chloride ions that starts the corrosion is function of the ratio of the chloride and hydroxide ion content. Due to the large spectra of concrete compositions and ambient conditions is impossible to draw a defined unique value of the concentration threshold.

The TC 124 RILEM recommendation suggests a value of the ion ratio equal to 0.6 and presents the table of figure 6. Moreover, with reference to figure 7, it is possible to define the field where the salt concentration and the temperature allow the pitting corrosion.

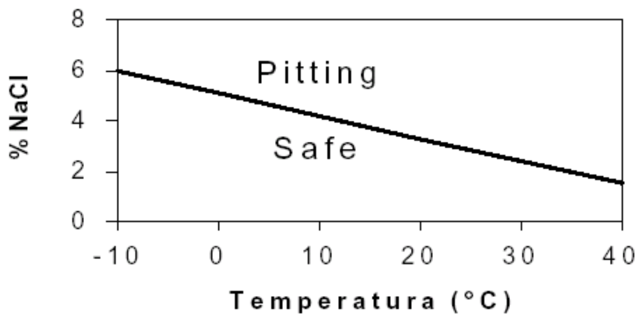

Fig.7: *Pitting corrosion domain defined by salt concentration and temperature of the concrete*

It can be suggested that the use of salt accelerant additive for concrete in cold regions is promoted by the very high salt concentrations required to start pitting corrosion when the temperature remains below 0°C for most of the service life.

In order to produce generalised corrosion is necessary a fair distribution of oxygen in contact with steel. Obviously this is the case when the cover is spilled off but, in general, the most dangerous phenomenon in the localised or pitting corrosion.

Pitting is characterized by the formation of a small anodic region inside a wide cathodic one with high speed of corrosion penetration and small weight loss of the bar. Due to the high current intensity, the corrosion bit effect is very fast and dangerous.

The pitting corrosion requires a chloride concentration always larger than the critical one; the reaction is depicted in figure 6, according to an auto stimulating process. The cathode reaction in the neighbourhood of the pit prevents the formation of other pits, thus concentrating the steel dissolution in the first formed concentration point.

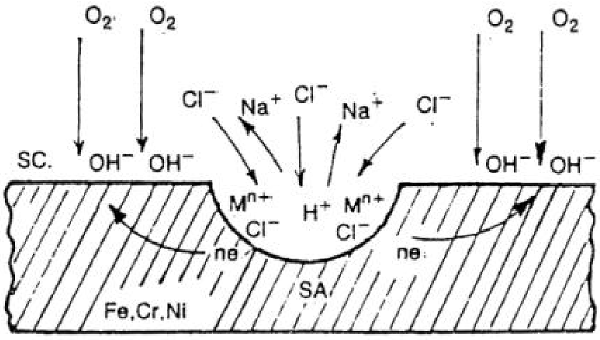

Fig. 8: *Auto stimulated nature of the localised pitting corrosion electrical circuit*

Evidence of damage suffered by Bulgarian Viaducts

The construction of viaducts in Bulgaria begun 40 year ago, with the reconstruction of the National road Sofia-Burgas in its part Pirdop-Rosino. In 70 years of 20th century was erect the “Asparuhov bridge” and in this period began the construction of Highways “Trakia” and “Hemus”. In their mounting parts in passes “Vitinia”, “Vakarel” and “Traianovi Vrata” was necessary the execution of many very long viaducts. Its system is based on simply supported beams with continuous slab. These bridges span 20, 27, 40 and 60 m, but are predominant viaducts with spans 40 m. In two construction works (“Bebresh” and “Korenishki dol”) were used 60 m prefabricated beams.

After 30-40 years in service the viaducts show the following characteristic defects:

a) In the construction the drainage was executed with vertical ducts arriving only at 20 cm approximately below the slab surface. The water flow from drainage combined with wind wetted the lateral surface of longitudinal beams. The situation in winter is very unfavorable, since the water contains saluted chloride salts. Actually these parts of the beams surface show corrosion of concrete, but retrofit is not yet executed. In the rehabilitation of the highways sections the drainage ducts were prolonged to 20 cm below to the lower beam surface. This work is certainly late, but it was extremely necessary.

b) In the first execution the viaducts included dilatation joints type "Maurer" made in Germany. After that, in Bulgaria was organized a production of joints, similar to "Maurer" ones. These Bulgarian joints had some operation defect. In fact, the Bulgarian joints designed for 75 mm dilatation used standard steel profiles. The bolts, which connected profiles, did not resist the actions of the intensive traffic. In Bulgaria were produced joints with different rubber profiles with "scissor" type mechanisms. These Bulgarian joints showed bad function in service and in some cases they were changed with joints made in Germany, France or Spain. The delay in substitution of the defected joints led to the development of corrosion in the structural components and created discomfort for traffic.

c) In the last years the increase in detachment of concrete cover and reinforcement corrosion was noticed in many supports of the viaducts. This has strong evidence in the "Bebresh" viaduct, where the corrosion affects a large concrete surface in tall pier with height 100-120 m, (fig.9a). The pier dimensions lead huge retrofit problems and special solutions based on innovative techniques could be effective; composite materials like FRP could be employed in order to reduce the quantity of metallic reinforcement exposed to rusting. Similar defects are present in the piers of the long span viaduct in "Hemus - Highway" presenting a steel orthotropic structure whose central span is 162 m, (fig 9b). In the part of the viaduct passing over an old road a severely damaged pier has been already repaired by means of sprayed concrete although this technique cannot stop the evolution of reinforcement corrosion (fig.10).

a) b)

Fig. 9: *View of the corrosion extension in the: a) "Bebresh" viaduc*t piers; b) piers of the steel structure viaduct with central span of 162 m.

Fig. 10: *Sprayed concrete repair of one pier in the steel structure viaduct near the central 162 m span*

d) Very slender bridge decks of reinforced concrete with span of 20 m developed inadmissible cracks and deflections. For one bridge in “Hemus-Highway” and two in “Trakia – Highway” was necessary the substitution of ordinary reinforced concrete structures with pre-stressed ones. A detailed discussion of this problem is presented in the paper of Ivanchev, Kisov, Reshkov in the Conference proceedings and other publications of the same authors.

Fig. 11: *Cover detachments showing the start of corrosion in two viaducts in “Trakia-Highway”*

e) Two viaducts in “Trakia-Highway” (side “Varshilata”) were constructed with 40 m beams (fig.11). The beams are composed by joining five prefabricated segments with length ca. 8 m. The segments were produced near Plovdiv and transported to construction site (at a distance ca. 80 km) with lorry. Near an abutment the segments were connected forming the beams with post-tension cables and after that placed with a mounting frame “Sicet” type. 20 years later the beams of the viaduct in show cracks and deflections and the safety margin of the structure is debated and under evaluation. A discussion about the deck substitution is open and also in this case a comparison among classical techniques and innovative ones based on fiber reinforced solutions could promote the selection of an optimal strategy.

The service conditions of railways bridges in Bulgaria

For 100 years approximately from the early beginning of the construction of the Bulgarian railway system in the second half of the 19^{th} century, the major part of the bridges was based on the use of steel solutions. In spite of the long service period their condition ranges from good to optimal maintenance level, due mainly to the regular painting of the structural elements. Some bridges however, underwent strengthening activities in order to carry on increased axle loads.

At the middle of the 20^{th} century, some new tracks were completed and the administration begun the doubling of some sections and the curvature radius increase or step reduction of some others, mainly by inserting new sections in the lines.

In this period, some of the bridges were based on arch structures made with plain concrete, generally with a split stone leaf on the lateral faces. Afterwards, the solutions turned on monolithic reinforced concrete systems, namely arches, simply supported or continuous beams. In the last 30 years the decks were erected by using mainly prefabricated structures. In some cases prestressing both by pre-tensioned or post-tensioned cables was introduced.

In the following photos (permission offered by courtesy of “VVD Consult”), the major defects of the bridges in the Sofia – Karlovo line are presented. The cases include masonry, plain concrete and reinforced concrete structures.

In figure 12 a bridge near Chelopech passing over a road and a river is shown. The main elements present cover detachment and severe rusting of steel reinforcement. The cover was ablated as a consequence of a vehicle impact.

Very similar situation is suffered by the bridge near Mircovo (fig. 13) where the main corrosion agents are the severe atmospheric conditions.

Fig. 12: *Railway bridge in Chelopech showing reinforcement corrosion due to weather exposure after a vehicle impact*

Fig.13: Railway bridge near Mircovo showing heavy rusting of the reinforcement due to cyclic wetting and freeze – thaw.

Fig. 14: Water flowing through the dilatation joint at the bridge abutment between the Pirdop and Anton station stops.

In the bridge in between the Pirdop and Anton stations (fig. 14) a significant water flow is passing through the dilatation joint at the right abutment, and this causes corrosion of the vertical structure. Due to salt migration also the bridge deck ceiling is presenting signs of starting corrosion.

In the figure 15 an arch bridge near Hadzhi Dimovo is shown. The bridge is affected by concrete contamination and weakening as a consequence of the poor water proofing. The arch bridge near Klisura (fig. 16) is in a very similar situation.

Fig. 15. *Arch bridge near Hadzhi Dimovo. The lower surface has strong evidence of contamination due to water wetting.*

Fig. 16. *Bridge near Klisura with evidence of steel rusting on the lower surfaces*

In figure 17 the later containment wall of an arch bridge near Makotzevo is shown. The river side retaining wall has large cracks and leans out due to large rear pressure

Fig. 17. *Large crack in the lateral retaining wall of the bridge near Makotzevo*

The presented defect evidences demonstrate that the Sofia-Karlovo railway include bridges aging more than 50 – 70 years which suffer deterioration due to environmental conditions and lack of maintenance. This severe situation requires a careful consideration and a quick retrofit is urged. Among the possible solutions, those based on FRP materials could be beneficial in avoiding continued rusting of the steel reinforcement and cover detachment.

An Example: The Fornello viaduct in Romagna County

Damage of reinforced concrete in structures like bridges and highways is related to factors like environmental aggression, service loads, construction technologies and, exceptionally, seismic events. Chemicals and physical agents may induce a gradual increase in the concrete porosity and permeability, causing loss of the material integrity.

In bridge decks, damage is also dependent on low fatigue level due to cyclic loads, on thermal loads and corrosion of steel bars. The object of the present study is a viaduct of the Orte-Ravenna highway, built in the first seventies. The reinforced concrete slab of the viaduct exhibited serious structural deficiencies and, therefore, was destined to demolition and subsequent reconstruction. Parts of the removed slab were carried to the Ferrara University Laboratories in order to evaluate the concrete damage level and to assess a possible strengthening technique for repair (for a detailed discussion see [3, 5, 6]). In the paper, the investigation results are reported. The investigation consists of non destructive evaluation as well as experimental tests on concrete specimens [8]. At the structural scale a thermography is carried out in order to observe homogeneity of concrete and to point out anomalous zones with cracks, cavities or incoherent aggregates [9]. Thermography is based on the emission of thermal radiation and, according to Wien law, the range of medium and far IR (2-15 μm) turns out to be suitable to characterise thermal emission of objects at ambient temperature.

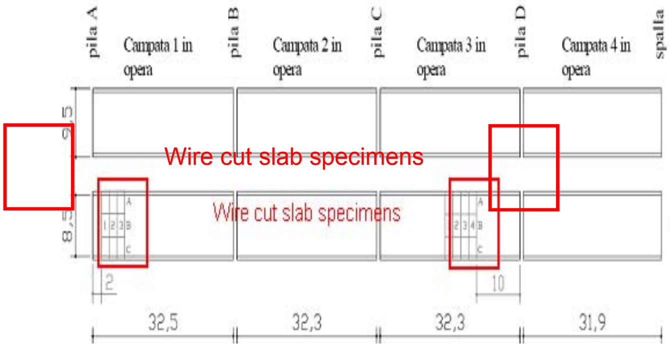

Fig. 18: *Plan of the Fornello viaduct in the Orte – Ravenna E45 highway*

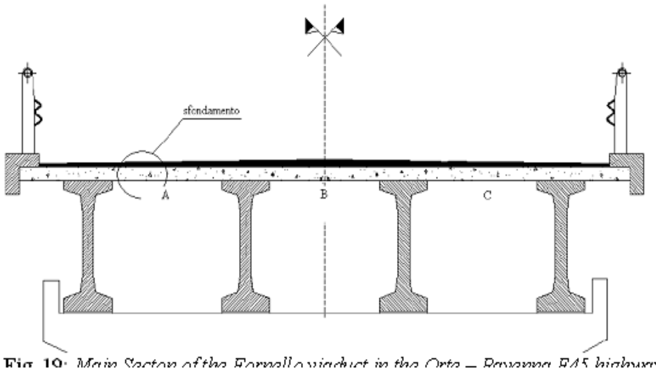

Fig. 19: *Main Secton of the Fornello viaduct in the Orte – Ravenna E45 highway*

Differences in surface temperature corresponds to different intensity of infrared rays and this produces, in the camera apparatus, contrast of the thermographic image. At the specimen scale, the investigation is carried out by mercury porosimetry, compression test, three-point bending test, ultrasound measurements, scanning electron microscope equipped with an energy dispersive X-ray (EDX) analyser.

Concerning the strengthening techniques, the application of fibre reinforced polymers (FRP) is considered. In particular, the problem of fibre debonding and concrete cover separation [2, 3], in the presence of damaged concrete, is addressed and experimental results are reported showing the effectiveness of different practical approaches to improve the composite-concrete interface.

Experimental investigation for concrete characterisation

The object of the present study is a viaduct, known as "Fornello", of the highway Orte-Ravenna (E45), Italy, managed by the National Organization for Highways (ANAS). This study was developed in co-operation with ANAS, Division of Emilia-Romagna.

The highway winds along a mountain route and the structure, built in the early seventies, presents damage problems due to freeze-thaw cycles and to abundant use of salt during the winter season, together with lack of maintenance. In the particular case of the

Fornello viaduct, the slabs showed very poor conditions of preservation and they needed urgent interventions of structural rehabilitation.

The viaduct slab was bound to demolition and part of it was transferred to the research laboratory for experimental investigation. The viaduct has 4 spans of length 32 m, and width 9.5 m in each direction (Fig. 18). The deck consists of 4 pre-stressed reinforced concrete girders and a R/C slab with average thickness of 200 mm (Fig. 19). A series of 21 samples of size 500 mm x 1800 mm were extracted from the slab. The position of samples is reported in Fig. 18.

Bars with round and square cross section constitute the steel reinforcement in the longitudinal and transversal direction. This is visible in Fig.20, where a portion of deck slab with spilling and heavy corrosion is shown. On the slab samples, infrared thermography was carried out. This is a non-destructive evaluation technique which uses differences in heat transfer through the structure to reveal the locations of hidden defects. Typical defects that can be located by means of infrared thermography are voids, delaminations, coating defects, such as blistering and subsurface corrosion spots.

Fig. 20: *View of the slab specimen cut from the bridge deck*

In particular, when the defects are near to the surface, they restrict the cooling rate due to an insulation blocking effect, and therefore produce “hot spots”. Of course, the amount of contrast observed at the surface depends of the defect dimensions and depth from the observed surface, the initial temperature rise and the material thermal properties [9].

For laboratory testing, concrete cores of 100 mm diameter were drilled from the slab (Fig.21). A significant percentage of coarse aggregates was noticed with characteristic size up to 50 mm. Mercury porosimetry, used to point out the cement paste pore distribution, revealed a total porosity ranging between 9% and 11% with prevailing micro pores of 0.01 μm (Fig.21).

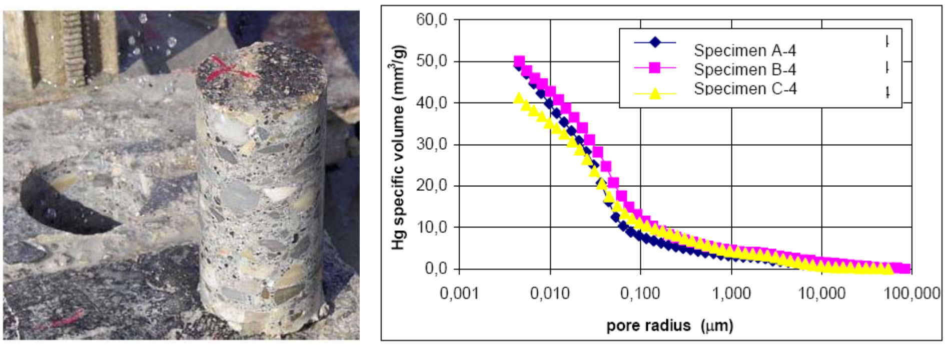

Fig. 21: *Core extracted specimens and mercury porosimetry curves*

For strength estimation, the concrete specimens were subjected to compression tests, showing compressive strength values between 23 and 33MPa. The behaviour of concrete under cyclic loading, at different levels of maximum load, was also investigated. To this purpose, the specimen was subjected to 13 load cycles: cycles 1-4 up to a load giving a maximum nominal stress of 9.5MPa; cycles 5-8 up to a maximum nominal stress of 14.4MPa; cycles 9-12 up to a maximum nominal stress of 18.7MPa and cycle 13 up to the specimen ultimate strength. Results are plotted in Fig.22 showing the slope variation of the stress-strain loading branch consequent to the change of the maximum load in load cycles.

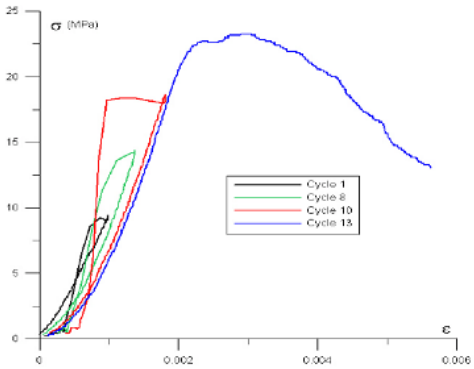

Fig. 22: *Stress – Strain paths obtained in compression test cycles*

For indirect evaluation of tensile strength, prismatic specimens (100 mm x 180 mm x 500 mm) were cut from the concrete slab and a double knife testing set up was employed. On the same specimens, ultrasonic pulse velocity tests [11] were made using piezo-electric pulse generator and receiver. Pulse velocity ranging from 3800 m/s to 4600 m/s were recorded.

As for the analysis of steel reinforcement (Fig.23), a scanning electron microscope (SEM) equipped with an energy dispersive X-ray (EDX) analyser was employed. In different points of the steel bars, a high percentage of chlorides and corrosion products were observed. The EDX spectrum recorded on the bar cross section is reported in Fig.23, indicating the presence of chlorides and agents of chemical attack (Na, Ca, S).

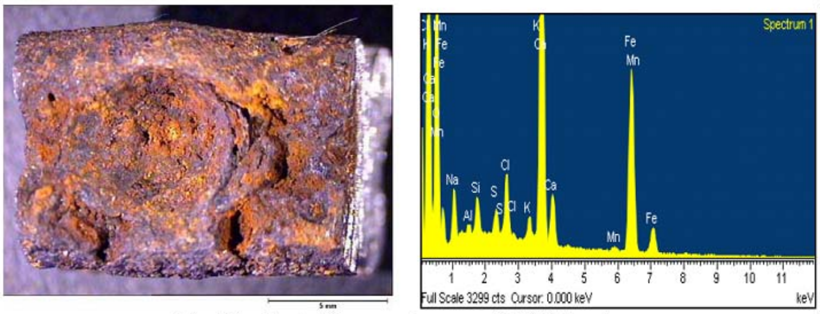

Fig. 23: *Rusted bar specimen and EDX Spectrum*

Bonding of FRP reinforcement

The performed experimental tests indicated that the concrete damage is wide but superficial, limited to the concrete cover depth of the slab upper side. Except than in this area, the actual mechanical properties of the concrete are rather constant along the slab height and consistent with the original concrete quality.

The good preservation of the slab lower side concrete suggests the possibility to strengthen the slab by bonding external FRP reinforcement on this side while replacing the upper side damaged concrete and rebars. An interesting issue concerns the possibility of bonding the FRP reinforcement on restored concrete [2, 3, 4].

In this study, this particular feature of the problem was investigated. A number of 78 prismatic specimens (100 mm x 180 mm x 500 mm) were cut from the concrete slab in order to try different repair techniques based on the restoration of the wasted superficial concrete and bonding of the FRP reinforcement on both the original (lower side of the slab) and the restored (upper side of the slab) concrete surfaces.

As Fig.24 shows, the typical specimen extracted from the slab exhibited deep cracks at the rebar level; the damaged concrete was then completely removed by means of a needle hammer and the rusted steel rebars were extracted. The concrete restoration was carried out making use of two different mortar qualities. A single component mortar with limited shrinkage and hight strength named Sika Monotop-624 A was applied to 12 of the cured specimens. As certified by the technical notes of the producer, the 28-day average strength of this mortar is around 60 MPa. A standard mortar with 3 parts of mixed 325 K cement, 2 parts of sand and 1 part of acrilic emulsion in water dispersion was applied to 12 of the cured specimens. The average 28-day concrete cilindrical strength was estimated around 32 MPa.

Finally, the FRP reinforcement was applied on the cured concrete surfaces. For this application, 50 mm x 400 mm strips of SikaWrap-230C were glued by means of Sikadur-330 epoxy. As certified by the technical notes of the producer, the average tensile strength and elastic modulus of this carbon tissue are respectively surfaces, where the original concrete support was arranged with a simple surface brushing before the FRP strip was glued.

Fig. 24: *Preparation of the slab specimens*

Fig. 25: *Restoration of the specimen surface and application of the FRP*

Fig. 26: *Accelerated ageing of the specimen by UV rays and water moisture*

The application was executed by expert workmanship, following the recommended application procedure (Fig.25). One half of the arranged specimens were subjected to artificial ageing treatments before the bond tests were executed. The ageing treatments consisted on 10 days of sun-rain cycles (2 and ½ hours of ultraviolet radiation and ½ hour of rain) and of freeze-thaw cycles (2 hours at -20° C and 2 hours at +20° C) (Fig.26).

The performed bond tests pointed out very interesting results. Superficial debonding (SD) due to the detachment of the FRP strip from the concrete support (Fig28) and deep debonding (DD) due to shear failure within the original concrete (Fig29) were observed during the bond tests of the restored concrete joints executed at the upper side of the slab. This last bond failure mode was much less frequent than the previous one. On the other hand, during the bond tests of the joints executed at the lower side of the slab, superficial debonding of the FRP strip from the specimen with coarse aggregates fracture was typically observed.

Fig. 27: *Testing of the bonding force in the universal machine by direct shear*

Fig. 28: *Debonding by thin topping material shear failure*

In order to evaluate the efficiency of the executed concrete restoration and to assess the validity of this strengthening technique, the experimental values of the bond forces are compared with the theoretical values estimated by means of the Italian CNR-DT 200/2004 [2]. In accordance with the above mentioned recommendations, the maximum bond force that can be transferred by the FRP-concrete interface is given by the fracture energy of the bonded interface. However is to consider that in this case we have two distinct interfaces.

Fig. 29: *Debonding by deep original concrete shear crushing failure*

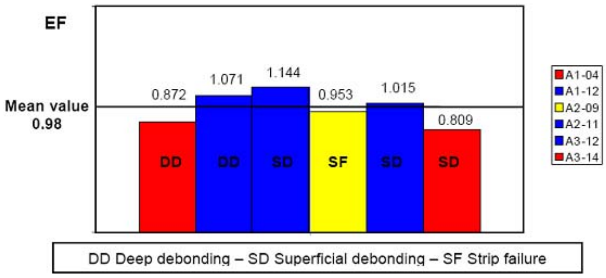

Fig. 30.a: *Efficiency factor of the bond in the SIKA Monotop section reconstruction*

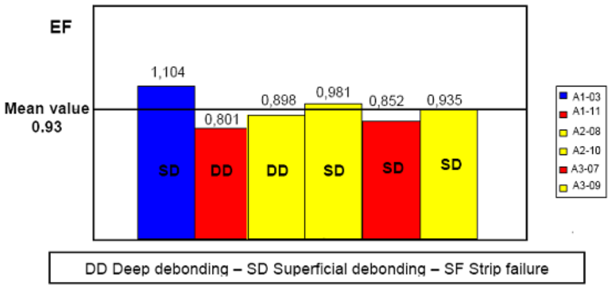

Fig. 30.b: *Efficiency factor of the bond in the normal mortar section reconstruction*

In Figs. 30.a and 30.b, the obtained ratios of the experimental versus analytical resistances are reported for the bond tests performed on the upper side of the slab, respectively when the Sika Monotop 624 A mortar and the standard mortar are employed for restoration. For both cases, an average efficiency factor very close to 1 was found (0.98 and 0.93 respectively), despite 4 of the 12 bond tests underwent failure with a deep debonding failure mode (Fig. 29).

In this case, a shear failure occurred within the original concrete support, as attested by the measured value of the bond forces. The retrofit joints exhibit bond forces at failure that are in good agreement with the theoretical previsions i.e. the loss of joint efficiency due to the concrete restoration is not appreciable.

In fact, the standard deviation of the obtained values is around 12%, which is included within the admitted uncertainty due to the rough execution procedure [4, 5]. It is to recall that the carried out application tried to reproduce the actual working site conditions.

The bond forces of the joints where the special mortar was applied are bigger of around 40% than where the standard mortar was applied. This result seems to address this special mortar as much more effective than the standard one; however, if the very high cost of the special product is considered (around 6:1 in weight), this simple effectiveness comparison is meaningful without an accurate analysis of the global cost to benefit ratio.

Bond tests were also executed on specimens subjected to ageing treatments. The obtained results did not put in evidence any appreciable damage effect.

CONCLUSIONS

This paper summarises the main concrete degradation phenomena which can occur due to physical and chemical attack; the role of carbon dioxide and chloride salts in starting the steel reinforcement corrosion is presented and discussed.

Based on the previous experience of the authors, the main degradation problems encountered in the Bulgarian bridges and viaducts are reviewed and discussed. The role of the combination of rain wetting and temperature are illustrated with reference of some practical cases of great importance in the Bulgarian infrastructure system.

As an example of solved cases, the paper reports the results of an extensive experimental investigation carried out on samples of damaged concrete slab demolished from a viaduct of the E45 Orte-Ravenna highway in Italy. The study allowed focusing on the following main results:

1) The damage state of the considered viaduct slab is wide but superficial, limited to the concrete cover depth of the slab upper side; however, the slab thickness reduction due to concrete damage and the loss of the concrete-steel rebar bond are such to produce serious structural deficiencies.

2) Except than in the upper part of the slab, the actual mechanical properties of the concrete are rather constant along the slab height and consistent with the original concrete quality.

3) The concrete cover restoration turns out to be adequate in order to strengthen the slab by means of FRP external reinforcements and guarantees an acceptable efficiency of this strengthening technique.

4) The FRP reinforcement applied to the lower part of the slab reveals to be particularly effective, producing bond forces +40% than the theoretical expectations.

Further experimental investigations are still in progress. A number of restored slab pieces have been strengthened with FRP on both the upper and the lower side and will be sooner subjected to four point bending tests up to failure.

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