Proceedings
PRO 29

Life Prediction and Aging Management of Concrete Structures

Proceedings of the
2nd International RILEM Workshop

Edited by D.J. Naus

RILEM Publications S.A.R.L.

A STATISTICAL APPROACH FOR MANAGING VISUAL INSPECTION AND IN SITU TESTING IN DESIGNING REHABILITATION PROJECT OF A 5 KM LONG VIADUCT

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Abstract
A 120 spans five km long viaduct on the A19 highway Catania –Palermo in Sicily (Italy) was considered for a rehabilitation project by the Italian National Road Administration. In consideration of the great extension of the viaduct and in order to optimise the rehabilitation cost a detailed visual survey and in situ inspection campaign was carried out. Potential mapping, carbonation and chloride profile determinations on concrete cores and concrete compression strength determination on concrete cores were extensively used as diagnostic techniques. The great amount of obtained data allowed a statistical evaluation of structural members degradation. Potential mapping was extensively used to evaluate corrosion of prestressing steel in beams. Four different potential values distributions were obtained for the following beam areas: heads of the left and right edge beams, heads of the internal beams and central zone of the beams.
In spite of the good concrete quality, a mean carbonation depth of 16 mm being evaluated on the beams after 30 years, extended degradation of the beam heads was observed as a consequence of a poor detailing in beam design and inadequate drainage systems.

1.Introduction

The A19 highway was built during the late 60's and the early 70's to connect Catania to Palermo and it is long 193 km. According to the building philosophy of the time several very long viaducts were planned to be inserted along the route of the highway. Cannatello viaduct is a 120 spans five km long viaduct built along the course of the Imera Meridionale river. Each span was made using prestressed concrete precast beams in a five T beams girder configuration (Figure 1) and based on circular pier (2.80 m in diameter) overlapped by a reinforced concrete pier cap (hammerhead type). The upper flanges of the beams, 33.94 m in length and 1.80 m height, constituted the span deck. Beams were connected each other by a longitudinal cast in place joint and by means of transversal Dywidag bars.
Longitudinal prestressing reinforcement on beams consisted of 54 steel strands (0.6 inch diameter) mainly located at the lower flange.
During the years the south to north carriageway was subjected to an unintentional salt spreading as a result of an intense traffic of trucks bearing mineral salt from a surrounding mine to Palermo harbour. Due to the extended degradation of the structures the viaduct was considered for a

rehabilitation project by the Italian National Road Administration. In consideration of the great extension and in order to optimise the rehabilitation cost a detailed visual survey and in situ inspection campaign was carried out.

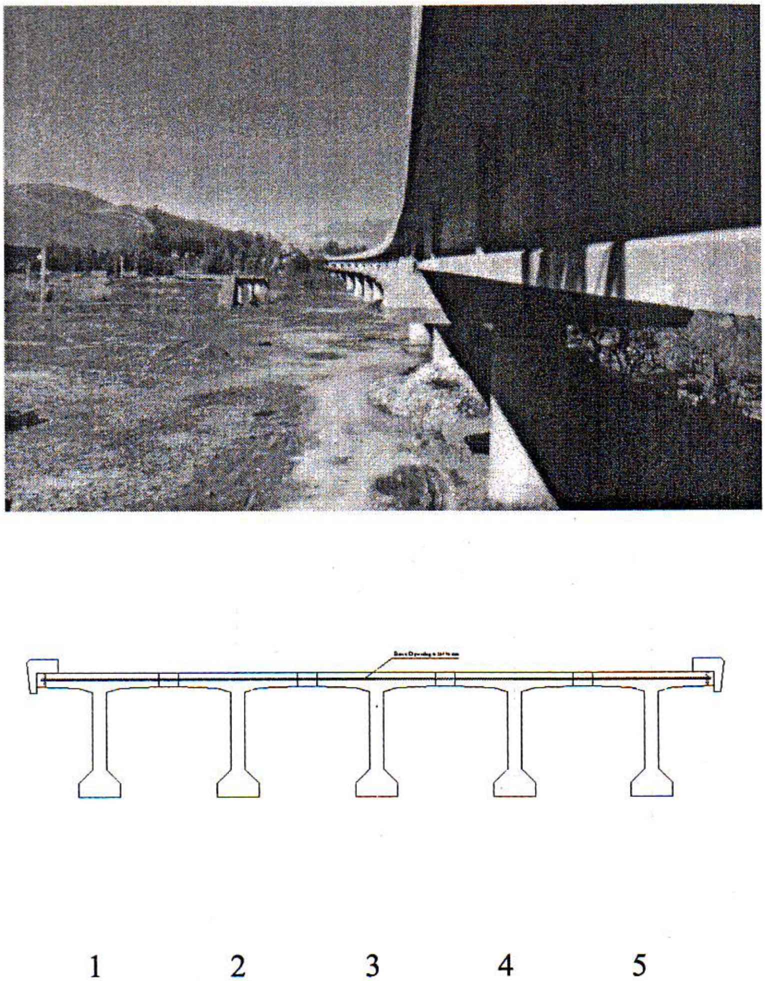

Figure 1: Side view and transversal section scheme of the Cannatello viaduct

2. Visual inspection

Three main structural members were identified for the in situ visual inspection: piers, pier caps and beams. Damage on structural members were identified by means of a four level Damage Index (DI) calculated taking into account type (cracks, rust spot, rebar corrosion, etc..) and extension of defects, their influence on member safety, and urgency of intervention. A DI of the span was then calculated as a weighted sum of structural members DIs [1]. From a statistical analyses of the visual inspection data it resulted that about 19% of the spans were classified with a DI higher than 3 (Figure 2).

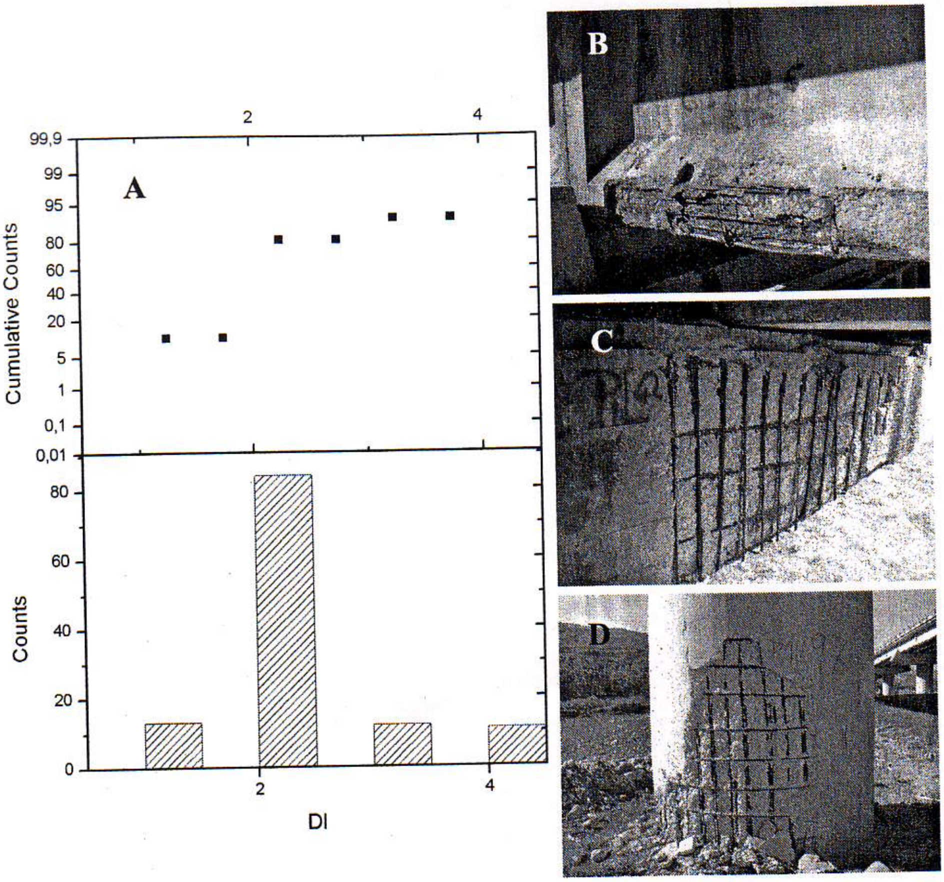

Figure 2: A) Frequency and cumulative frequency plot for DI calculated for the 120 spans. B) Example of DI 4 for a beam. C) Example of DI 4 for a pier cap. D) Example of DI 4 for a pier.

On beams the damages were concentrated at lower flanges of the beam ends as a consequence of poor detailing and inadequate drainage system (Figure 2B). A similar problem was found on some pier caps were expansion joints resulted damaged or an inadequate deck waterproofing was present (Figure 2C). As far as pier degradation is concerning it was found that degradation originated from a different cause. The great part of the damages were in fact concentrated at the pier bases (Figure 2D) as a consequence of polluted water suction from ground water table.
On the basis of the visual inspection it was planned an in situ testing campaign optimised to the evaluation of concrete degradation and to the evaluation of the beam strand corrosion status. Potential mapping, carbonation and chloride profile determinations on concrete cores and concrete compression strength determination on concrete cores were extensively used as diagnostic techniques. Since the great extension of the viaduct in order to limit inspection costs it was decided to limit the testing to specific area section of the structure considered to be more critical or more representative of the degradation status (DI 2 to 4).

3. Concrete strength

Concrete strength was evaluated on concrete cores (100 mm nominal diameter) following BS 1881[2] and calculated as estimated in-situ cube strength (f_{cu}). From a statistical analyses it resulted that concrete strength values for piers and pier caps belong to two different distributions as it is shown in Figure 3. Mean calculated value for pier was 32.40 MPa, whilst for pier caps it was 39.45 MPa. If we consider as a reference a value of 30 MPa it is possible to note that for piers about 52% of the determinations resulted lower than it, while for pier caps such percentage it reduces to about 12%.

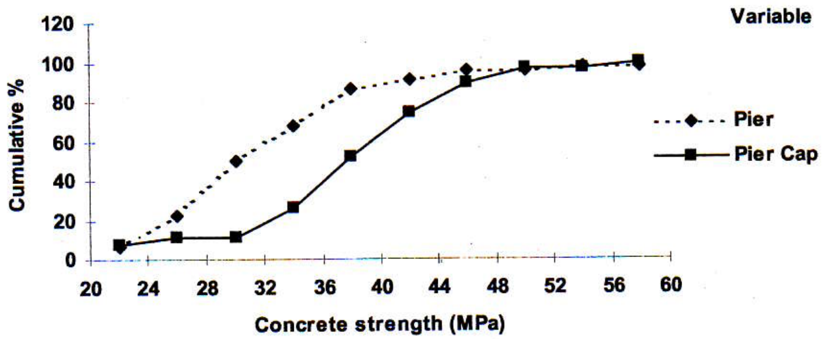

Figure 3: Cumulative percentage frequency for concrete strength values of piers and pier caps evaluated on cores

A similar result was obtained for elastic modulus (E) values, where two different distribution were observed. E mean value for pier was 16,962 MPa whilst for pier caps E mean value was 23,743 MPa, that is about 84% and 19% of measured values were below 20,000 MPa for piers and for pier caps respectively (Figure 4).

It should be evidenced however that above mentioned distribution were influenced by the presence of strongly deteriorated structural members. It was observed in fact the minimum values of concrete strength were related to the higher levels of concrete contamination by chloride and/or sulphate or to the higher levels of carbonation.

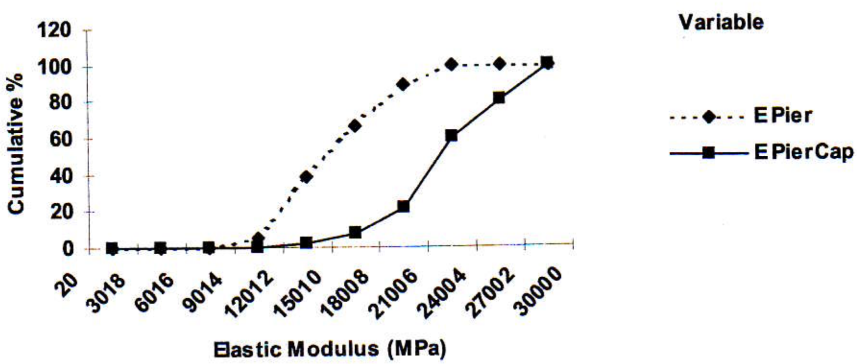

Figure 4: Cumulative percentage frequency for elastic modulus of concrete: piers and pier cap

4. Chloride and sulphate content and carbonation

4.1 Piers

Total chloride and sulphate concentration profile was evaluated in concrete cored at a height of 0.5 m from ground. On piers their content varied along the route of the viaduct (Figure 5). More precisely high contaminant content was observed for those piers that founded directly on the river bed. Chloride content values were frequently higher than 0.5 % (weight by concrete) at a depth of 1.5 cm. In some case such values were maintained up to a depth of 5 cm. Also sulphate content was particular high in critical piers with values ranging between 1 and 1.8 % (weight by concrete). In those case frequently an inversion between the internal (3-5 cm depth) and the external (0-1.5 cm depth) values occurred as a consequence of pollution originating from ground water capillary suction instead than from water percolation on external surface. That was confirmed by the lower contaminant content observed on concrete cored at a height of 2 m.
Carbonation depth, evaluated by means of phenolphthalein test, ranged from 40 and 50 mm with minor values obtained in piers with the higher contaminant content congruently with the consideration that an higher contaminant content corresponds to a higher water content and a lower carbon dioxide diffusivity.

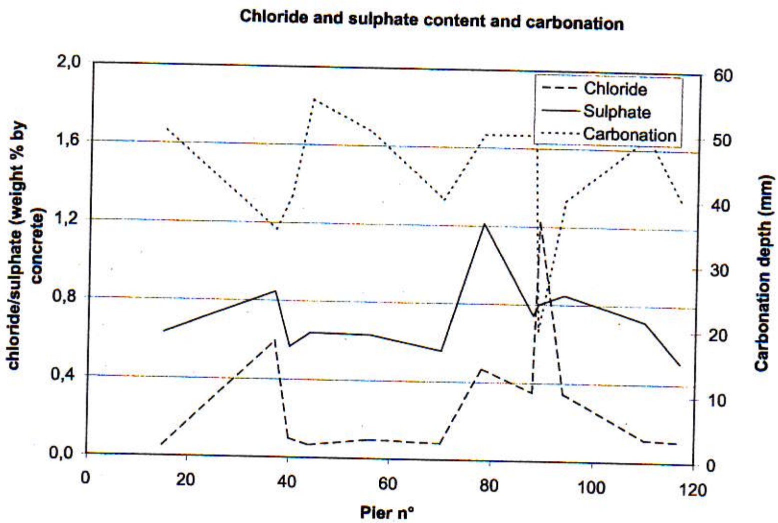

Figure 5: Chloride and sulphate content and carbonation depth on piers (mean values on 3 cm concrete depth)

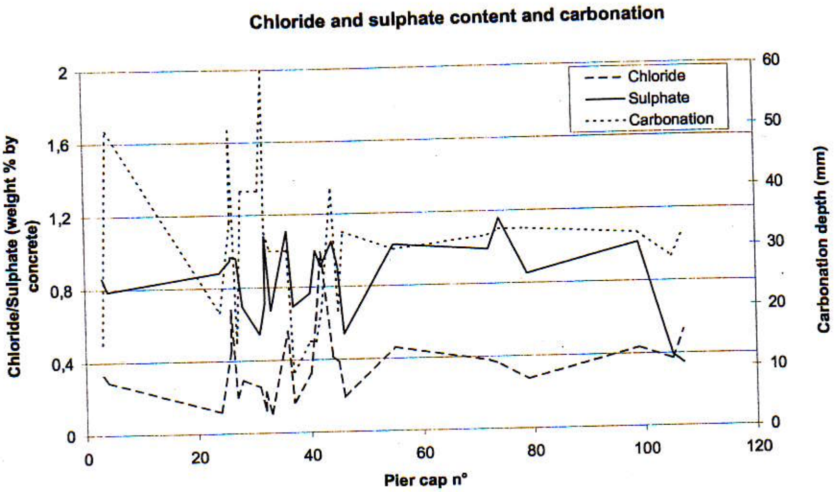

Figure 6: Chloride and sulphate content and carbonation depth on pier caps (mean values on 3 cm concrete depth)

4.2 Pier caps

Contaminant level resulted quite high in pier caps (Figure 6), and values remained high also at a depth of 5 cm. On more damaged piers it was observed an inversion of contaminant content on depth (higher level at a depth of 3-5 cm than on the surface). A leaching effect by water percolating from deck was supposed. Mean carbonation depth was about 30 mm, even if in some cases peaks as high as 60 mm were recorded.

4.3 Beams

Chloride content was evaluated in concrete cored from lower flange of the beams at a distance of 0.5 m from bearings. In this case three different homogeneous populations were identified, that is concrete samples collected from edge beams (number 1 and number 5 external beams, see Figure 1), concrete samples collected from internal beams (number 2, 3 and 4 beams) and concrete samples collected from the middle of the beams. Since chloride contamination originated from defective expansion joints of the deck and drainage systems (generally located near the bearing), concrete contamination was not in this last case influenced by the beam location in the girder. Three different chloride content distribution were in fact observed (Figure 7): external beams having the higher chloride content compared to internal beams and to beam middles (mean chloride content equal to 0.24, 0.15 and 0.03 weight % by concrete were observed respectively).

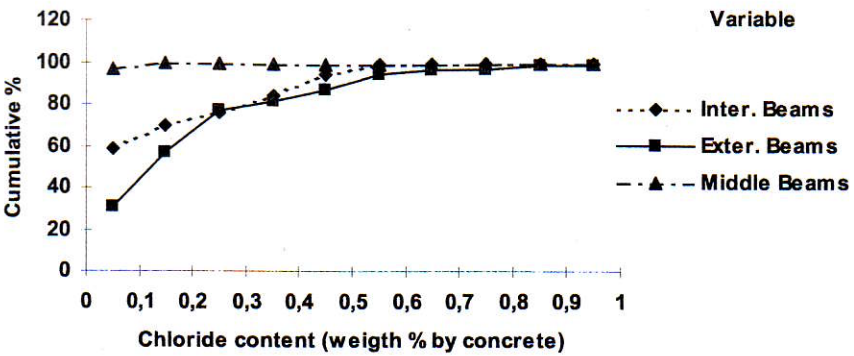

Figure 7: Cumulative percentage frequency of chloride content in the beams (means on 3 cm concrete depth)

Considering the good concrete quality (a mean carbonation depth of 16 mm was recorded on concrete cores after 30 years) the extremely high values of chloride content measured on some beams evidenced the critical state of some bridge spans.

These critical areas of the viaduct can be easily identified by plotting chloride content versus span number as reported in Figure 8. Here contamination level of concrete cored from deck was also reported. The high values reported for internal beams on span 17 and on span 27 (arrows in figure 8) were connected to water percolation from a local damage of the deck in first case and, in he second case, to water percolation from north to south carriageway that, at this point cross over the south to north carriageway. High level of pollution observed in external beams of the spans numbers 98 through 106 was supposed to be connected to the curvature of the carriageway which could favourite loss of salt from trucks.

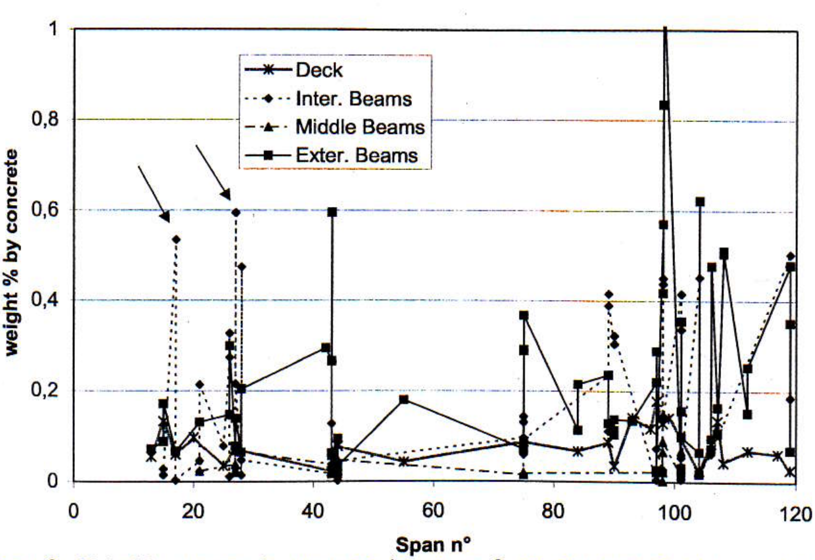

Figure 8: Chloride content in concrete (mean on 3 cm concrete depth) cored from beams and deck versus span number.

5. Potential mapping

Potential mapping was performed only on limited areas of the beams (1.6 x 3 m with 90 reading points) located at the lower beam flange by mean of electrical connection to prestressing steel. According to the same consideration done for concrete core location, inspection areas were placed near the bearings and (as a reference) in the middle of the beams. Four homogeneous areas were therefore identified: north and south exposition external beams, that is beam number 1 and 5 respectively (it was supposed that sun exposition could affect potential values), internal beams and beam middles. Four different distributions of the potential values were recognized as evidenced in Figure 9 where cumulative percentage frequency for each population is reported. Beam middles areas were characterized by very positive values (dry concrete and passive steel) being 99% of the measurements higher than 0 V (vs Cu/CuSO4 reference electrode). Positive and very negative (lower than –0.350 V) potential values were instead recorded on areas located near bearings. For external beams, south exposition and north exposition, more than 30% of the values were below –0.200 V (34 % and 31.5 % respectively) and almost 16% of the values less than –0.350 V (for both distributions). Such percentage reduces to 14% and 5.5 % respectively for internal beams.

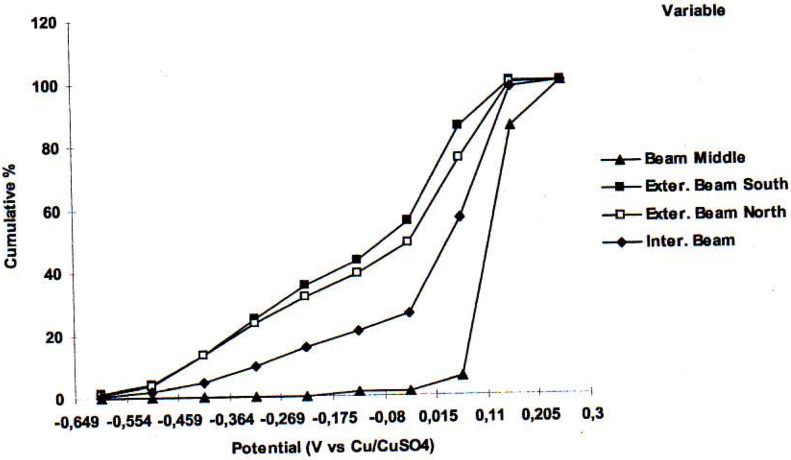

Figure 9: Cumulative percentage frequency of potential mapping values

Further information about corrosion status of prestressing steel could be obtained by plotting the mean potential values (mean on 90 readings) of the inspected area along the route of the viaduct (Figure 10). After span n° 27 (cross over point) there is an inversion between "north exposition" external beam and "south exposition" external beam , the last one being initially characterized by lower values. An "U" trend is evident for all readings with a gradual decrease of mean values in the middle of the viaduct route, where it is possible to evidence two "hot spots" (span number 43 and number 71) characterized by mean values less than –0.350 V.

6. Conclusion and consideration about the rehabilitation project

In consideration of the great extension of the viaduct is extremely important to use correctly inspection data in order to define the extension of the rehabilitation intervention.

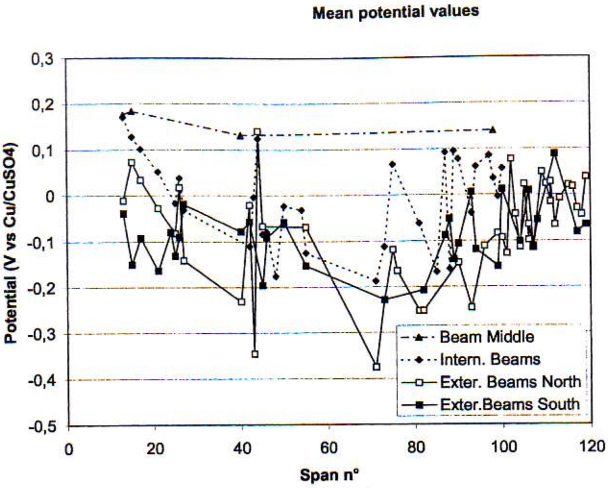

Figure 10: Mean potential values for the different four areas along the viaduct route

Results showed a general status of the structure particular severe. Degradation level was particularly high though limited to specific areas.
As far as the support structures are concerning we have to differentiate consideration about piers and pier caps. For the former the main problem was connected to capillary suction of ground water particularly rich in chloride and sulphate salt. Pier caps, even if characterized by a concrete of higher quality than that used for piers, suffered of a high level of deterioration as a consequence of water percolation from defective expansion joints and inadequate water drainage system. The same problem affects the superstructure (beams and decks), but concentrates such a degradation to a very limited area: the bearing area. In such a way shear beam behaviour was seriously compromised. If we consider that 16% of the inspected area of the external beams are seriously damaged by corrosion and that the extension of damaged area is generally restricted to 1/3 of the total area we can assume that 48% of the external beam needs to be structurally rehabilitated. The typology of the beam (precast prestressed) require a complex rehabilitation techniques that could take into account the use of FRP and of external post-compression systems. In consideration of the great number of beams interested to the problem only an economical comparison to the cost of the beam substitution could help to properly define the technical choice to be undertaken.

7. References

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Civil infrastructure facilities, such as industrial buildings, transportation, dam and navigation facilities, and nuclear power plants, include concrete structures whose performance and function are necessary for the safety and protection of operating personnel and the general public. Aging of these structures may adversely impact their ability to withstand future operating conditions, extreme environmental challenges or accidents, and increase risk to public safety if not controlled. Methodologies for service life prediction of concrete structures and to manage the effects of aging currently are under development in many fields of civil engineering. Recent improvements have been made with respect to surveillance, inspection/testing, and maintenance techniques to help ensure continued safe, functional, and economical operation of concrete structures. The state-of-the-art has progressed to the point that specific guidelines and technical criteria required to meet these objectives in existing and new concrete construction can be proposed.

The 2nd International RILEM Workshop on Life Prediction and Aging Management of Concrete Structures was held 5-6 May 2003 in Paris, France. It brought together experts for the purpose of defining the current state-of-the-art with respect to modeling aging of concrete structures and providing recommendations (to the extent possible) for a unified approach to facility management. Participants included designers and engineers, facility managers, researchers, and regulators. The Workshop provided an international forum for highlighting recent advances in the technology underlying aging management and service life prediction of concrete structures. Approaches to service life prediction, performance monitoring, and aging management of industrial buildings, bridges and other transportation structures, dams, and navigation facilities were presented and critically appraised. The Workshop updated participants on research, presented novel ideas and strategies for management of aging of infrastructure-related facilities, provided an opportunity for exchanging practical experiences on implementation of aging management policies, and initiated new approaches to aging management. The Workshop strongly emphasized the practical aspects and the links between research and real applications. Thirty-seven papers from 20 countries were presented at the workshop. The papers were selected by members of the Scientific Committee. A sampling of specific topics that were discussed at the Workshop included:

1. Durability of concrete structures
2. Modeling techniques for service life prediction
3. Material selection and design considerations for durability
4. Assessment methods and instrumentation systems
5. Maintenance and repair approaches for service life extension
6. Probabilistic approaches to performance estimation
7. Aging management
8. Risk-informed decision-making
9. Case histories, performance assessments of degraded structures, and economics of successful aging management

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