performance evaluation of repair systems and products

TECHNICAL REPORT

duratiNet

TR 7.1

REPAIR SYSTEMS IN STRUCTURES

PRACTICAL CASES

ATLANTIC AREA
Transnational Programme
investing in our common future
European Union
European Regional Development Fund

Performance evaluation

of repair systems and products

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REPAIR SYSTEMS IN STRUCTURES - PRACTICAL CASES

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NOTE:

The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented.

PREFACE

The main subjects concerned in this TR were discussed and a general review was made inside the working group WG A7 – Performance evaluation of structural materials and new repair products. The WG was created in the DURATINET project with the aim to evaluate the adequacy of repair materials and new repair products.

This TR deals with the evaluation of existing reinforced concrete structures damaged by chlorides due to their exposure to the harsh environment of the Atlantic Area and presents four structures, in order to cover different climates and marine environments. Three structures were chosen as case studies: one in Portugal, one in France and one in Ireland.

The techniques used for their rehabilitation are presented and their performance discussed. The aim is to give to the reader information on the application and performance of these repair system in different structure types and exposure conditions.

<table>
<tr><th colspan="3">WG A7 – Performance evaluation of structural materials and new repair products</th></tr>
<tr><td colspan="3">WG Leader:
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CONTENTS

1 Introduction

This report deals with the examination of existing reinforced concrete structures damaged by chlorides due to their exposure to the harsh environment of the Atlantic Area.

In order to cover different climates and marine environments, the structures chosen as case studies within the framework of the DURATINET project are as follows:

- Barra-Bridge (Portugal),
- Ferrycarrig Bridge (Ireland),
- Saint-Nazaire Wharf (France),

Two of these structures, Barra Bridge and Ferrycarrig Bridge, have already been repaired: they are also being monitored as part of an ongoing maintenance regime. Different investigation techniques, included non-destructive ones, are used for this monitoring.

The techniques used for their rehabilitation are presented and their performance discussed. The aim is to give to the reader information on the application and performance of these repair system in different structure types and exposure conditions.

2 Description of the structures

2.1 Barra Bridge - Portugal

2.1.1 General identification

Barra Bridge over the Mira Canal, near Aveiro city (75km south of Oporto city).


a)


b)

Fig. 1. a) Barra Bridge location b) General view of Barra Bridge

2.1.2 Historical data

The first signals of a less adequate performance of the bridge was detected after starting operation in 1987, with the development of excessive deformation of the two longitudinal cantilevers forming the central span.

The technical visits that were then performed showed the existence of various problems, mainly related with the conservation of the bridge.

In July 2001, the technical visit and test plan was submitted together with the state of conservation of the bridge, namely, of the reinforced concrete. The results of that technical visit showed that the bridge was already in an advanced deterioration condition. On the basis of those elements, a decision was made to carry out an extensive rehabilitation and strengthening of the entire bridge.

In order to avoid the consequences of that situation, in April 2002, a Temporary Anchorage design for these piles was submitted, precisely after a bituminous recharge performed in July 2001. The main purpose of that anchorage was to increase temporarily the safety of the work for the period in which the final rehabilitation and strengthening works were not yet performed. That temporary anchorage was carried out at the beginning of 2003.

In 2003, a specific technical visit was carried out, which comprised: the mapping of anomalies existing in the concrete, the identification and quantification of corrosion in the different zones and the chloride penetration levels, the detection of other types of chemical deterioration in the concrete and determination of the mechanical characteristics of the concrete used in the different structural elements. It was carried out an underwater inspection for visual observation of the concrete and for collection of samples.

2.1.3 Type of structure and structural elements

The bridge has a 578.0 m length, between the support axes on the abutments. The centre span is 80.00 m and the access viaducts, symmetric to the centre span, are each 249,00 m in length, formed by one 25,00 m span and seven spans of 32,00 m.

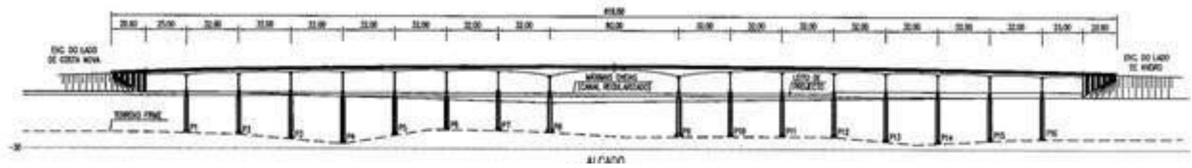


Fig. 2. Structure of Barra Bridge

Deck with 4 longitudinal Beams (viaducts), deck with two caissons (in the three central spans), piers (composed by two superposed portal frames), abutments (hollow boxes).

The structural material used is reinforced pre-stressed concrete

2.1.4 Environment

Coastal and Marine Environments.

This region is classified as one of the most aggressive according to the Portuguese Maps of Atmospheric Corrosiveness. Relative humidity of the air > 80% (Aveiro has the highest values of along the west Portuguese coast);

Classification: NP ENV 206 (4a), LNEC E 378 (ECl3)

2.1.5 Concrete class

Original (1970)

Foundations (B225-300 with 300-400 kg of cement/m^3);

Abutments and Columns (B300 with 300 kg of cement/m^3);

Deck (B350 with 450 kg of cement/m^3)

In-situ tests (2005)

The compressive tests performed demonstrated that the concrete used in the piles is of class B30 and in the beams of the deck of class B35.

Concrete Shafts (36,5 MPa)

Transverse Beams (28,8 MPa)

Piers (29,1 MPa)

Transversal Beams (31,3 MPa)

Longitudinal Beams (32,7 MPa)

Rehabilitation

Abutments (C20/25), Piers (C25/30), Deck (C30/37);

2.1.6 Steel class

Original (1970)

Main reinforcement (A 24) (A160-180 for Ø < 7 mm)

Active reinforcement (A120 for Ø ≥ 16 mm)

Rehabilitation

Ordinary reinforcement: (A 500 NR);

Active reinforcement: Cord (Y1860S7), Bars (Y1050H);

Structural steel: General (S235 JR)

Carbon fibers: Laminated (Etk = 205 GPa)

2.1.7 Cover depht

Original (1970)

Minimum/Average cover depths of main reinforcement are 30/40 mm in the zones of the piles, cross-beams and lintels and of 15/30 mm in the zones of the beams and slab.

Rehabilitation

40-60 mm

2.1.8 Other data

Additional data from In Situ Tests (2005)

Porosity wide index ≥ 11 %

W/C ≥ 0,55-0,60

Porosity Volume 1,3 – 2,5

Capillarity Index 13,7% - 17,6%

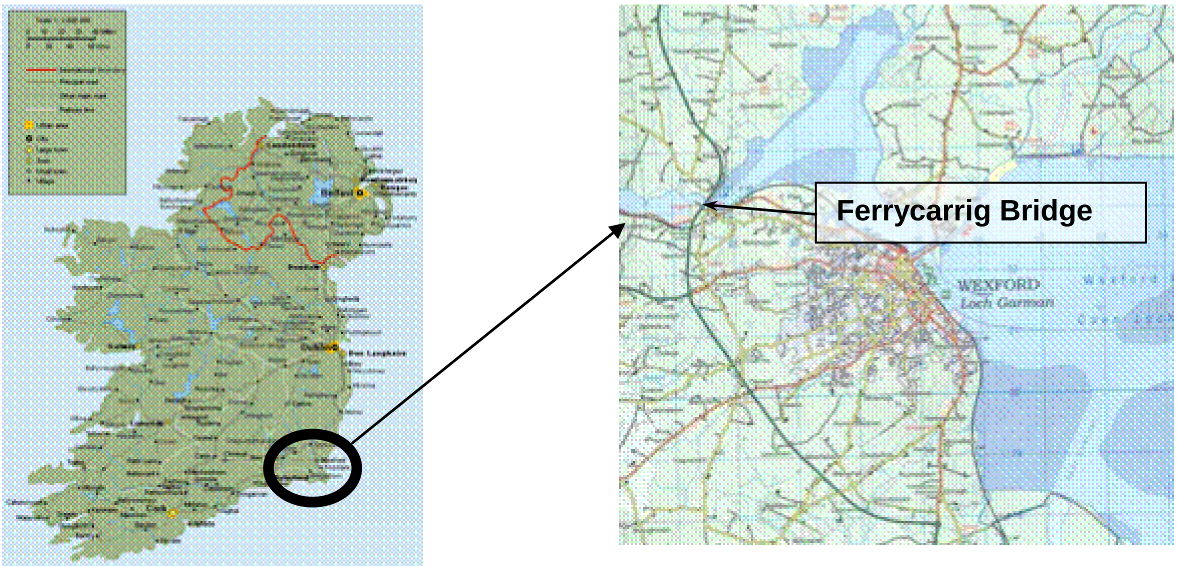
Carbonation ≤ 4mm

Chloride Penetration : 0,4% (of weight of cement) at about 6cm depth, in almost all zones At the level of the reinforcement in the tidal zone, the chloride values reached 0.5 and 2.5%.

2.2 Ferrycarrig Bridge - Ireland

2.2.1 General identification

The structure is located in County Wexford on the South East coast of Ireland. It was constructed in 1980 and is managed by the Irish National Roads Authority (NRA). The Bridge carries the N11 single carriageway road over the River Slaney as shown in Figure 3.



(a) (b)

Fig. 3. Bridge location on south east coast of Ireland

The bridge is a 125.6m long beam structure consisting of 8 equal spans of precast, prestressed beams with a reinforced in-situ concrete infill, supported on intermediate piled pier foundations with reinforced concrete abutments at both ends. Figure 4 shows Ferrycarrig bridge layout and a cross-section of bridge at pier no. 4, the bridge's middle support. The bridge is continuous over all piers except the middle pier where an expansion joint has been provided in the deck. The deck is integral with the abutments. The intermediate supports consist of two separate walls encasing steel tubular piles which are driven to rock as can be seen in Figure 4.

2.2.2 Historical data

As previously stated the bridge was constructed in 1980. In May 2002 an EIRSPAN combined inventory and principal inspection was carried out and the final stages of this inspection were completed in August 2002. The recommendations of the inspection were to carry out a special inspection on a number of elements of the bridge. This special inspection was carried out in September 2004. A structural assessment was also carried out in December 2005 as part of the special inspection. The recommendation of the reports was to carryout extensive repairs on Ferrycarrig Bridge. The repair works commenced in July 2007 and were completed in January 2008.

2.2.3 Type of structure and structural elements

The bridge is a 125.6m long beam structure consisting of 8 equal spans of precast, prestressed beams with a reinforced in-situ concrete infill, supported on intermediate piled pier foundations with reinforced concrete abutments at both ends. Figure 4 shows Ferrycarrig bridge layout and a cross-section of bridge at pier no. 4, the bridge's middle support. The bridge is continuous over all piers except the middle pier where an expansion joint has been provided in the deck. The deck is integral with the abutments. The intermediate supports consist of two separate walls encasing steel tubular piles which are driven to rock as can be seen in Figure 4.

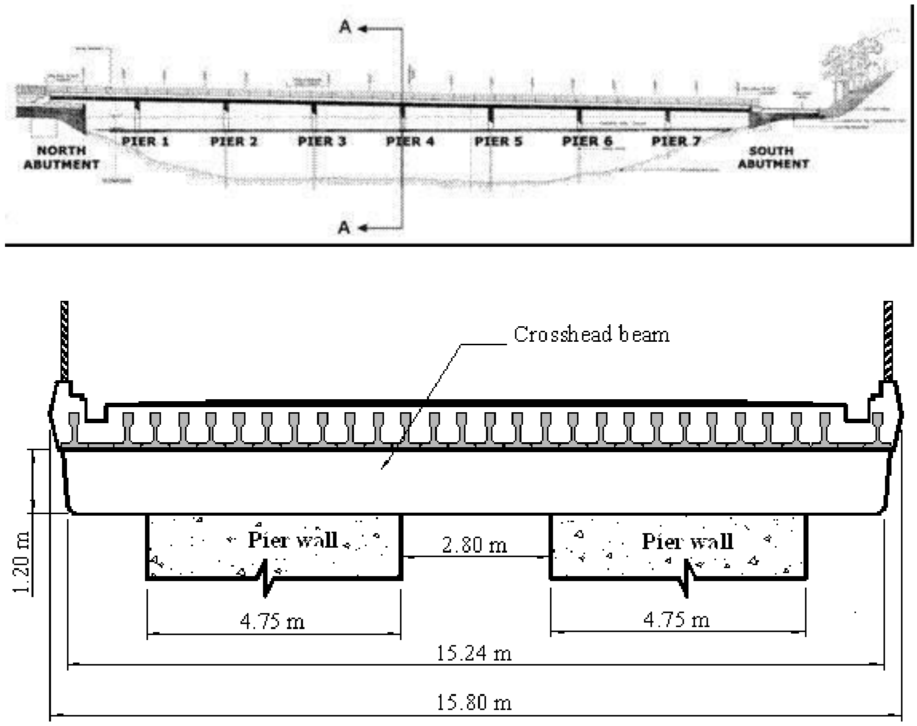


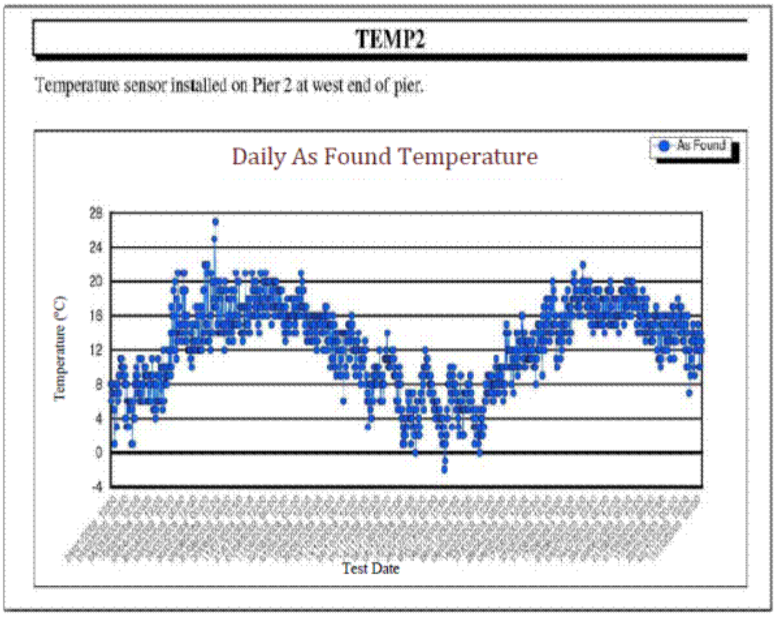
Fig. 4. Ferrrycarrig bridge layout and cross section through central pier support

Each support consists of two groups of four piles encased in in-situ concrete from the low water level to the cross head beams. The encasings are labelled pier wall in Figure 4. A photograph of an intermediate support is shown below in Figure 5. As can be seen from the figure the bridge deck sits on the crosshead beams which in turn sit on the pier walls. Each of the pier walls encase four 410 x 19mm CHS piles which are driven to rock.

Fig. 5. Intermediate Support at Ferrycarrig Bridge

2.2.4 Environment

The bridge is located in a tidal estuary on the South East Coast of Ireland. The uncased sections of the piles are constantly immersed. The pier walls are in the tidal immersion zone while the crosshead beams, bridge decks and parapets are in the splash zone. The bridge is located in a rural area. Figure 6 and 7 below present the temperature and relative humidity at the site from January 2008 to September 2009. The data comes from sensors which have been installed on one of the pier supports. As can be seen from the Figure 6 the average temperature at the site ranges from a low of approximately 4-5ºC in winter to an average high of approximately 17-18ºC in summer. The relative humidity at the site varies substantially from day to day seems to be varying at random about 70-90% relative humidity throughout the year. The relative humidity is in the range of 70-80% in the summer months and varies around 90% in the winter months.



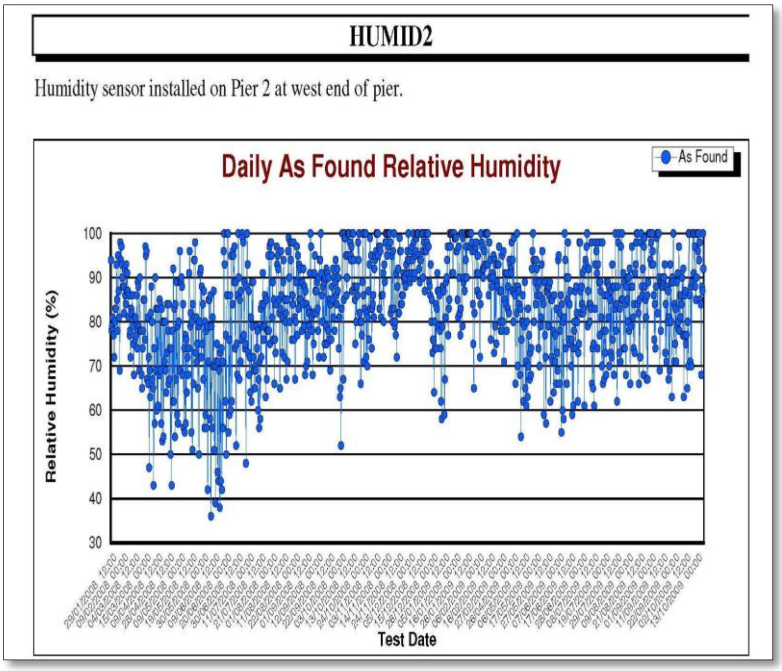


Fig. 7. Temperature and humidity data for Ferrycarrig Bridge

2.3 Saint-Nazaire Warf - France

2.3.1 General identification

This structure is "poste 1" of the "Agro-alimentaire" terminal which is built in 1971. It is located in the Loire estuary very close to the Atlantic coast (5 km) on the French west coast in Brittany (figure 8 and 9). The Nantes authority called PANSN (Port Autonome de Nantes Saint-Nazaire) managed the structure and wanted to diagnostic it for planning reparation.

Fig. 8. Quay location on west coast of France

Fig. 9. "Agro-alimentaire" Terminal of P.A.N.S.N

2.3.2 Type of structure and structural elements

The quay, 270 m long and 45 m in width, is divided in three zones. It consists of cast-in-place concrete elements: transverse, longitudinal and masterly beams (Figure 10 and 11) and precast slab elements (figure 10 and 11). Each element is spatially referenced.

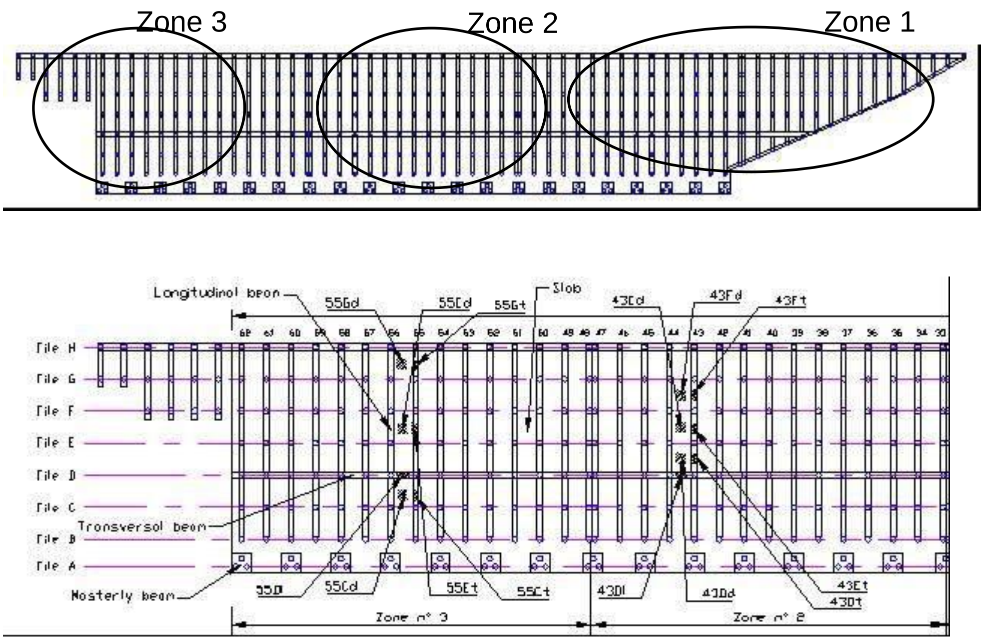


Fig.10. General sketch of the quay and zoom on zones 2 and 3.

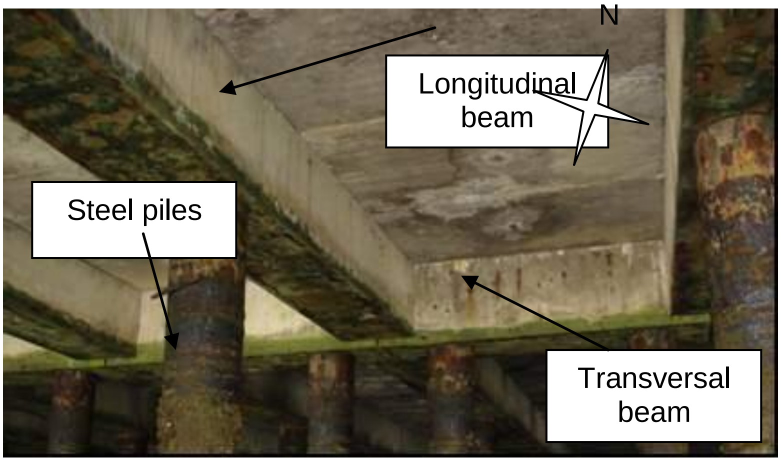


Fig. 11. Longitudinal Beam 55

The wharf authority wanted to obtain the total chloride profiles. These profiles were obtained by a private enterprise in 2005 and the repair works were completed in August 2009.

3 Inspection/Diagnosis

3.1 Barra Bridge - Portugal

In 2003, a technical visit was carried out to assist in the planning of the major rehabilitation works, this comprised:

1. The mapping of anomalies existing in the concrete;
2. The identification and quantification of corrosion in the different zones and the chloride penetration levels;
3. The detection of other types of chemical deterioration in the concrete and determination of the mechanical characteristics of the concrete used in the different structural elements;

In all structural elements (pile piers, lintels, cross beams, transversal beams, main beams, "swallow tails", slabs, caisson deck, abutments).

3.1.1 Visual inspection

Inspections showed Maintenance problems, mainly due to aggressive environmental effects. The visual inspections have shown that there is a disorganised cracking in many zones, particularly in the lintels of the piles and in the most exposed zones to sea water and rain water, which indicates the existence of mechanisms of alkali-aggregate swelling reactions or internal sulphate attack. On the pier columns, even on the most distant ones from the Ria the cracking also follows the direction of the longitudinal reinforcement, with openings between 0.1 and 0.2. In the lintel area, it is possible to observe longitudinal cracks reaching sometimes 5 mm.

Apart from that cracking, it is possible to observe zones of delamination of the concrete all over the bridge, even though being more significant on lintels and on the external beams of the deck and on the upper cross-beams of the pile. It was also possible to detect the existence of concrete zones where only the longitudinal cracking could be observed, but in which the concrete had already lost the adhesion to reinforcement and, sometimes, the reduction in the cross-section of the reinforcement was significant.

3.1.2 Compressive and impact tests

The compressive tests performed demonstrated that the concrete used in the piles is of class B30 and in the beams of the deck of class B35. Furthermore, the homogeneity of the concrete, assessed by measurement of the surface hardness by the impact test, is comparatively good.

3.1.3 Cover depth

The minimum/average cover depths of main reinforcement are 30/40 in the zones of the piles, cross-beams and lintels and of 15/30 in the zones of the beams and slab.

3.1.4 Carbonation and chlorides profile

The carbonation measured reaches maximum values of 4mm, lower values being observed in tidal zones.

Chloride penetration reaches values higher than 0.4% (of weight of cement) at about 6cm depth, in almost all zones, the highest values being observed in the tidal zones and on the downstream side. At the level of the reinforcement in the tidal zone, the chloride values reached 1.5 and 2.5%.

3.1.5 Corrosion measurements

The corrosion measurements (corrosion potential and concrete resistivity) have shown that all piles in water and even the most distant ones have already demonstrated corrosion signs.

Some of the lintels piles, the corrosion potentials range from 350 and 550 mV (copper sulphate electrode). The electric resistivity of the concrete is less than 40 kΩ.cm and in the zones of the lintels values less than 10 kΩ.cm were measured.

3.1.6 Detailed inspection

Cover depht

The cover depths of steel reinforcements in the different zones were measured using a scanning cover depth meter.

The distributions of measured parameters, on the inspected zones of the structure, were made to identify correlations with the environmental exposure conditions. Different distributions have been obtained with data collected from the inspected zones, such as, the upper crossbeams of piers, the longitudinal beams of the deck, two height levels on piers in water and on land. On each group of piers in water or on land only one distribution was obtained at each height level which gives the indication that the two zones with different sea wind orientations have similar performance to chlorides damage.

Chloride penetration into the concrete

In the cores extracted from the different zones inspected, the carbonation depths have been determined by spraying the concrete with a phenolphthalein solution after extraction from the structure. On laboratory, concrete powder samples have been collected from the cores by drilling at successive depths of 5 mm from the surface to obtain the chloride profiles. Total chloride content in the concrete was determined after dissolution in hot nitric acid by direct potentiometry using a chloride selective electrode.

Microscopic tests and other tests on the concrete

Concrete samples were also extracted for observing the concrete microstructure under polarizing and fluorescence microscopy. The microscopic examination indicates that the concrete has a homogeneous and compact Portland cement paste and some heterogeneity in the type of coarse aggregates used.

Performance data:

After various reports in the previous decade, in 2001 planned inspections and testing were conducted leading to more reliable conclusions.

1. very porous and permeable concrete
2. small cover thickness
3. high water-cement ratio
4. absence of protective coating
5. Chloride concentration of 0,4% minimum and 0,9% maximum (weight of cement)
6. Rebars electrical potential and concrete electrical conductivity reveal active corrosion in the splash zones

3.2 Ferrycarrig Bridge - Ireland

The first recorded inspection of Ferrycarrig Bridge was carried out between May to August 2002. This was an EIRSPAN combined Inventory and Principal Inspection. Table 1 presents the main defects found during the inspection and the recommended actions.

The led to a Special Inspection being commissioned in September 2004. A engineering consultancy firm were contracted to carry out all the structural investigations and testing of the bridge. The work to be carried out as part of this special inspection was as follows:

- Carry out extensive concrete conditional surveys on the pier cross heads and abutments to determine the cause and extent of cracking, leaching and staining.
- Determine the adequacy/integrity of the existing waterproof membrane (if present).
- Determine the strength and adequacy of the existing parapet system and its compliance or otherwise with current standards.
- Carry out a structural assessment to determine the load carrying capacity.
- Recommend repair or rehabilitation and strengthening works to those elements that are exhibiting deterioration or non-compliance with current standards.

Table 1. Defects identified and recommended actions

Component	Damage description	Condition rating	Action / repair recommended
Expansion Joints	Cracking over middle pier. All joints are failing as observed by water damage to cross heads and the south abutment.	2	Replacement of joint
Wingwalls / retaining walls	Cracking and calcareaous staining to south walls	2	No repair recommended
Abutments	Cracking and calcareaous staining to south abutment	2	No repair recommended
Piers	Cracking of concrete, water and reinforcement staining visible	2	Special inspection recommended
Beams / girders / transverse beams	Minor concrete splitting beneath shear links	2	Special inspection recommended

The site based work was completed between the 5th and 7th of October 2004. The on-site testing required to achieve the objectives of the special inspection report included:

Concrete condition surveys on the abutments, crossheads and beams to determine the extent and severity of the deterioration of the concrete and reinforcement

Concrete conditional surveys of the test panels to determine whether chloride contamination of the deck slab and corrosion of the reinforcement under any waterproof membrane has occurred

Excavation of Test Panels in the deck surfacing sufficient to inspect the existence and integrity of the waterproofing membranes

- Concrete core samples to determine depth of cracking;

- Covermeter surveys.
- Crack mapping survey of all crossheads and piers to determine cause, extent and severity of deterioration.

In order to determine the cause, rate and severity of any deterioration currently taking place and the potential cause of this deterioration a targeted testing program was executed. Three representative areas were chosen for this targeted testing program:

- The top surface of the prestressed beam and infill concrete bridge deck;
- The pier casings and reinforced concrete crosshead beams;
- The reinforced concrete parapet up-stand plinths.

Each of the representative areas were tested for chloride ion content, depth of concrete carbonation, depth of concrete cover to reinforcement, reinforcement corrosion using half-cell potential measurement, as well as carrying out an extensive hammer tapping survey to identify areas of delaminated concrete. Extensive mapping of cracks was carried out on all crosshead beams and piers included in the inspection. In addition, 50mm cores were taken to assess the nature of the cracks in the crosshead beams and abutments.

The following section presents is a brief summary of the findings of the on-site testing and the structural assessment.

3.2.1 Visual Inspection

The visual inspection of the bridge deck indicated that, in general, the prestressed beams were in good condition with only some minor isolated areas of low cover to shear links observed. Generally, the existing road surface appeared to be in good condition. However, the carriageway over the centre pier, corresponding to the position of expansion joint in the bridge deck, was found to be seriously deteriorated with break up of the wearing course, cracking of the surfacing and water leakage to the substructure observed.

The available drawings and documentation indicate that a single layer of ‘radmat’ epoxy was applied to the bridge deck as a waterproofing membrane. The condition of the waterproofing was found to be poor in each of the three trial pits excavated with closely spaced longitudinal cracks were apparent in two of the trial pits. One of the trial pits was located above the expansion joint. The condition of the carriageway and the joint at this location was such that the complete replacement of the joint was recommended.

Each of the accessible river piers and the reinforced concrete crosshead beams were inspected using an underbridge inspection unit. The visual inspection and hammer tapping survey revealed that although the concrete was sound with no areas of delamination found, there was a consistent pattern of both vertical and horizontal cracking the faces of each crosshead beam which is consistent with long term shrinkage or restraint to early thermal contraction effects. Finally, the steel sliding bearings which were installed at the central support were found to be completely corroded due to the failed expansion joint above and the passage of water through the joint, consequently the bearings were highly unlikely to be capable of accommodating movement.

3.2.2 Chloride ingress

The extent of chloride ion ingress in the three representative areas was assessed through the analysis of drill samples at depths 5-30mm, 30-55mm and 55-80mm. The cement content of the dust samples was also analysed. A summary of the ranges of the cement contents and chloride levels can be seen below in Table 2.

The levels of chlorides found in the structure were moderately high after 24 years. The chloride levels were highest in the crosshead beams with a maximum value of 0.64% by mass of cement as can be seen from the table. There was some debate as to the level of chloride concentration required to initiate corrosion in reinforced concrete. A commonly adopted value is 0.4% by mass of cement.

Consequently the levels of chlorides in the crosshead beams and possibly the parapet up-stands are of sufficient concentration to initiate corrosion. The levels of chlorides in the deck infill concrete at the time of investigation were not sufficient to initiate corrosion.

Table 2. Chloride and cement content results

Location	Cement content*	Chloride content**	
		Minimum	Maximum
Pier crosshead beams (Samples F1 – F18, F21)	16.4%	0.03	0.64
Trial pits – deck infill concrete (Sample F22)	14.9%	0.12	0.26
Parapet upstand beams (Samples F19 and F20)	14.9%	0.11	0.35

* Cement content determined from testing
** Percentage chloride by weight of cement

3.2.3 Carbonation survey

Concrete carbonation depths were measured in all of the test areas and in general, the levels of carbonation penetration were found to be very low and are well below the typical cover to reinforcement, with a maximum measured penetration of 17mm.

3.2.4 Covermeter survey

A covermeter survey was carried out in localised areas throughout the bridge. In general the results confirmed the reinforcement arrangement shown in the original design drawings from 1980; however the prestressed beams were an exception to this.

The drawing indicated a minimum depth of cover of 40mm on all exposed surfaces and 25mm elsewhere. The recorded cover measured in the crossheads is generally consistent with this (22 – 63mm) with an average depth of cover of 48mm noted. The drawings indicated cover to the prestressed beams of 33mm. However, in general, cover to links in the prestressed beams was found to be very low with some links exposed near to the pier supports. The cover survey carried out indicated that the cover to the links in the prestressed beams varies from 0mm to 25mm with an average cover depth of 17mm determined.

3.2.5 Half cell potential survey

A half-cell potential survey was carried out on all accessible surfaces of the pier crosshead beams. The resulting data is summarised in Table 3. In order to interpret half cell potential measurements the risk of corrosion associated with the various ranges of readings must first be defined (ASTM, 2009, Concrete Society, 2004).

- Reading >-200 mV Copper Sulphate Electrode (CSE): high likelihood that no corrosion is occurring – Low Risk
- Reading -200 mV CSE to -350mV CSE: corrosion activity is uncertain – Medium Risk
- Reading <-350 mV CSE: high likelihood that there is active corrosion – High Risk

Table 3. Half-cell potential results

Location	Half-Cell Potentials		
	> -210 mV	-210 mV to -360mV	< -360 mV
Pier 1	93%	7%	0%
Pier 2	37%	59%	4%
Pier 3	35%	64%	1%
Pier 4	54%	24%	22%
Pier 5	76%	23%	1%
Pier 6	74%	26%	0%

The half-cell potential data obtained at the time of the inspection suggests that the probability of corrosion was generally low to uncertain in each of the pier crosshead beams except at Pier 4 where very negative electrochemical half-cell potentials were observed in the cantilever sections. It is likely that the high concentration of chlorides and availability of both oxygen and water has led to the negatively depressed electrochemical half-cell readings in these areas. However, the condition of the concrete at the level of the reinforcement is such that corrosion had not been initiated at the time of testing.

3.2.6 Concrete cores

The cores were drilled to a depth of 90mm at various locations on the crosshead beams. It was found that the cracking did not taper and were evident for the full depth of core. This was considered to be consistent with either long term drying shrinkage or restraint to early thermal contraction but tended to rule out flexural bending cracks. In addition, this indicated that the cracks were of serious concern with regard to the long term durability of the structure as the cracking extended well beyond the depth of the reinforcement.

3.2.7 Structural diagnostics

For deterministic assessment the structure was modelled as a 3-dimenionsal space frame model as illustrated in Figure 12. This model was used for two purposes: (a) the calculation of ultimate load effects for superimposed dead and live loads in order to determine the ultimate load effects on the structure and (b) the calculation of the serviceability and working load effects for superimposed dead loads, live loads, creep and shrinkage, temperature and differential settlement. The latter was used to develop an understanding of the possible reasons for development of cracking observed at the pier crossheads and at the top of the deck over the pier diaphragms.

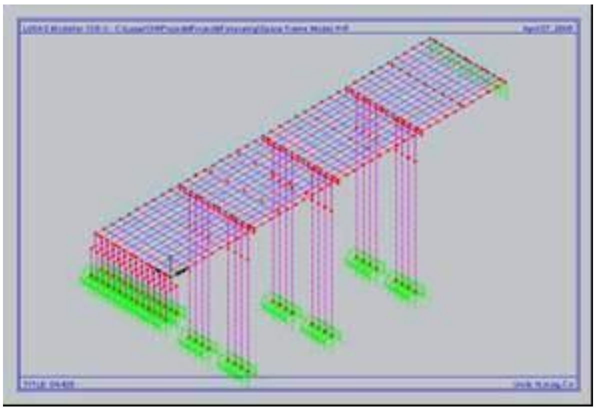


Figure 12. FE Model of structure

Bridge Deck

The structural assessment showed that the bridge deck was capable of carrying the ultimate loads due to the 40/44 tonnes HA Assessment Live Loading together with 45 units of HB in isolation or combination as prescribed by the Irish National Road Authority's Design Manual for Roads and Bridges (NRA, 2002). Stress indices of 0.91 and 0.82 were determined in bending and shear respectively. The stress index was taken as the ratio of calculated assessment load effect to the respective assessment resistance. As such a stress index of 1.0 or less was taken to indicate compliance with the standard.

The results of the serviceability limit state checks indicated the level of overstress at the top of the in-situ concrete was an issue and the concrete would crack at this location under HA and HB loading.

Pier Diaphragms

In order to investigate the transverse cracks in the pier crosshead a transverse stress analysis was conducted. It was found that in the transverse direction two groups of four piles create a significant restraint to the shrinkage of the in-situ concrete. Further assessment of the pier diaphragms indicated that there was a significant shortfall in the reinforcement provided to resist the induced shrinkage strains. The analysis demonstrated that the ULS torsion moments (under HA and HB loading) exceeded the torsion capacity of the pier diaphragm. The torsion capacity of the diaphragm may also have been exceeded during construction, depending on the construction sequence.

Abutments and Parapets

The abutments were shown to have sufficient capacity to carry the factored loading criteria of BD 21/01 (Highways Agency, 2001) and BD 37/01 (NRA, 2002) in accordance with the Highways Agency design manual. A dimensional review of the existing steel parapets found that they conformed with the British Steel P2 (113) vehicle pedestrian parapet thereby satisfying the requirements of BD 52/93 (Highways Agency, 1993).

3.2.8 Conclusions of deterministic assessment and special inspection

Following the completion of the inspection, testing and structural assessment of Ferrycarrig Bridge, a number of deficiencies from standards that affected the long term serviceability of the structure were identified that required repair and strengthening works to be carried out. These issues are summarised as follows:

- The observed cracking in the pier crosshead beams was considered to be due to a lack of reinforcement to resist the SLS stresses (i.e. shrinkage, thermal, creep). In addition, there was insufficient reinforcement to resist the applied ULS torsion moments. As it was anticipated that the cracks would continue to develop and would thereby ultimately compromise the integrity of the piers it was concluded that pier strengthening works should be carried out;
- Chloride levels in the concrete indicated a distinct concentration gradient decreasing rapidly through the concrete. The levels of chloride ion concentration in the crosshead beams were considered moderately high.
- Other elements of the bridge needed to be replaced, namely, the bridge deck waterproofing system, the bridge deck expansion joint and the mechanical bearings at Pier 4.

The costs associated with three repair options were explored. The 3 strategies, sorted by Present Value are given in Table 4 below.

Table 4: Cost associated with three repair options

Option	Strategy	Present Value	Factor
1	**Immediate Refurbishment of Structure**	**€1,422,234**	**1.00**
2	**Wait 10 years before refurbishing the bridge**	**€1,687,734**	**1.19**
3	**Do nothing**	**€6,007,289**	**4.22**

3.3. Saint-nazare wharf - France

The inspection was carried out in 2005 and consisted only in visual inspection and chloride profiles to determine the polluted concrete depth. As this structure is in marine environment the depth of concrete carbonation was not determined.

The following section presents is a brief summary of the findings of the on-site testing and the structural assessment.

3.3.1 Visual Inspection

The visual inspection of the bridge deck indicated that, in general, the masterly beams and the precasted slabs have low damaging. The visual inspection was done for each structural element and reported on tables. The damaging rate was classified in three types:

- Type 1 : low degradation : no apparent corrosion
- Type 2: medium degradation
- Type 3: high degradation

Table 5 - Visual inspection. Example of longitudinal beams report

Location	Photo	Comment	degradation type
17D/18D		Cracking and corrosion stains in angles	2
18D/19D	329	Apparent steel + concrete expansion + corrosion craking	3
19D/20D		Apparent corroded steel + concrete expansion	3
20D/21D		Cracking and corrosion stains in angles	2
21D/22D	327	Craking and corrosion stains in angles	2
22D/23D		Craking and corrosion stains in angles	2
23D/24D		Craking and corrosion stains in angles	2
24D/25D		Craking and corrosion stains in angles	2
25D/26D		Craking and corrosion stains in angles	2
26D/27D	319	Craking and corrosion stains in angles	2
27D/28D	320	Concrete delamination + Corrosion stains in angles + corrosion craking	3
28D/29D		Craking and corrosion stains in angles	2
29D/30D		Craking and corrosion stains in angles	2

30D/31D		Concrete delamination + Corrosion stains in angles + corrosion craking	3

Photos are reported on the structures as shown in figure 13 to determine chloride profile localisation. The profiles were done for the three degradation types.

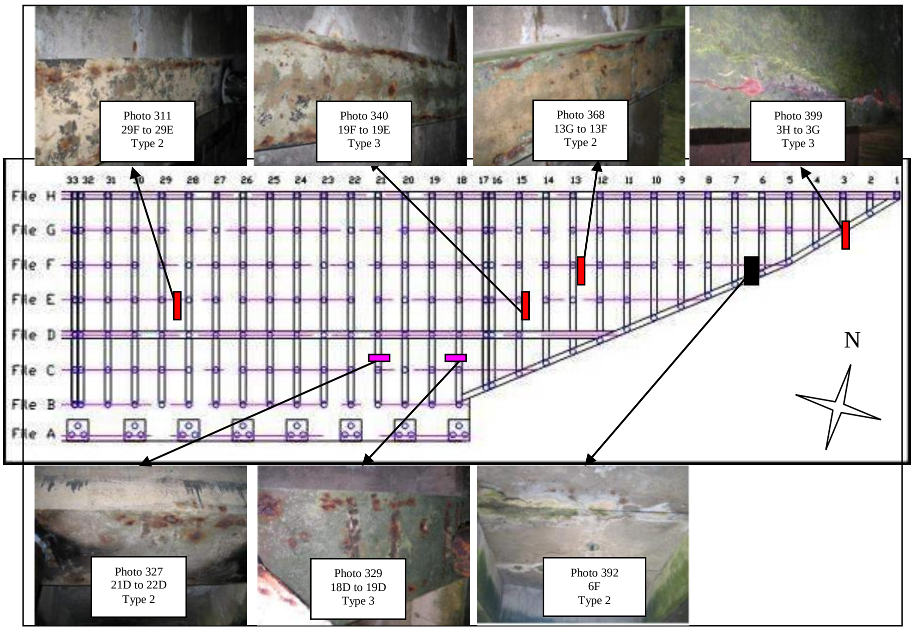


Fig.13. Photo reporting visual inspection

3.3.2 Chloride ingress

For the transversal beam, each marked point includes three measurements: in the east face (E), west face (W) and under face (UF). Only the total chloride ions profiles were measured on these cores. The chloride ions concentrations are determined as following: the cores of 40 mm diameter and 80 mm length are extracted by wet process, and then are sawn and crushed to obtain a powder (figure 14a). The objective is to evaluate the chloride concentration every 20 mm. Then to get the value at the middle point of one 20 mm length sample, the laboratory crushes 25 cm^3 of concrete. This wetting method does not have influence on the measured concentration (Wood *et al*, 1997), (Truc, 2002), (Amiri *et al*, 1997), (Friedmann *et al*, 2004), (Amiri *et al*, 2004) (Castellotte *et al*, 2001), (Kirkpatrick *et al*, 2002), (Gaal *et al*, 2003). The chloride ions are extracted from the concrete powder by acid attack and filtering to get a chloride solution which is titrated by the potentiometer method (figure 14b and 14c).

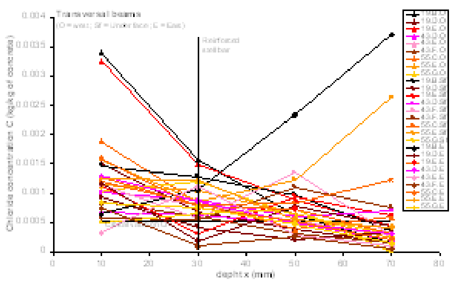
a) *b)* *c)*

Fig. 14 Chloride profile determination. a) concrete coring, b) filtering, c) titration

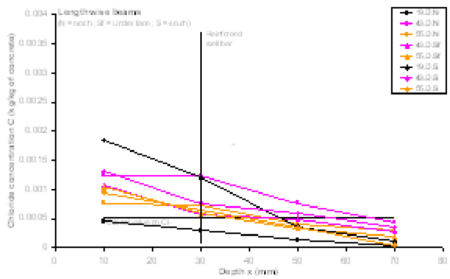
The total amount of chloride is given in chloride by weight of concrete as no data were found on the concrete mixing.

All the studied elements are subjected only to the spray sea water and are never in direct contact with sea water excepted during exceptional storms. As these elements are under the quay, the climate exposure is not influent (no sun, no rain) except for the masterly beams which are in front of the quay.

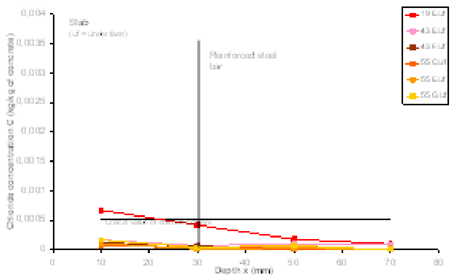
The chloride profiles are similar for longitudinal transversal and masterly beams. Only slab have low chloride content which agrees with the visual inspection: no corrosion stain were detected. These slabs are less exposed to sea water because these are higher.



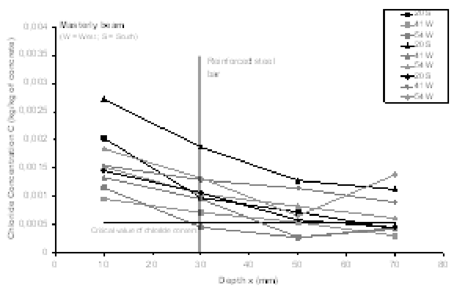
15a: transversal beams



15b: Longitudinal beams



15c: Slabs



15d: Masterly beams

Fig.15. Total chloride profile

By comparing these profiles with the critical chloride concentration value and the position of the first reinforcement (3 cm), it is obvious that almost all steel bars present a strong risk of corrosion, even if this is not visible on the beams surface. Let us recall that corrosion is supposed to start for concrete reinforcement when the chloride concentration reaches 0.00052 kg/kg of concrete (value suggested by European rule EN 206: 0.4% of chloride ions by cement weight (NF EN 206-1).

4 REPAIR

4.1 Barra bridge - Portugal

4.1.1 Proposed Reinforcement Solution

The aim of the rehabilitation design was to adjust the bridge to the new code rules, thus securing the improvement of its performance levels. The structure durability performance was highly conditioned by the atmospheric corrosion.

Reinforce of the deck

- External longitudinal Pre-Stressed cables;
- Use of laminated carbon fibers (LFC);
- Reinforce of the caissons bottom slabs;
- Reinforce of the caissons bottom web;
- Supports reinforcement of the simply supported span.

Piles/Column Reinforcement

- Use of laminated carbon fibers (LFC);
- Anchor devices of P7 and P10 piles.

Intervention in the Abutments

- Releasing the deck in the longitudinal direction;
- Fixing the simply supported span deck, in the longitudinal direction;
- Introducing viscous-elastic devices in the deck connections;

4.1.2 Repair Zones (Proposed concrete solution)

Crosshead 1: Zones where chloride contents have not yet reached the reinforcement: Repair with pre-mixed mortar;

Crosshead 2: Zones where critical chloride contents reached at the level of reinforcement: Removal of all the cover concrete until 2cm beyond the reinforcement and replacement with low A/C sprayed concrete.

Substitution of the contaminated concrete and increase of rebar concrete cover thickness with sprayed concrete or hand applied mortar - including corrosion inhibitors;

Suitable protective coating for different micro-environments;

Application of corrosion inhibitors of migration in all the surfaces where there was no removal of the concrete and painting with a thick and elastic cover, of low permeability to water and to chloride diffusion but permeable to water vapour and with anti-fungi.

4.1.3 Surface protection:

Application of corrosion inhibitors of migration in all the surfaces where there was no removal of the concrete and painting with a thick and elastic cover, of low permeability to water and to chloride diffusion but permeable to water vapour and with anti-fungi.

4.2. Ferrycarrig bridge - Ireland

Due the strategic importance of Ferrycarrig Bridge, i.e. it lies on the so-called Euroroute from Dublin to Rosslare Port, a decision was made to repair and strengthen the structure immediately. From a network maintenance planning perspective, it was apparent that the bridge rehabilitation works would afford the NRA a unique opportunity going forward to gather information regarding the efficiency of typical alternative concrete repair options in Irish marine environments. It was therefore decided to employ five different concrete repair strategies for the seven crosshead beams. Six crossheads would be instrumented and remotely monitored so that the relative efficiency of the various methods could be studied over time. In all cases hydro-demolition was used at the crossheads to remove concrete to a depth of 1.5 times the reinforcement diameter beyond the existing reinforcement before additional steel and repair concrete was put in place. The alternative concrete repair strategies developed are as described in the following sub-sections.

In July 2007 refurbishment works on Ferrycarrig Bridge commenced. An Irish contractor was awarded the contract to perform:

i. Extensive concrete repair to all crosshead beams,
ii. Re-waterproofing of the existing bridge deck including footways,
iii. Replacement of the existing bearings at the central pier,
iv. Replacement of the expansion joint over the central pier,
v. Repainting of the existing parapet system and
vi. Crack injection of the abutments. Works were completed in January 2008.

Crosshead 1 - Ordinary Portland cement (OPC) Mix

This repair option (used for Crossheads 1 and 7) was selected to act as a control for the other repairs. Here it was intended to follow the consultants proposed rehabilitation option for the crossheads of the structure, conforming to the NRA standard specification, with a standard 50mm cover to the new reinforcement. The replacement of the old concrete surrounding the reinforcement with the OPC mix is expected to restore the alkalinity surrounding the reinforcement.

Crosshead 2 – Ordinary Portland Cement with Increased Cover

In this option the cover was increased over that employed in option 1 from 50 mm to 70 mm. As increasing the cover increases the distance that chloride ions need to migrate to reach the reinforcement, this option is expected to increase the time to corrosion initiation. However, the increase in cover was achieved by repositioning the reinforcing bars without increasing the overall dimensions of the cross-head so the structural efficiency of the reinforcement was reduced. In addition, the literature states that cover should not be increased beyond 80-100 mm as this volume of concrete devoid of reinforcement could lead to excessive cracking due to shrinkage and thermal stresses (Nevile, 1995).

Crosshead 3 – Ordinary Portland Cement + Surface Treatment

For Crosshead 3, an OPC mix was used together with a surface impregnation treatment to prevent penetration of chlorides from the exterior environment. Two coats of monomeric alkyl (isobutyl) trialkoxy-silane, commonly known as silane, were applied to all the surfaces of the crosshead beam. The silane penetrates the substrate of the concrete and creates a hydrophobic layer which prevents water and waterborne contaminants from entering the substrate and causing premature deterioration in the crosshead beam.

Crosshead 4 – Ground Granulated Blast Furnace Slag (GGBS) Mix

For Crosshead 4 it was decided to use GGBS as a partial replacement in the Portland cement mix. Available literature demonstrates that the chloride diffusivity into the concrete significantly decreases when GGBS is used in the range of 50% to 70% per weight of binder (Snidel, 2007). This is primarily due to the low permeability of the OPC - GGBS binder mix which reduces the ingress of water into the concrete and thus reduces the rate of chloride ion ingress. It is also thought that the use of GGBS, like other blended cements, increases the resistivity of the hardened concrete (Nevile, 1995). A minimum value of 60% GGBS by weight of total cement was used at Ferrycarrig bridge. It is significant to note that Pier 4 is the location of the expansion joint in the structure and is thus likely to be exposed to a more severe environment than the other crosshead beams.

Crosshead 5 – same as Crosshead 1 + mixed-in corrosion inhibitors

For crosshead 5 an OPC mix was utilised with the addition of an organic type mixed in corrosion inhibitor which addresses both the anodic and the cathodic reactions of the electrochemical corrosion process.

Crosshead 6 – same as Crosshead 4

As discussed above, Crosshead 4 was repaired using GGBS in the mix design. The purpose of employing the same repair option for Crosshead 6 was to compare the performance of the GGBS mix at and away from the expansion joint.

Crosshead 7 – same as Crosshead 1

Crosshead 7 was repaired in the same way as Crosshead 1. This was done in order to provide an opportunity to study the spatial variability of chloride ingress for the structure. Crossheads 1 and 7 act as the control in the study.

4.3 Saint-Nazaire Wharf - France

After inspection, the Nantes wharf authority decides to remove damaged and contaminated concrete by hydraulic shooting. It was decided to repair with dry shotcrete which is a traditional way. This repaired quay was monitored by IFSTTAR and GeM to control durability parameter. Complementary tests are still carried out: they concern natural and accelerated tests of chloride ingress for several repair materials used on-site. Moreover, a monitoring based of the measure of resistivity has been developed and tested. Results will be presented in further publication.

To evaluate durability performance an exhaustive experimental study was done on different repaired method with materials which are easy available with French furnishers. This research project was also done in the framework of a French national project called MAREO ("MAintenance et REparation des Ouvrages litoraux en béton" - http://www.pole-geniecivil-ecoconstruction.fr/download_assets/12). This study focus then on two different aspects:

- the durability of four coupled repair techniques/materials based on ready to mix concrete;
- the comparison of carbon (FeE500) and stainless steel (grade I.4362) performance associated with the previous repairing methods.

The four coupled repair techniques/materials based on ready to mix concrete tested and characterized are:

- Dry shotcrete (aggregates size 0-8 mm);
- Wet shotcrete (aggregates size 0-2 mm);

- Mortar implemented manually with the same material than the latter one (aggregates size 0-2 mm);
- Formed concrete (aggregates size 0-10 mm).

This experimental work is structured around tests on structures (beams in natural area) and laboratory tests. When the beams were repaired with the studied method/material, slabs were also cast for laboratory tests: some to determine indicator durability for these materials and some to evaluate durability performance with accelerated tests. The details and the results of these tests are presented in technical report (7.2 Repair systems in small scale samples exposed in experimental sites)

5 DATA OF IN-SITU PERFORMANCE

5.1. Barra bridge - Portugal

After the rehabilitation, it was established a monitoring system based on sensors embedded in the concrete to act as support instruments to the maintenance of the structure.

The main objective was the long term evaluation of the evolution of the corrosion at the main reinforcement and efficiency of the repairs at the structure.

The system consists on an automatic acquisition of data, which allows informing about the evolution of chlorides and carbonation at the concrete and also the detection of corrosion at the reinforcement due to the action of previous agents described.

The system developed, based on galvanic current, corrosion potential, concrete resistivity, and temperature measurements, was also designed to evaluate the efficiency of the localized repair processes of concrete adopted in some areas, as well to evaluate the efficiency of the coatings and inhibitors of corrosion in corrosion protection of reinforcement.

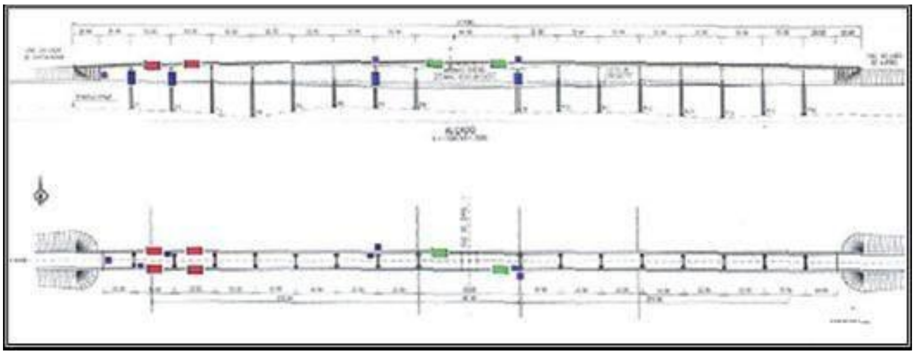


Fig. 16. Location of the instrumented areas - (i) 4 spots: external side of longitudinal beam, between P1, P2 e P2,P3 piers; (ii) 2 spots: external side of box girder, between P8 e P9 piers; (iii) 12 spots: piers P1, P2, P7 e P9.

Test Methods

The test methods used depending on the repairs done (1) total or (2) partial removal of the bar cover concrete, followed by replacement of the reinforcement in some areas, replacement of bar cover, covering the areas of concrete removed, application of a corrosion inhibitor and a general surface protection coat. For the instrumentation were created three types of zones:

- Areas of the original concrete removed:

  2 resistivity sensors at 15 e 30 mm of bar cover

  2 galvanic current sensors at 15 e 30 mm of bar cover

  1 thermometer at 15 mm of bar cover

  1 corrosion potential reinforcement sensor

- Areas without removal of concrete:

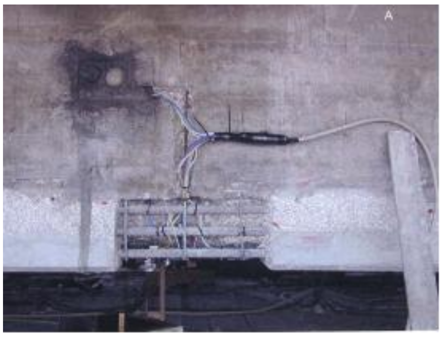
  2 resistivity sensors at 15 e 30 mm of bar cover

  1 galvanic current sensor at 15 mm of bar cover

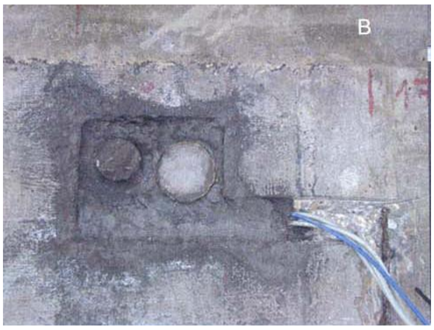
1corrosion potential reinforcement sensor

- Interface areas between the original concrete and concrete used in repair:

  1corrosion potential reinforcement sensor



a)



b)

c)

Fig. 17. a) Instrumentation of the longitudinal beam; b) instrumentation: were original concrete wasn´t removed; c) Instrumentation: were original concrete was removed

Fig. 18. Instrumentation of the pier.

Results

The results analysis from the instrumented areas during the period 25/7/2008 and 25/10/2010 showed the following:

- Structural steel was in a passive state since the corrosion potentional values register are superior to -320mV vs MnO_2 at the aerial areas of the structure. At the lintels the corrosion potential values were lower, and an average of -524mV vs MnO_2 at pier P1 and -509mV vs MnO_2, predictable situation due to the high degree of saturation of the concrete at this site and due to the type of external layer applied to the pier surface.
- The values of galvanic current register are low and indicated that in the areas of where the concrete was removed; At all instrumented sections the penetration by the corrosion agents isn't observed till 1.5 cm of bar cover.
- In 2010 the galvanic current sensors at the external longitudinal beam and between the piers P2 and P3, register oscillating values compared with 2009 results, due to the starting of the acquisition of the data system installed at the pier P3 after disruption of the current.
- In September 2008, that was a massive reduction of the electric resistance at the longitudinal beam between P8 and P9 at 15mm of bar cover, which could be due to a possible crack at the concrete. These results weren't register at 30 mm which indicate that the crack doesn't prolong to the inside. In July 2009 the electric resistance results at 15 mm of bar cover were increased which indicate one "colmatation of microcracks".
- At the remaining sites apart from the initial values variations due to the concrete curing there were register some variations threw time but were only due to different environment temperatures. However, there was verifiable deficiencies at the electricity supply to the data acquisition systems which brought to invalid results or even losses of data in 2008 at the P2 and P7, in 2009 at the P2, P3 and P9, and in 2010 at the P3 and P9 which envolve in a loose of efficiency on the monitoring systems.

5.2. Ferrycarrig bridge - Ireland

In order to facilitate monitoring of the relative efficiency of the different repair techniques six crosshead beams were instrumented. The instrumentation scheme involved the installation of three different types of probes:

i. corrosion potential probes,
ii. chloride ion penetration depth probes and
iii. corrosion rate probes.

These were secured to the retrofitted crosshead reinforcement cage. In total ten probes were embedded in each crosshead. The southern and northern faces of the crossheads were each instrumented with one chloride ion penetration depth probe, two corrosion potential probes and one corrosion rate probe. The eastern, or seaward, end of the crosshead beams were instrumented with a chloride ion penetration depth probe and a corrosion potential probe. The undersides and western end of the crosshead beams were not instrumented. Temperature and humidity sensors were installed on the eastern face of crosshead 2, the eastern face of crosshead 5 and the western side of the South abutment. Figure 19 shows the three different types of corrosion probes attached to the retrofitted reinforcement cage before the repair concrete was placed. In the figure the chloride-ion penetration depth probe is labelled A, the corrosion potential probe is labelled B, and the corrosion rate probe is labelled C.

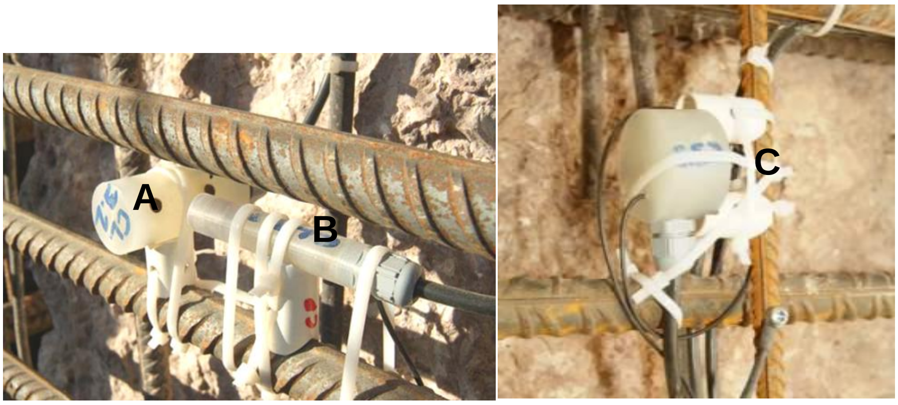


Fig. 19. Chloride and corrosion probes

It is now 2 years and six months since the probe monitoring system has been put in place. According to the monitoring system managers the monitoring system is still in it's initial bedding in period meaning the long-term benefits of each repair strategy cannot be determined due to the fact that the responses observed may be to a greater or lesser degree due to the extensive disruption to the crossheads during the repair sequence. Consequently, observed differentiation between repair strategies may not become apparent for some years, thus the probe results to date will not be discussed in this document.

A visual inspection of Ferrycarrig Bridge was carried out in on the 19th of January 2010. The bridge repairs were found to be performing well with the exception of the expansion joint. It was found that the expansion joint had failed just two years after construction. The precise cause of the failure has yet to be discovered. The effect of the failed expansion joint can be seen below in Figure 18. The expansion joint is located above the central support, the support closest to the camera. The water staining on this crosshead beam can clearly be seen. The initial steps to solve the problem are currently been taken.

Fig. 20. Photograph of effects of failed expansion joint

6 CONCLUSION

This report summarises a series of case studies relating to the deterioration, repair and subsequent performance of reinforced concrete structures in a marine environment. The repair techniques are described and the relevance of their use is highlighted. The structures are located in different Atlantic climates (in the north of Europe in case of Ferrycarrig in Ireland, in the middle for Saint Nazaire Wharf and in the south for Barra Bridge, and Alfeite Base). The repair techniques used were typical systems that are well known in each country. It is clear from an examination of these case studies that patch repair is the most common technique, a result of the relatively low costs involved.

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