00479 ELECTROCHEMICAL CORROSION TESTING IN TWO MEXICAN PIERS

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ABSTRACT RÉSUMÉ

An electrochemical evaluation of two pier substructures was performed in order to contrast corrosion parameters and to find similarities or differences that could help to elucidate the environment effect on the structural performance, and to outline maintenance actions to extend both piers service lives.

The results are discussed in terms of mechanical, chemical, and electrochemical tests performed to both piers such as: Compressive strength, sclerometric index; corrosion potential; carbonation front; chloride content; detections and quantifications of the steel reinforcement; and identification and characterization of the observed damages.

The results shown that the recommendations to Port administrators included the rehabilitation of the piles, using two rehabilitation procedures: 1) 15% of North Pier piles and 20% of South Pier piles should be repaired by replacing corroded reinforcement and damaged concrete; 2) 24% of North Pier piles and 48% of South Pier piles should be repaired through cleaning of corroded steel and replacement of the damaged concrete section. All repaired piles (61% and 72% for North and South Piers, respectively) should be protected cathodically.

1. INTRODUCTION

The present investigation includes the corrosion-induced damage of two Mexican ports located in the Pacific Ocean. The first one, called North Pier, is a multimodal option which connects Asia markets with U.S.A., Mexico, and South America. The second one, called South Pier, is located closer to the most important industrial areas of Mexico. It is the mayor entry port of containerised cargo; it ranks seventh among the major port terminals in Latin America.

Corrosion damage is a mayor concern among Mexican Port Authorities, especially because most Mexican ports have aging piers. Few exceptions like the one located in Progreso, Yucatan Peninsula, is in good durability conditions after its main structure (2 km long concrete pier viaduct) has being in service for more than 65 years [1]. It is relevant to state that this pier has special preservation considerations due to its historical background and relevance, so has been consistently kept under close performance supervision

North and South Piers were built in 1954 and in the 70's, respectively. North Pier was expanded and improved around the 90's, South Pier in the 80's. However, the repairs made in both piers were inadequate, thus, in a very short period of time, corrosion damages started again. The repairs in both piers were basically placing a reinforced concrete jacket, surrounding the original pile sections. Due to low quality concrete used in these external jackets, its reinforcement started to corrode less than 5 year after placement. This repair method helped to protect the original reinforcement basically because the active steel in the jackets worked as sacrificial anodes. In 2003 an earthquake damaged some piles of South Pier, a general inspection took place and some piles were repaired using similar method (reinforced concrete jackets).

During 2005 and 2006 both piers were inspected to establish the deterioration level due to corrosion damage and to propose their rehabilitation projects [2, 3]. Based on the results of these projects, the rehabilitation costs for North Pier was $6 million dollars (US) and $11 million dollars (US) for South Pier.

The objective of this investigation is to assess the electrochemical evaluation of the two environmentally damaged piers substructures, in order to contrast corrosion parameters and to find similarities or differences that could help to elucidate the environment effect with their structural performance, so maintenance actions can be outlined to extend both piers service lives.

2. EXPERIMENTAL DETAIL

Durability-based inspections were performed for the 50 years old pier (South) and the 35 years old pier (North), both located in a tropical marine environment. North Pier has 2,664 piles, and South Pier 4,177 piles. Both were inspected between 2005 and 2006. A photographic catalogue and a damage survey in all piles, from both piers, were performed to have a reference for the rehabilitation work needed.

A detailed visual inspection of the structure was carried out in order to a) identify the different substructure damage types observed, b) estimate the extent of such damages on both pier substructures, and c) decide the location of field and laboratory tests to be conducted and the extent of sampling to be done. Additionally, mechanical, chemical and electrochemical tests were performed to characterize each Pier’s material condition. Such tests included: Concrete mechanical properties (concrete cover depth, compressive strength, and sclerometric index); concrete chemical attack (carbonation profile and chloride concentration); steel reinforcement properties (diameter, location, and half cell potential); Damage type and location at the pile surface. Table 1 shows a summary of the tests performed.

Table 1 – Resume of the testing done

Tests	Number of Tests	
	North Pier	South Pier
Damage survey (# piles)	2664	4177
Concrete cores	26	41
Compressive strength	15	26
Chloride concentration	39	48
Carbonation depth	11	12
Sclerometric index (series)	100	115
Half cell potential (m^2)	180	200
Concrete cover	25	41
Rebar diameter	25	41

3. RESULTS AND DISCUSSION

3.1. Remarks over the detailed visual inspection:

• All the major damage was found to be located on the pier substructure. The top surface of the concrete decks of both piers had no apparent defects except for abrasion or wearing of the concrete surface.

• Spalling of concrete was the most common defect found at the beams and the piles at both Piers. The spalling was noted at various stages, from developing crack lines to spalled cover concrete. This can be seen in Figure 1. Rusty reinforcement, or lack of reinforcement due to corrosion, was noticed at some areas where the concrete cover had spalled off. At locations where previous repairs were instaled, the whole patch repairs had started to crack and spall due to the expected phenomenon of macro-cell corrosion. Figure 2 shows the occurrence of incorrect repairing.

Figure 1- Lack of reinforcement due to advanced state of corrosion on the North and South Pier respectively.

Figure 2- Incorrect repairing of a corroded pile on the North and South Pier respectively.

In both piers, the environmental effects, combined with deficient construction processes caused considerable deterioration. Greatest damage was present at the piles tidal and splash areas. Cracks, delaminations, and steel reinforcement cross section loss, were the most common pathological manifestations shown in both piers. Most damages in the piles were located precisely on the area identified with a higher content of chlorides, and more negative corrosion potentials.

In some areas, steel corrosion that began at the tidal area, spread over time towards higher heights of the piles, even though in those places chloride levels were lower than tidal zone, its concentration was larger than the 0.4 % (by cement weight) which is the so called threshold value. Therefore, if thinner concrete covers, cracks, and other constructive defects were present, the corrosion spread quickly. Figure 3 shows an example of the damage survey performed included the identification and measurement of crack lengths, delaminations areas, and corrosion spots on each pile face in both piers. Location drawings were plotted allowing a rapid location of each pile in particular.

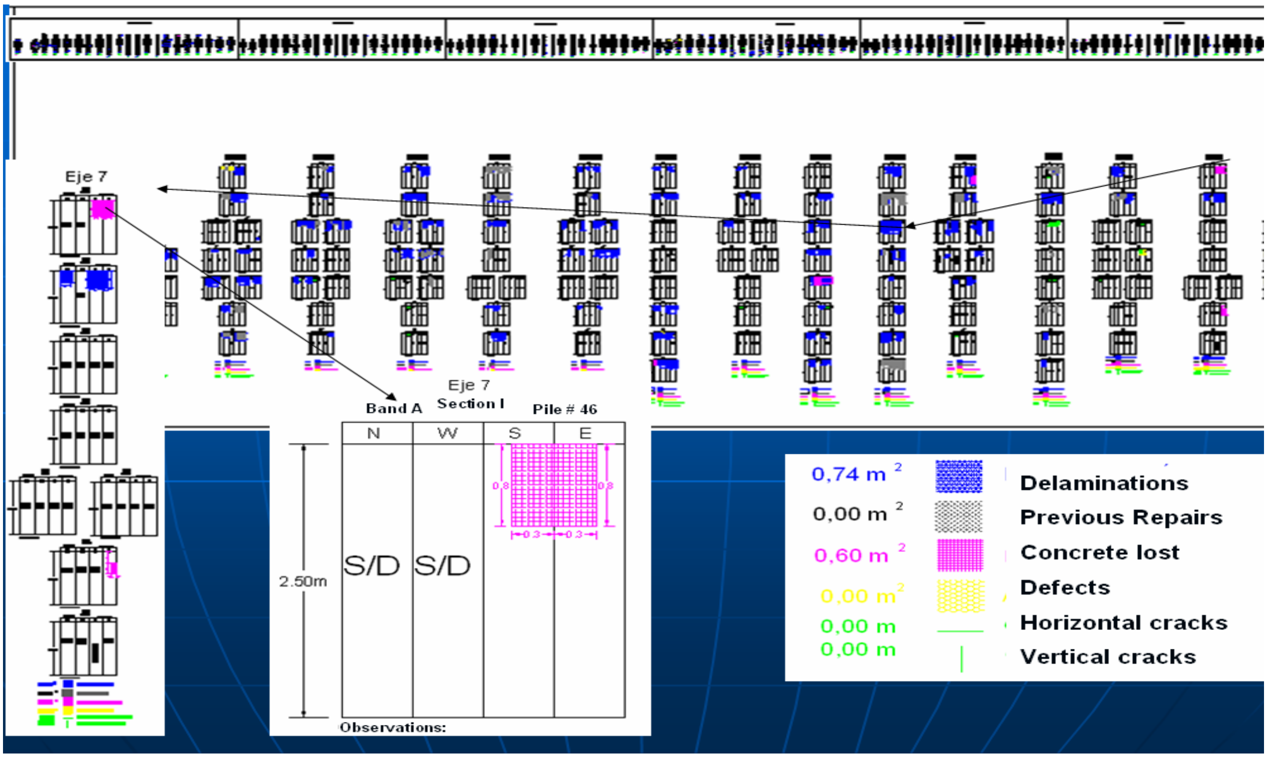

Figure 3 - Data survey example

3.2 Compressive Strength

Figure 4 represents, in a statistical cumulative percentage, the strength resistance, for both piers. South Pier presented the highest compressive strength of its piles. The acceptance criterion was $f'_{c\text{-estimated}} \geq f'_{c\text{-design}}$, where $f'_{c\text{-estimated}}$ is the value obtained from extracted cores and $f'_{c\text{-design}}$ is the design compressive strength based on plan specifications. The average $f'_{c\text{-estimated}}$ was 39.7±8 MPa (value written as average±standard deviation) for South Pier and 35.6±9 for North Pier. On the other hand, $f'_{c\text{-design}}$ was 25 MPa for North Pier and 35 MPa for South Pier. Based on these estimates, the acceptance criterion was met (only two data points for North Pier were below 25 MPa and six data points were below 35 MPa for South Pier, see Figure 4). The variation coefficient provides a way to estimate the homogeneity of the concrete: average 26% for North Pier and average 23 % for South Pier is indicative of significant variations in the concrete fabrication procedure (mixture materials, water/cement ratio, concrete placement and compaction, curing, etc.).

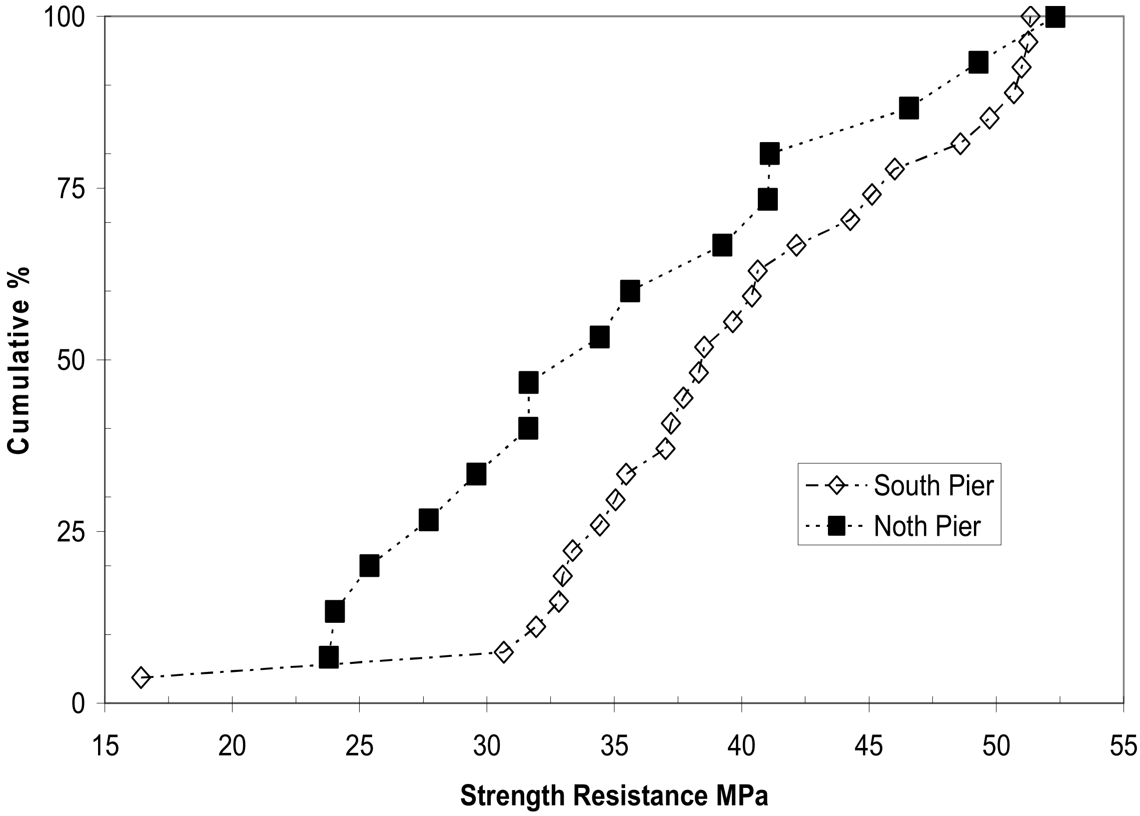

Figure 4 - Strength Resistance for the North and South Piers.

Even though, the concrete compressive strength values complied with specifications ($f'_{c\text{-estimated}} \geq f'_{c\text{-design}}$,), the variability between data from piles at different locations in same pier is indicative of little quality control during construction of theses two piers, as it is observed from Figure 4 results.

3.3 Chloride concentration

Chloride concentration levels (at three different concrete depths: 0-2.5, 2.5-5.0, and 5.0-7.5 cm) on South Pier were lower than those of North Pier (at similar depths). As seen in Figure 5, 50% of the data points at 0-2.5 mm depth were higher than the steel corrosion initiation threshold (0.4% per cement weight) [4]; only 25% of the data points at 2.5-5.0 cm exceeded the threshold for steel corrosion initiation (0.4% by cement weight); and no data points exceeded this threshold for concrete depths between 5.0-7.5 mm. The chloride concentration data obtained implies that the concrete service life was in the active corrosion or propagation stage. Even though chlorides level measurements were not carried out on the beams (due to the difficulty in obtaining samples), the fairly visible and widespread spalling of the concrete indicated that chloride levels on the beams would also have exceeded the threshold value.

3.4 Carbonation Front

Apparently the high moisture of the closed environment where the piles are located, allowed the saturation of the concrete pores, thus preventing the penetration of CO_2 or SO_2 gas. Carbonation front reached only 0.9±0.1 cm on South Pier and 25 m on North pier, far from the more superficial steel reinforcement (3.5 cm) measured at South Pier and North Pier (4.2 cm). As can be seen in Figure 6, even though North Pier had a larger front than South Pier, both have not reached the reinforcement depth, so the carbonation did not contributed to the corrosion process.

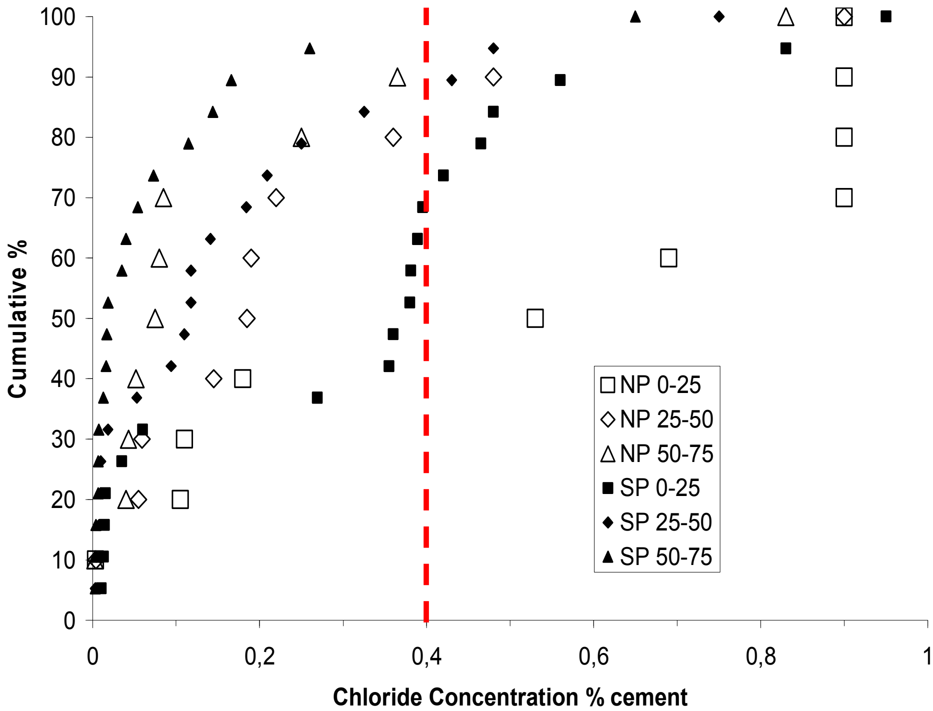

Figure 5 – Chloride Concentration for both Piers at three depths.

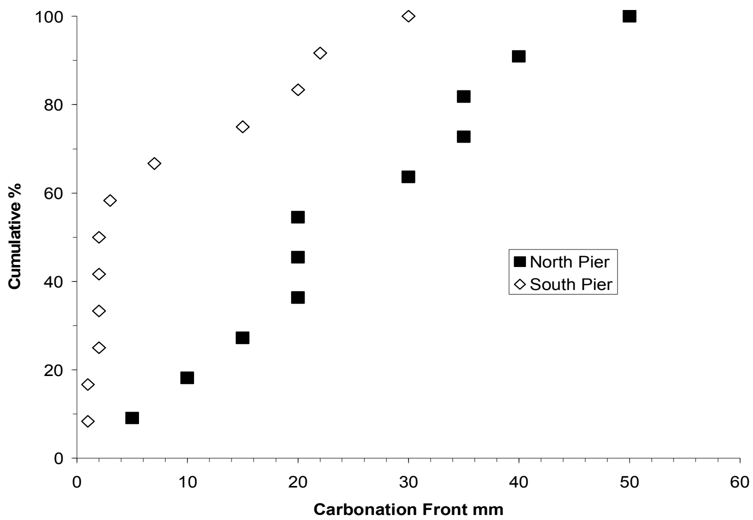

Figure 6 - Carbonation Front for the North and South Piers.

3.5 Sclerometric Index

Figure 7 represents, in a cumulative percentage, the sclerometric index results obtained from both piers. The Rebound Index Number (RIN) was useful to determine the homogeneity of the concrete. The average value obtained from this test were for North Pier 46 RIN and for South Pier was 45 RIN (just as a reference, 1 MPa approximately 1.04 RIN). The experimental data variation coefficient for North Pier was larger (23 %) than the obtained for South Pier (7%) in concordance with the strength resistance as can be seen in Figure 4. This behaviour may be due to higher carbonation depths of North Pier, which in turn increase its apparent strength by filling the surface pores by calcium carbonate by products.

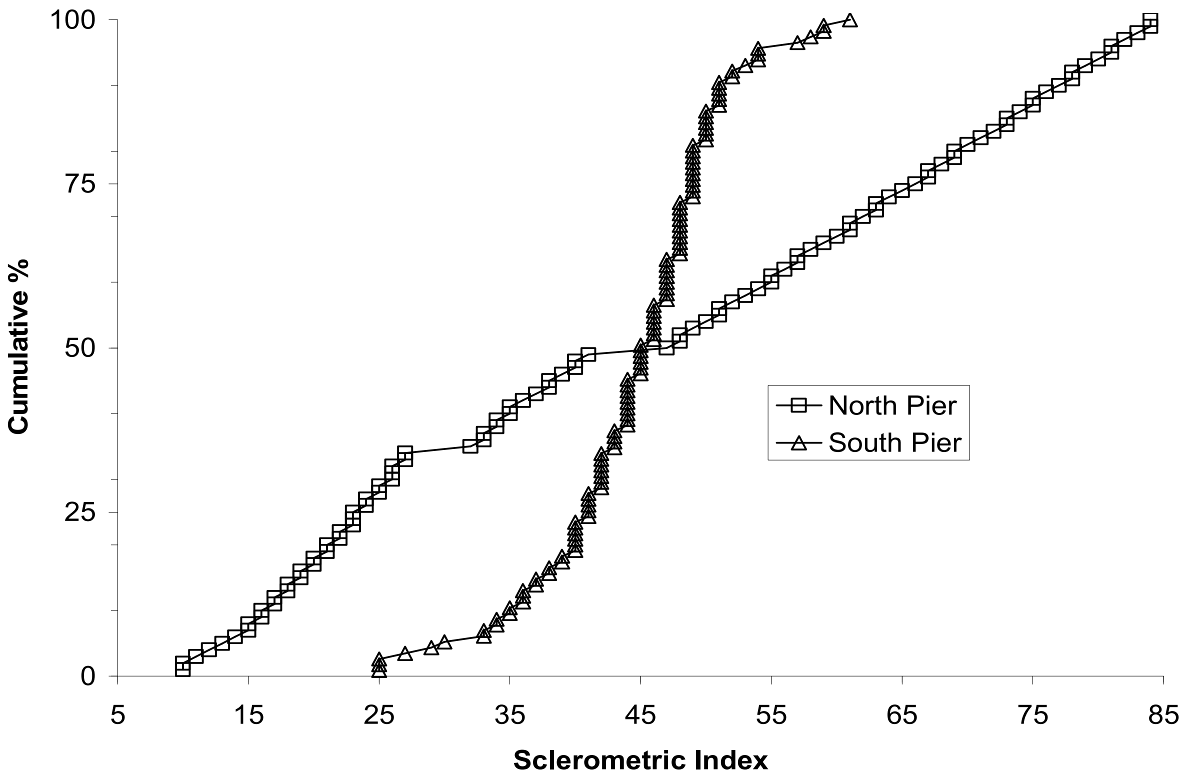

Figure 7 – Sclerometric Index for both Piers.

3.6 Half cell potential

Experimental half cell potential measurements are presented in Figure 8 Results are presented as cumulative percentage. The values were presented using different symbols for easy location: pile face (North, ♦NF; South, ■SF East, ▲EF; and West ●WF;), and height above high tide, AHT, (white symbol for 2 m AHT for North Pier and 1.7 and 1.3 m AHT for South Pier; light grey symbol for 1.2 m AHT for both Piers; dark grey symbol for 0.8 m AHT for both Piers; and black symbol for 0.4 m AHT for both Piers).

From this figure 40% of the measurements for North Pier and 25% for South Pier were typically in the range of -350 mV to -450 mV, mostly in the aerial zone where no spalling had occurred. According to ASTM C876-91:1999 [5], there is a 90% probability that corrosion had started in these zones, based on the half potential measured. It was also observed pitting corrosion of the steel in piles, resulting in a cross section reduction in many cases over 15%, hereby affecting the load carrying capacity of the piles and consequently the stability of the entire structure.

The values obtained on North and South Piers revealed active corrosion potential located at approximately ±40 cm from high tidal level. In this area, about 60% of readings were more negative than -250 mV, and among them, over 25% were more negative than -350 mV, which means that there is a likelihood of 90% of active corrosion at this zone. Eventually, some piles presented more negative potential values than -350 mV until -550 mV for North Pier and -450 mV for South Pier. Well defined potential differences were observed as a function of height AHT. As observed from Figure 8, pile face orientation did not show any half cell potential difference in any of the Piers. On the other hand, results obtained show more negative reinforcement potential values at lower pile heights, in accordance with chloride concentrations obtained at similar position AHT.

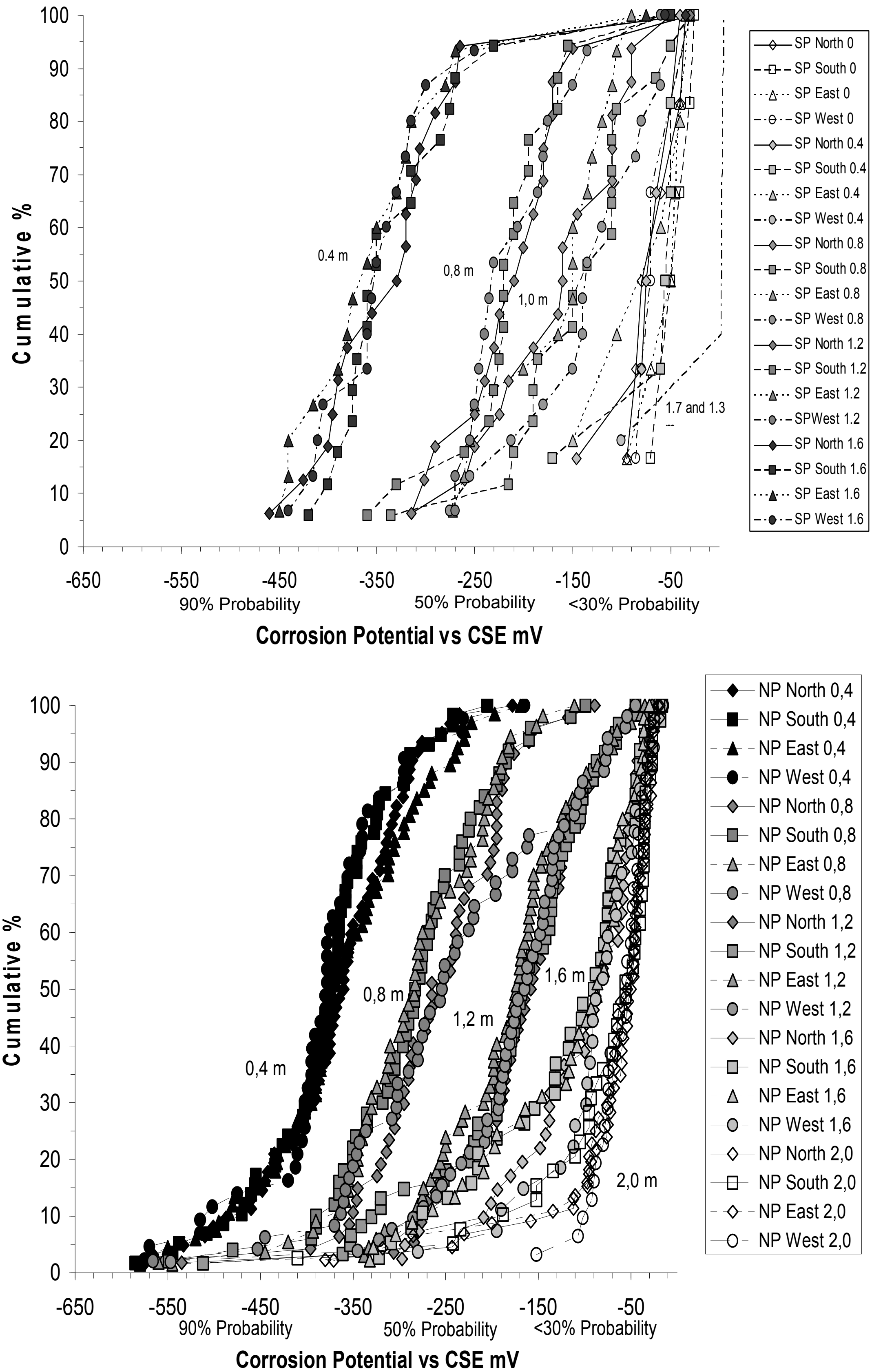

Figure 8 – Rebar half cell potential for North and South Piers, respectively, as a function of height AHT.

3.7 Concrete cover depth and reinforcement diameter

The main purpose of this test was to find the concrete cover depth to which the reinforcement was located, as well as their diameter. Noticeable variation registered in the measured steel reinforcement concrete cover depth: between 2.5 to 13.2 cm (8.4±3.1 cm) for North Pier, and 3.5 to 9.5 cm (6.6±1.1 cm) for South Pier. Figure 9 shows all measurements obtained, implying once again inconsistencies during the construction process and/or during the previous repairs.

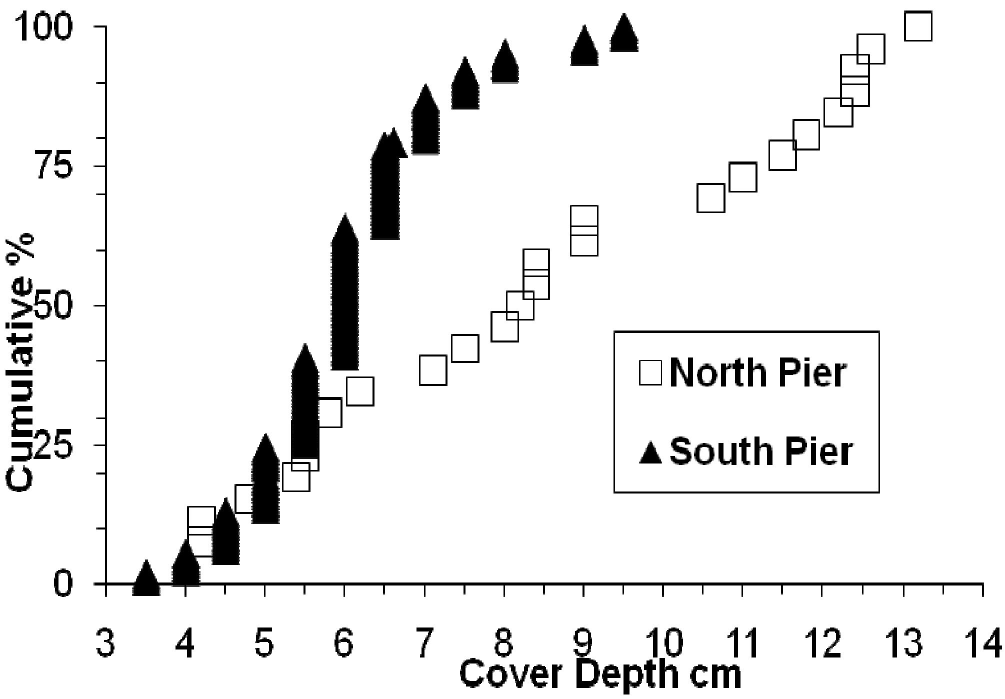

Figure 9 – Reinforcement Cover Depth for both Piers.

3.8 Rehabilitation project

A general rehabilitation procedure for both piers consisted of removing damaged concrete and clean and/or rehabilitate reinforcement steel. Even though each pile got specific repairing instructions according to its particular condition, the general procedure was as follows:

1. Detect and mark the steel rebar position. Explore by tapping with a hammer and delimit the damaged areas using a disk cutting machine. The cut must be done keeping the disc always perpendicular to the surface and penetrating depth of at least 1.5-2 cm or the minimum necessary to avoid damaging the existing steel. Only in the upper horizontal cuts of areas to repair, the cut should be from 30 to 45 ° inclination downward, in order to ensure that the upper parts of the area is completely fill. (Figure 10).

2. Remove the concrete in the demarcated area using pneumatic hammers of maximum 20 pounds to avoid weakening the healthy concrete or hurting the steel reinforcement. It should be reviewed that the equipment used does not have any damage which could mean an oil spillage which would pollute the area and could endanger adherence with the repairing materials. The clearance should be to a depth of at least 2 cm behind the corroded steel so it can be cleaned up in its entirety. If beyond the borders of the demarcated area corrosion is found, the removal of concrete should continue at least 10

cm after finding healthy steel. The edges of this new clearance also must be shaped with disk cutting machine as described above. It is very important to seek precautions during this work as regards a correct underpinning of the pile. The underpinning should not be withdrawn until the repair is completed and the repairing material has reached at least 80% of its strength. It is also recommended to switch both transverse and longitudinal axes to do not lead to the formation of a weak area in the structure.

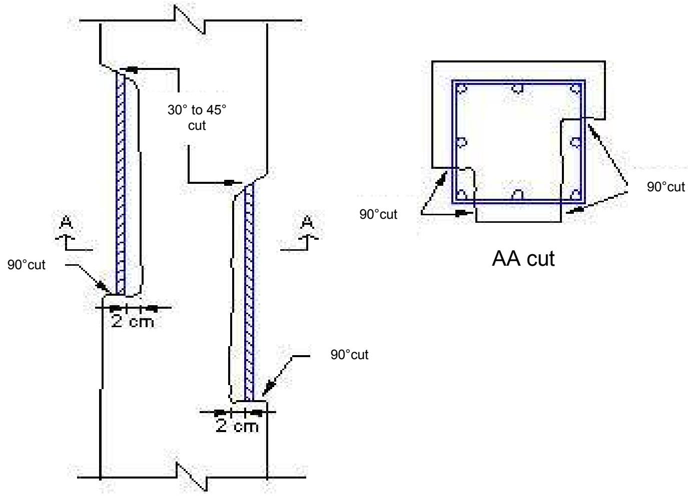

Figure 10 – Rehabilitation procedure for both Piers.

3. Completely clean steel reinforcement using free chloride water jet at high pressure (hydroblast), to totally remove the oxide across the steel surface.

4. Once the steel is fully clean, it should be checked if there is loss of the cross section, both longitudinal and transversal steel, where more than 15% of area loss (not the diameter) is found, the structure must be strengthened by adding new steel of the same diameter and the same properties to the existing. In almost total steel loss cases, the best way is to replace completely the damaged section. To replace the damaged steel, mechanical connectors should be used since that prevents almost entirely the excessive removal of concrete

5. Clean the entire surface of the concrete also through hydroblast, to leave it completely clean, i.e. free of dust, damaged, loose or weak material, fat, or any other contaminant.

6. Use metallic molds with a suitable demoulding product to recover the shoring. The shoring shall be sufficiently tight to prevent the departure of the material (it is convenient to seal the edges with rubber strips or similar material). Ensure that the shoring system allows placing a minimum concrete thickness of 6 cm. To expedite the work, it is convenient to pre-manufacturer parts of the shoring to simply assemble them on the site.

7. Add a marine grout with 40% by weight of 1/2 inch maximum size gravel and strain using a tremie type system coupled with a 2-inch diameter hose. The filling with the hose must be carried out from the bottom and going up gradually as it fills, thus the displacement of the water without the material dispersion is achieved. A continuous vibration of the tremie system to prevent honeycombing is needed.

8. For greater than 5 cm thickness, add 3/8 inch gravel in a proportion of 30% in weight to the grout. To replace the transverse steel (Stirrups) that have lost more than 15% of the area of its section (not the diameter), place strips of carbon fiber around the pile. The fiber

must be adhered to concrete with epoxy resin. The width of the fiber strips shall be in accordance with the following: for a stirrup with section loss of over 50% of its section, place a strip made up of 3 superimposed layers of 10 cm wide whose center is in the same position as the original steel strap. For a stirrup with loss of section less than 50% of his section, place a strip made up of 3 layers of 5 cm wide whose center is in the same position as the original steel strap. The overlap between the initial and end extremes of fiber strips must be at least 15 cm. Overlapping fiber strips must not match the position between consecutive strips of fiber; these must be distributed in the faces of the element. Carbon fiber strips should not be placed until after 21 days after casting the repairing concrete.

9. Apply a cathodic protection system. The necessary connections to the reinforcing steel, as well as application procedures must be made according to the specifications of the supplier.

The criteria used to classify each pile to be repaired was the following: a repair Type 1 was followed when the concrete's damage was over 0.8 m long, the steel had lost more than 15% of the sectional area, and all 4 faces were damaged; Type 2, when the damage was over 0.8 m long but the steel remained within 15% of section loss; when there was not any damage (either concrete nor steel), the classification was Type 3. This helped the preparation of a list with the identification of the pile, the number of faces which should be repaired, the dimensions of such repairs, and its classification. The classification of the repairing processes implied the following:

- Type 1: The concrete should be delimited and demolished, steel must be replaced, concrete should be replaced, and a cathodic protection system should be installed.
- Type 2: The concrete should be delimited and demolished, steel must be cleaned on all faces, concrete should be replaced, and a cathodic protection system should be installed.
- Type 3: A cathodic protection system should be installed.

As a result of the used methodology a rehabilitation project was generated and it is summarized below:

Piles classified as Type 1 (412 piles for North Pier and 826 piles for South Pier) should be repaired using a concrete with micro-silica and super-fluidificants to be capable of supporting a tremie type system. Steel should be replaced by using mechanical connectors and to warranty the adhesion of new to old concrete, the area should be moistened with free chlorides water for at least 4 h previous to the concrete positioning, additionally, it should be applied a galvanic cathodic protection system.

Piles classified as Type 2 (651 piles for North Pier and 2014 piles for South Pier) should be repaired using an self-levelling expansive mortar, with aggregates of 12 mm up to 30 % maximum weight of the mortar and the area to repair should moistened with chlorides-free water for at least 4 h before concrete positioning. In addition it should be installed a galvanic cathodic protection system to the boundary of patch repairs to protect against macro cell corrosion, thereby mitigating corrosion of steel around patch repairs.

Piles classified as Type 3 (1601 piles for North Pier and 1337 piles for South Pier) should be installed a cathodic protection system as a sustainable long term remedial option, the sacrificial anodes would be installed at non-repaired areas which have highly negative potentials so to prevent further corrosion of the steel reinforcement. The installation and use of a cathodic protection system was seen to be the most economical alternative from a life cycle perspective.

Figure 11 shows the differences on the kind of rehabilitation required on both piers, this means that the structure's characteristics play an important role on its durability. Even though South Pier was built 20 years after North Pier, the first had a higher chlorides concentration and a lower concrete cover which could be a determining factor on the corrosion process. South Pier had a more generalized corrosion, 68% of the piles must be repaired, compared with the 39% of the North Pier.

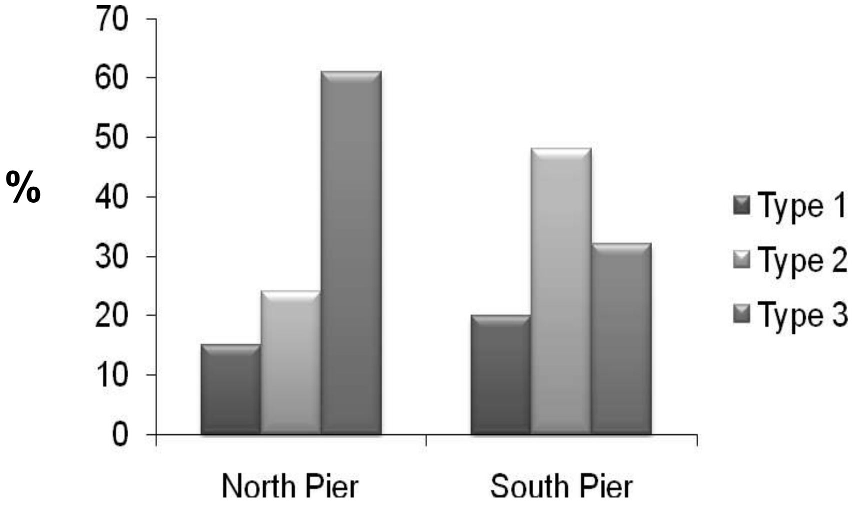

Figure 11 – Comparision of rehabilitation type between North and South Piers.

4. CONCLUSIONS

Integrity assessment and performance analysis of marine reinforced concrete structures, such as Piers, require a multidisciplinary approach to determine corrosion degradation and how to stop it or prevent it.

The materials mechanical characterization of the concrete (compressive strength, rebound number index) complied with the design project for both piers. From the chemical characterization, concrete did not show carbonation chemical attack; only chloride concentration at the reinforcement level of the piles is higher than critical threshold of 0.4% at -0.2 – 0.5 m AHT.

Reinforcement did show corrosion activation in piles at -0.2 to 0.5 m AHT in both piers.

From the detailed damage survey performed, 15% of North Pier piles and 20% of South Pier piles were classified as Type 1 rehabilitation. Thus replacement of steel, concrete replacement, and galvanic cathodic protection system was considered for the repair of such piles.

A total of 24% for North Pier piles and 48% for South Pier piles were within classification Type 2 rehabilitation, so it was required to repair them by concrete replacement and a galvanic cathodic protection system.

61% of North Pier piles and 32 % of South Pier piles were within classification Type 3, where only a galvanic cathodic protection system was sugested.

Finally, a stochastic durability model will be studied with the data obtained from both piers.

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