ELECTRO TECH CP©

Manual Reference:	IR302915-11	**Revision**	A	**Date of Issue:**	06/16/11

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Technical Approval: P A Noyce

Distribution:

Abstract:

Revision	Comment	Date
A	First Issue	06/16/11

CORROSION INSPECTION
BIG BEND COAL DOCK

Tampa Electric (TECO)
Big Bend Road
Tampa, Florida

Manual Reference
IR302915-11-Rev A

electrotechCP

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Executive Summary

The investigation of Teco coal dock has identified a number of key requirements that need to be addressed as soon as possible.

As you will see within this report we have broken down the dock into key components and have detailed the work required on a phased approach based on our findings.

The corrosion inspection identified active areas of corrosion on over 50% of the areas tested which when combined with the delamination survey accounts for a rapid rate of deterioration, which was confirmed by the corrosion models carried out. The models showed that the threshold of corrosion will be at maximum within a period of time of less than two years. The consequences of not addressing this will results in significant additional cost.

The investigation also found significant damage to the dolphins so bad that one of the "H" piles had completely corroded. In addition to the dolphins the supporting piles which are prestressed were also showing signs of defects where a zero tolerance is permitted from chloride diffusion.

Other areas of concern was the general condition of the transverse beams as these play a major role in transferring the deck load into the supporting piles. In nearly every location inspected large horizontal cracks were evident on both the underside and the exterior face. These beams require a long term solution and we would recommend that these locations are treated with a galvanic cathodic protection system in the form of a lifejacket.

The main deck and bridges also show massive areas of delamination to the top slab which was attributable to the top reinforcing steel in the wearing course. Our concern here is that all of the structures on the deck are fixed through this slab and these structures could be compromised as a result of these delaminations.

As the main deck also consist of access walkways around the dock which do not have a wearing course significant damage was found on the top and bottom reinforcing steel and as a result of this we would recommend a long term corrosion control system in the use of impressed current cathodic protection.

All expansion joints were defective and offering little to no protection to contaminants reaching the prestressed deck. We would recommend all expansion joints be replaced with a suitable movement joint capable of surviving in the environment they are to be used in.

In summary as the dock has been defined as a Class I structure it is essential that repair work be carried out in less than one year to avoid health and safety issues and additional cost.

Section 01

Introduction

Section 01

Introduction

History

The Big Bend Station Coal unloading dock was completed in 1970 by Stone & Webster Engineering Corporation, Boston, Massachusetts. Construction consists of 642 number 18" square pre-stressed piles supporting a pre-stressed slab with a 6" topping slab. There are two access bridges located in bay 3 (North Bridge) and bay 9 (south bridge).

The dock is separated into 9 number bays, 2 of which are reinforced hurricane tie-down bays for the peco and dravo un-loaders. On the West side of the dock (channel side) there are 9 number steel dolphins to protect the dock from damage by incoming ships.

Additions were made to the dock in 1980 with a third access bridge (bay 7) which has been identified as the middle bridge.

Big Bend Power Station has four coal-fired units with a combined output of more than 1,700 megawatts. The first unit began service in 1970; the second and third generating units were added in 1973 and 1976, respectively; and Unit Four was added in 1985. A natural gas- and fuel oil-fired peaking unit was installed in 2009 to provide additional power during periods of peak demand.

Structure Type/Classification

The structure is named as a coal offloading facility and is defined as a marine dock. The dock consist of a deck slab, access bridges, support structure and driven pile supports.

The structure type classification is built up from the following tables:

	< 1 Mile	1 – 10 Miles	> 10 Miles
Coastal	Yes		
Temperature	> 70 F (75% of the Year)	> 70F (50 – 75% of the Year)	> 70F (less than 50% of the Year)
	Yes		

Table 1 – Principles of Location.

Function	**Industrial**	**Infrastructure**	**Commercial**
	Yes		
Material	**Concrete**	**Masonry**	**Steel**
	Yes		
Technological	**Monolithic**	**Prefabrication**	**Masonry**
	Yes		
Size	**Super Structures**	**High Rise**	**Long Span/Single Story**
	Yes		
Important factors	**Critical Plant**	**Marine**	**Contaminants**
	Yes	Yes	Yes

Table 2 – Principles of Type of Structure.

The previous two tables allow us to define the type, location, important factors and exposure which enable a classification to be formed as follows:

Definition	**Description**	**Rating**	**Big Bend Coal Dock**
Class I	High Risk Is defined as a structure with high importance in operations. A structure where high contaminants are present and located in a tropical type environment.	≥18	Yes
Class II	Medium Risk Typical structure where certain principles could be high and individual factors need to be assessed	7 to 17	
Class III	Low Risk All conditions and factors indicate a structure of low importance and in a non aggressive environment.	≤6	

Table 3 – Classification of Structure.

Scope of Work

Electro Tech CP LLC carried out a corrosion condition assessment between May 16 and May 25, 2011 on all facets of the TECO dock where concrete elements were accessible and viably able to be tested. Electro Tech CP LLC (ETCP) was assisted in this work by the Florida West office of Structural (Structural Preservation Systems). Dive-Tech was also engaged to carry out the dive inspection of the dock support piles & dolphins.

The scope of work included a full visual inspection of the above water concrete elements of the dock and a select number of submerged piles and dolphins.

The following is the scope of non destructive testing carried out:

- Half-Cell Potentials

  Were carried out to the majority of the deck slab of the dock, access bridges and the inboard face of the dock

- Corrosion Rate Tests

  Were carried out at select locations of the dock (slab, outboard, inboard) and the access bridge deck slabs.

- Hammer survey

  Was carried out to the deck slab, the outboard and inboard portions of the dock.

- Cover Survey

  Was carried out at the locations of the corrosion rate tests as supporting information to the extent of corrosion.

- Schmidt Hammer Survey

  Were carried out at select locations of the dock (slab, outboard, inboard) and the access bridge deck slabs.

In addition to the non destructive test the following destructive test were also carried out

- Concrete Core Samples for Petrographic Testing

  Two locations from the overhang of the deck were selected for petrographic analysis to be performed by Highbridge Materials Consulting.

- Chloride dust samples

  Were taken in five locations throughout the structure for laboratory analysis by ETCP. After extraction of the dust in situ carbonation testing was performed.

Basis of Investigation

All procedures carried out under the direction of ETCP are used to provide the client with a corrosion condition assessment. This is the first basis of the investigation. From the test procedures used, the results are mathematically calculated to determine and assess the durability of the structure.

The resulting assessment provides a durability analysis, which is a projection of how the structure will endure over time based on the current conditions, contamination factors, future diffusion of contaminants, and loss of steel section (corrosion activity).

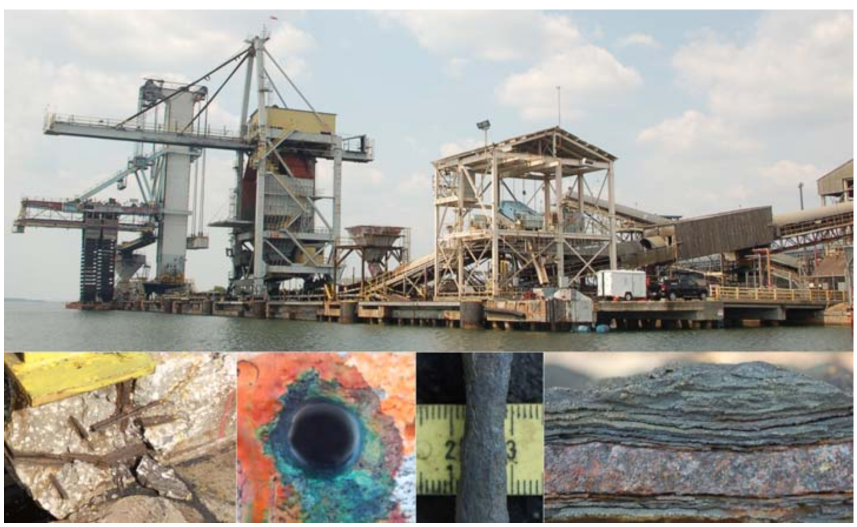
The second basis of the investigation is to use the information gained to quantify and classify damage types. After a determination of current damages and conditions, an appropriate repair and corrosion mitigation strategy can be formulated. Corrosion related deterioration is complex as are the associate repairs.

Finally, the information gathered will be utilized to determine the best repair type with the required service life for the dock.

Dependant upon the clients' expectations of service life, the repair types can be assessed for longevity. Cost benefit analysis can be carried out with the client to assist in choosing the most effective and durable solution based on the findings of the investigation.

Section 02

Test Methods



Section 02

Test Methods

Visual Inspection

A visual inspection is aimed at identifying concrete deterioration in the form of cracking, delaminations, spalls or other distress. All records of the visual inspection is documented in the form of drawings and detailed within Section 3 of this document.

Hammer Survey

A standard hammer survey consists of using a 2lb hammer which is used to strike the surface of the concrete. Hollow or dull tones that can be heard and recognized by the inspector indicate the existence of relatively shallow delaminations in the structure.

Hammer sounding can also be useful for locating areas of severe damage to the cement matrix that correspond to reduced strength.

Schmidt Hammer Survey

In 1948 Ernst Schmidt invented a device which made nondestructive compressive strength testing feasible. A Schmidt hammer is a device to measure the elastic properties or strength of concrete or rock. Today Schmidt hammers are in use throughout the world for estimating strength of concrete. The European standard for Schmidt hammer testing is EN 12504-2.

The test hammer hits the concrete with a spring-driven pin at a defined energy, and then measures the rebound (in rebound units). Its rebound is dependent on the hardness of the concrete and is measured by test equipment. When conducting the test the hammer should be held perpendicular to the surface which in turn should be flat and smooth. Note that the Schmidt hammer does not work well for small samples and will make marks. By reference to the conversion tables, the rebound value can be used to determine the compressive strength. Schmidt hammers are available from their original manufacturers in several different energy ranges.

Although, rebound hammer provides a quick inexpensive means of checking the uniformity of concrete, it has limitations. The test is also sensitive to local variation in the sample. To minimize this it is recommended to take a selection of readings and take an average value. In an assessment of the influence of internal rock moisture content on Schmidt hammer readings, rebound (R) values are found to decrease with increasing moisture content. Other influences are type of coarse aggregate, cement, mold, and carbonation of the concrete surface.

Each hammer varies considerably in performance and needs calibration for use on concrete made with the aggregates from specific sources. The test can be conducted horizontally, vertically or at intermediate angles. At each angle the rebound number will be different for the same concrete and will require a separate calibration or correction chart .

Investigations have shown that there is a general correlation between compressive strength of concrete and rebound number; however, there is a wide degree of disagreement among various research workers regarding the accuracy of estimation of strength from rebound readings. The variation of strength of a properly calibrated hammer may lie between +/- 15% and +/- 20%.

Potential Survey

Since the early 1980s, the use of half-cell potential measurements for the identification of reinforcing steel corrosion in concrete or the assessment of the condition of existing concrete structures has been frequently employed. This method only provides an indication of the relative probability of corrosion activity through measurements of the potential differences between a standard portable half-cell and the reinforcing steel. The method is unable to determine corrosion rates or the degree of corrosion that has occurred.

Factors such as chloride and oxygen concentrations, temperature and moisture content can affect half-cell potentials over a certain range. The ASTM standard C876[1] provides interpretative guidelines for the evaluation of corrosion probability for reinforcing steel in concrete. According to this standard there is a 10% probability of reinforcement corrosion if the half-cell potentials are more positive than -200 mV; an a 90% probability of reinforcement corrosion if the half-cell potentials are less than -350 mV with reference to a copper/copper sulphate half-cell ($Cu/CuSO_4$). See table 1.

Corrosion Potential (Volts vs. $Cu/CuSO_4$)	Probability of Corrosion
>-0.200	<10 %
-0.200 to -0.350	Uncertain
<-0.350	**>90 %**

Table 4 - Likelihood of corrosion damage as a function of the corrosion potential.

The predicted corrosion conditions based on the guidelines can be quite different from the actual corrosion conditions. Severe discrepancies between the assessment of the corrosion state using the ASTM guidelines and the actual deterioration that has been observed at the time of repairs have resulted in a different set of interpretive guidelines.

Corrosion products have a volume several times greater than the volume of steel from which they are derived. The build-up of corrosion products on the surface of the steel creates tensile forces in the concrete. When the embedded steel becomes corroded, the production of a voluminous corrosion product induces internal stresses in the concrete surrounding the reinforcement. When the corrosion internal tensile stresses exceed the concrete tensile strength, cracking and spalling of the concrete cover will occur.

Half-cell potentials are very sensitive to the ambient environment, especially the oxygen concentration at the interface between the reinforcing steel and the concrete. Usually, a decrease in O_2 can drive the half-cell potential significantly towards more negative values. Completely water-saturated concrete can lead to O_2 starvation, resulting in potential values more negative by up to 200 mV[2]

If the concrete cover is saturated by water, O_2 is not freely available at the metal surface because O_2 is not very soluble in aqueous solutions. The rate of reinforcement corrosion is controlled by the rate of arrival of oxygen at the metal surface. Because of the diminution in O_2 concentration the corrosion potential will shift to more negative values. These values can be much more negative than –350 mV vs. $Cu/CuSO_4$ in many conditions and can lead to an improper prediction of the state of corrosion even though the reinforcement is still in good condition.

Corrosion Rates

Measurements of the rate of corrosion are usually made using sensors on the surface of the concrete. However, sensors are also available that can be attached to the rebar and can be used to measure the corrosion rate in the concrete environment.

The most common surface method for determining Corrosion Rates is by utilizing a method known as Linear Polarization Resistance (LPR). The LPR measurement is where a small (20 mV) potential difference is applied between the steel and a secondary electrode on the surface which results in a small current flow. This is then proportional to the inverse of the polarization resistance and hence is directly proportional to the corrosion rate.

[1] ASTM C876-09 (2009): Standard Test Method for Half-cell Potentials of Reinforcement in Concrete, ASTM, Philadelphia
2 Vassie P. R., TRRL Application Guide 9, 1991, pp 30

As part of this test the steel potential is also measured. The combination of these tests allow for an assessment of the most active areas of corrosion to be identified. With LPR testing and an understanding of the structure, the location of underlying factors that may be directly affecting the corrosion of the steel may be more easily identified. Early detection of these factors can guide the repair design process for the most effectual treatments.

The loss of section is determined from measuring the amount of steel dissolving and forming oxide (rust). This is carried out by determining the electric current generated at the anodic reaction.

Anodic reaction $Fe - Fe^{2+} + 2^{e-}$

This is then consumed at the cathodic reaction:

Cathodic reaction $H_2O + O_2 + 2^{e-} - 2OH^-$

You then convert the current flow using Faraday's Law into metal loss

Faraday's Law $m = Mit/zF$

m = Mass of Steel

M = Atomic weight of metal (56g for Fe)

i = Current

t = Time

z = Ionic charge (2 for $Fe- Fe^{2+} + 2^{e-}$)

F = 96500 As

This equates to 1µA cm^{-2} = 11.6µm steel section loss per year or 0.155 in^{-2} = 0.45704mpy steel section loss per year.

Rate of Corrosion	Corrosion Current Density, (icorr) µA/cm²	Corrosion penetration, m/yr
High	10-100	100-1000
Medium	1-10	10-100
Low	0.1-1	1-10
Passive	<0.1	<1

Table 5 – Corrosion Rates of Steel in Concrete

icorr (µA/cm²)	icorr (µA/in²)	Severity of Damage
<0.2	<0.031	No corrosion damage expected
0.2-1.0	0.031 to 0.155	Corrosion damage possible in 10 to 15 years
1.0-10	0.155 to 1.55	Corrosion damage expected in 2 to 10 years
>10	>1.55	Corrosion damage expected in 2 years or less

Table 6 –Corrosion Rate and Remaining Service Life

The following table is based on an average of 3 times the volume of oxide.

icorr (μA/in²)	Metal Loss (mpy)	Section Loss
<0.0155	0.04334	Section loss 0.13mpy Rust Growth
0.0775	0.22458	Section loss 0.67mpy Rust Growth
0.155	0.45704	Section loss 1.37mpy Rust Growth
1.55	4.5704	Section loss 13.7mpy Rust Growth

Table 7 – Typical Section Loss

NOTE: The expansive oxide growth between 0.394mil (10μm) and 3.94mil (100μm) (0.01 to 0.1mm) will cause cracking.

Cover Survey

The cover survey is use to determine the depth of the reinforcing steel and aid in the interpretation of the corrosion behavior.

Concrete cover to the reinforcing steel is used in conjunction with the survey of corrosion rates to assist in the interpretation of the potential readings and to improve the assessment of the likelihood of corrosion.

The survey is carried out on a grid basis following the pattern of the corrosion rates measurements. The following table is used to define low, average and good cover.

Concrete cover is the distance from the concrete surface to the closest reinforcing steel bar embedded within the concrete. First of all minimum cover to reinforcement is specified to control access of deleterious agents (carbonation and chlorides) to the reinforcement.

Additionally the cover controls the rate of access of the 'fuels' for corrosion, moisture and oxygen, to anodic and cathodic sites. Finally the concrete cover also affects the structural effects of corrosion once active, as seen in the following table.

Cover to bar diameter ratio	likely type of damage
1	local spalling, cracks from bar surface
2	larger spalls; possible delamination
3	delamination between bars

Table 8 – Guide for likelihood of damages depending on cover depth

Petrographic Testing

The testing is performed using thin section petrography in accordance with ASTM C 856. The analysis will include an identification of constituents, assessment of overall concrete quality, and investigation of potential causes of distress. Features related to the original placement such as approximate water/cement ratio, finish, consolidation, bleed, etc. are addressed where possible.

Also included is a characterization of the occurrence and distribution of any secondary effects such as carbonation, aggregate reaction, sulfate attack and their relationship to any visible distress.

Water-soluble chloride determination will be carried out at multiple depths in the concrete core in accordance with ASTM C 1218. Wafers are sliced at 1", 2" and 3" and homogenized to obtain the chloride value.

Acid-soluble sulfate determination will be carried out at multiple depths in the concrete core in general accordance with ASTM C 114 modified to accommodate concrete as opposed to hydraulic cement. Wafers are sliced at each requested depth 1", 2" and 3" and homogenized to obtain the sulfate value.

Dust Samples

A concrete dust sample is collected using a drill and dust collecting pan with a clamp and anchor. After mixing and quartering, the dust sample is weighed and added to one of the prefilled acid extraction bottles. The sample is mixed into a distinct amount of extraction liquid and shaken for five minutes. The extraction liquid removes disturbing ions, such as sulfide ions, and extracts the chloride ions in the sample.

A calibrated electrode is submerged into the solution to determine the amount of chloride ion, which is expressed as percentage of concrete mass.

The accuracy of the Rapid Chloride Test (RCT) results compared with the known amount of chlorides is as good as with the AASHTO T 260 potentiometric titration method. The average deviation of the RCT results from the known amount of chlorides is within ± 4 %.

Carbonation Testing

The natural alkalinity of cement paste in concrete results in a protective oxide coating on steel reinforcement that prevents the steel from rusting. When carbon dioxide (CO_2) in the air penetrates into concrete, it reacts with the calcium hydroxide ($CaOH_2$) in the cement paste producing calcium carbonate ($CaCO_3$). This reaction is called carbonation, and it causes the alkalinity of the paste to decrease, that is, the pH decreases below its normal value of about 13. When the pH drops below 9, the protective oxide coating is destroyed and, in the presence of moisture and oxygen, the steel will corrode. Thus measurement of the depth of carbonation is an essential step for a corrosion evaluation of a reinforced concrete structure.

To measure the pH of the cement paste, the drilled holes for the dust sample are sprayed with the pH indicator, and allowed to dry. The approximate pH of the paste is indicated by colors as illustrated below.

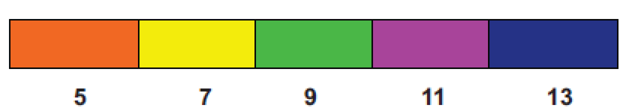
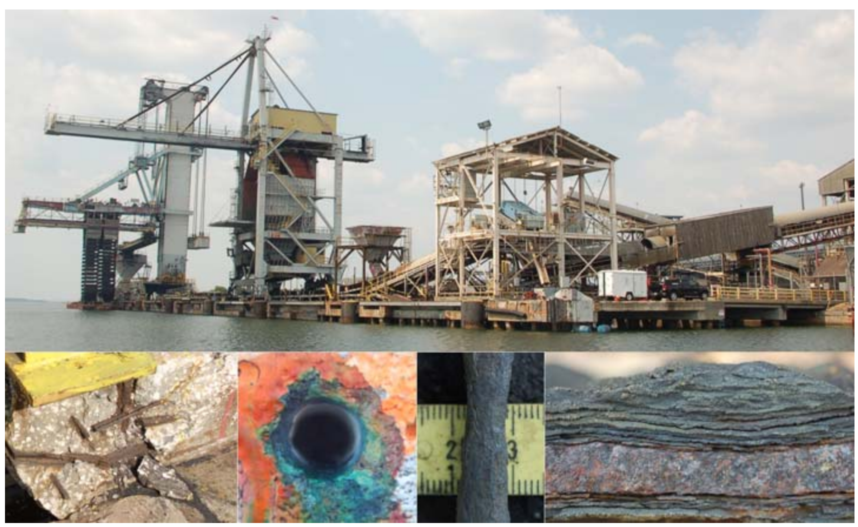


Figure 1 – Rainbow Indicator for Carbonation

Section 03

Condition Assessment



SECTION 3

Condition Assessment

Covered in this section

- Visual Inspection
- Hammer Survey
- Schmidt Hammer Survey

North Bridge

The North Bridge is located at the northern portion of Bay 3, a hurricane tie down bay. The bridge is 20 feet in width and ties into the dock structure between the 14th and 18th pre-stressed slabs. The bridge is located 20 feet 3 inches from the north face of the bay and 51 feet 9 inches from the south end of the bay. The length of the bridge is 65 feet 4 inches.

The slab is constructed from pre-stressed concrete piles with reinforced concrete curbs and rail supports (edge) to the upper and outer edges of the deck. The deck sits on transverse beams further supported by concrete piles.

The topping slab is delaminated at the outer edges of deck up to 3 feet from the curbs on either side. The central portion of the topping slab (12-14 feet) was in fair condition and had not yet begun to deteriorate.

At the northern elevation there are spalls and exposed rebar at the rail curb. The spalls coincide with the rail supports. The spalls are over two feet in length, one foot in height, and 3 – 5 inches deep. The underside of the pre-stressed slab is in good condition with only slight staining. At the central transverse beam, there is slight cracking, hairline to 1mm and corrosion of the reinforcing steel is apparent at small spalls where rebar is exposed.

At the transverse beam closest to the dock, there is a large crack and delamination at the left corner that extends over 4 feet in length, 1/8 to ¼ inch in thickness.

Middle Bridge

The middle bridge was constructed in 1980. It is 22 feet wide, and 58 feet long. The pre-stressed slab and transverse beams are in fair condition. The topping slab has delaminated 100%.

The edge curbs are spalled where the rail supports are tied into the reinforced concrete. The piles have spalls and slight delaminations.

South Bridge

The South Bridge is located at Bay 9, 4 feet from the south corner of the bay. The bridge has the same dimensions and construction typology as the North Bridge. Both bridges were constructed when the coal dock was erected in 1969.

The general condition of the pre-stressed slab is good, while the topping slab has delaminated 100%. There are minors spalls on the edges of the bridge deck, like the North Bridge, the spalls are in line with the rail supports.

The transverse concrete beams are exhibiting large horizontal spalls at the lower rebar. At the central beam, the rebar is exposed at the central beam and the corrosion scale could be removed from the rebar. This spall extends the length of the beam. The concrete loss is in excess of 4 inches at the base of the beam. The piles have spalls and slight deterioration above the water line. The bridge deck support beam adjacent to the dock has a delamination and crack at the lower rebar. This crack is 5 feet in length.

Main Deck

The main deck consist of one main area in the way of the supporting deck which consist of a 6" wearing course and prestressed slab. The second area is defined as the walkway slab which in the main consists of a 12" concrete slab.

Our inspection identified that the majority of the deck in both areas is suffering from corrosion of the top reinforcement. There are a number of areas on the main deck where exposed reinforcement with considerable loss of section was found.

As the deterioration of the main deck has been identified as the top reinforcement in the wearing course further investigation to the prestressed slab would be recommended. As the main wearing course is failed all of the support structures bolted into this slab could be considered compromised.

The walkway slab defects were identified in a lot of locations on the top and bottom reinforcement indicating that a loss of capacity is likely. As this area is utilized by personnel during the use of the dock we are recommending repairs are carried out as a priority.

Outboard Elevation

Visual Observations Bay 1

Soffit (Walkway)

The soffit on both portions of the bay have delaminations and spalling There are two areas which have significant spalls and delaminations larger than 4 feet in length, between 3 feet and the back of the soffit to in width and are up to 4 inches in depth. In these locations rebar is exposed and corroding. The adjacent concrete has diagonal and parallel emanating from the rebar, corrosion staining, and delaminations.

Spalling has led to the loss of 11 inches of concrete at the southern most portion of Bay 1 at the expansion joint to Bay 2. This spall extends 17 inches in length at the soffit and face of the walkway. and at the northernmost end of the deck. Large concrete spalls are found at the set back corners for the dolphins where the chains are embedded into the concrete. Where top side anchor bolts penetrate through the concrete, there is transverse vertical cracking.

The top of the walkway has evidence of previous repairs in a number of locations where the patches are cracking and delaminating from the concrete substrate. They are no longer serving their intended purpose.

A steel plate has been installed over the dolphin due to the deterioration of the top side of the walkway deck.

Transverse Beam

The transverse beam (set back) has low horizontal cracking which extends the length of the bay. Concrete dissolution is occurring at the lower portions of this area as evidenced by the larger exposed aggregate. A gunite spray material from a previous repair campaign has been observed in this location.

Pile Caps (Lower Beams)

Extensive loss of concrete and spalling has occurred on the south portion pile cap at Bay 1 along the ledge. Rebar is exposed and has corroded extensively. There is further damage to the concrete in all locations at the top and bottom rebar. The pile caps (outward face and ledge tops) are suffering from extensive cracking at the water line. Cracking extends back the ledge up to 14 inches and along the face the length of the pile cap. There is significant spalling at the step out south of the dolphin.

At the protruding pile cap on the southern portion of the bay, there is a large spall with missing concrete, and 3/8" and greater cracking at the bottom of the beam to the corner of the recessed pile cap.

There is significant cracking at the base of the structure at the set back behind the dolphin.

At the northern portion of the bay at pile caps, cracking and spalling is present.

The end caps for the dock have spalls and cracks.

Rust staining is evident along the cracks at the water level.

Visual Observations Bay 2

Soffit (Walkway)

The soffit of Bay 2 has a significant spall at the southern portion of the bay. This spall is over 3 feet in length, extends the width of the walkway and is up to 4 inches in depth. The rebar at this location is exposed. Continuing evidence of concrete deterioration can be seen above the rebar.

At the top of the walkway there is a spall over 6 feet in length.

Cracking and spalling is present at the fascia and top of the walkway.

Metal plates have been installed immediately north of the impact dolphin. The fascia could not be inspected at this location. Further north, the outward face is horizontally cracked the entire length of the exposed concrete and spalled at both the top and the bottom of the fascia. Vertical cracking at the fascia coincides with the bolts through the deck slab.

Rust staining is present at the top, fascia and underside of the walkway.

Transverse Beam

The transverse beam in the southern portion of the bay has vertical cracking in line with rebar, and horizontal cracking throughout the middle face of the beam. The concrete in this location is delaminated.

At the northern portion of the bay, the entire face of the transverse beam is in poor condition. This are is delaminated and spalling. Where the concrete has not fallen away from the wall, it is displaced outward up to 2 inches. Vertical rebar is exposed. Epoxy injection has been carried out at this location during a previous repair campaign.

Pile Caps

A parge coat/ gunite spray has been applied to the concrete pile caps. There is horizontal and vertical cracking in the coating. In the worst location, this coating is debonded and cracking is present at the rebar. Concrete spalling has occurred due to corrosion leading to a loss of concrete. Rust staining is present.

Visual Observations Bay 3

Soffit (Walkway)

The walkway of the southern half of the bay has extensive cracking and de-bonding of previous patch repairs at the top surface. These extend to the fascia where exposed. The fascia nearest to the dolphin in this location is covered with a steel beam/ plate. There are through deck bolts that are corroding. Cracking patterns coincide with rebar both the length and width of the soffit and also emanating from bolt to bolt.

Concrete spalls are present in a number of locations at the vertical corners of the recess which accommodates the dolphin.

At the northern portion of the walkway a section of the soffit is delaminated and displaced downward. This large piece of concrete will likely fall soon. The depth of the delaminations extend to the rebar, it is over 3 feet in width. There is extensive cracking along the fascia and top deck, as well as concrete exudation.

Transverse Beam

The transverse beam is cracked horizontally and vertically at the outward faces and recesses. Delaminations from the back beam are present. Deterioration of the gunite is occurring in multiple locations and vertical cracking is evident on the out ward sections of the beams.

Rust staining is present at and around cracks. At the base of the beams above the pile caps, concrete dissolution is occurring.

Pile Caps

At the time this area was surveyed, the tide was relatively high. The pile caps were partially submerged, which made photography difficult. What can be noted is that there is cracking the length of the pile caps, spalling at the ledge corners, corrosion staining at the cracks and a large spall at the face of the cap south of the dolphin.

Visual Observations Bay 4

Soffit (Walkway)

The soffit in Bay 4 is in serious disrepair. This area should be considered unsafe.

The entire southern portion of the fascia is covered with a bumper making a visual inspection impossible without removal. The soffit has cracked the length of the walkway, with displaced concrete being held in place by the bumper ledge. The concrete displacement is over 1 inch and the most extensive cracking runs both the length and width of the soffit.

The northern portion of soffit and walkway is in critical condition. This section is separated with a bent. To the right of the bent, the soffit is cracked and delaminating in parallel lines toward the outward face of the soffit at 8 inches and 14 inches . The delamination is displaced 1 inch downward. Additionally, rust staining is present.

To the left of the bent, there is a significant spall which measures 16 feet in length, 3 feet in width, and over 4 inches in depth. More than 1/3 of the concrete is missing from the walkway from beneath. The bottom mat of rebar is fully exposed with some rebar having no contact at all with the concrete. The rebar has extensive corrosion scale.

Transverse Beam

The transverse beam has vertical and horizontal cracking in line with the reinforcing steel. At the left side, southern half of the beam there are large delaminations.

At the northern portion of the beam, there is horizontal cracking the length of the beam both to the right and left of the bent. Small pockets of rebar are exposed, corrosion staining is present and the concrete matrix appears to be disintegrating at the beam, pile cap interface.

Returns on the transverse beams have full height vertical cracking, and at the bent returns there is both horizontal and vertical cracking.

In the location of the access ladder, there is significant vertical cracking and staining. There is also horizontal cracking at the water line at the recess which accommodates the dolphin. Displacement is occurring at these crack locations that are at the water level. These run the length of the concrete recess.

Pile Caps

The pile caps are exhibiting cracking the length the bay at the top and bottom reinforcing steel. The concrete is delaminated in these areas. There is one impact spall at the step out. Corrosion stains are coming from the cracks. This concrete, though delaminated, significantly cracked and stained does not yet have any major spalls.

Visual Observations Bay 5

Soffit (Walkway)

The soffit of Bay 5 is in fair condition. There are cracks and spalls on the fascia. At the south section, there is cracking at the back of the soffit which coincides with damages on the fascia. In this location, there is a large section of concrete missing from the walkway. Additionally in this area, there is staining and holes in the deck due to the removal of previous through deck bolts.

To the north, the soffit is separated by a bent. There is a steel plate that has been installed to the right of the bent, rebar is exposed in this location. The face of the walkway was unobservable and there is minor cracking at the soffit in this location, with the most significant being a crack the width of the walkway where the steel plate has been installed. This crack extends to the transverse beam. There is evidence of rust staining in this location.

To the left of the bent, there are two cracks and spalls which extend the width of the walkway. The northern most crack is displaced downward indicative of a delamination. Smaller cracks appear to be in line with the rebar matt. Rust staining is present.

Transverse Beam

The transverse beams have typical horizontal and vertical cracking in line with the rebar. At the far south edge, close to the expansion joint, there is a vertical crack which extends the height of the beam. There are delaminations in these locations though the concrete appears visually to be in better condition than other locations. The gunite spray has deteriorated. At the recessed area beyond the dolphin, there is full height vertical cracking and horizontal cracking at the base near the pile caps.

At the left return of the bent in the north portion, rebar is vertically exposed at the back of the recess. The face of the bent is cracked at the corners leading to the returns.

Pile Caps

The pile caps in this bay are cracked at upper and lower rebar to the south. At the protruded pile caps there is diagonal, vertical and horizontal cracking of the concrete. The recess has horizontal cracking to the set back pile cap, which coincides with the face cracking at the pile cap.

The north most section the pile cap has vertical and horizontal cracking in line with the lower rebar.

There is corrosion staining at the base of the pile cap in locations of cracks. Cracking also extends to the ledge tops. In this bay there is not significant loss of concrete yet though the concrete has delaminated.

Visual Observations Bay 6

Soffit (Walkway)

A significant portion of Bay 6 walkway fascia is covered with steel plates. Where the concrete is exposed from the outboard side and above, there is significant cracking and previous patch repairs which are cracking. The soffit has cracking, delaminations, displaced concrete and corrosion staining.

The south end is in slightly better condition, though there are cracks, staining, delaminations and concrete displacement. This damage extends half the length of the southern portion of the soffit. A number of through deck bolts are present in the concrete slab and there is staining and discoloration at the outward portion of the soffit in this location

At the north end of the bay, there is a spall which is 4 feet by 3 feet by 4 inches deep. Beyond this spall is a large delaminated piece of concrete which is being held in place by the steel impact bumper. This is displaced downward over 2 inches.

At the recess behind the dolphin, there is horizontal cracking and exposed rebar.

Transverse Beam

At the south most portion of the bay, where the beam steps out, there is significant cracking and spalling at the corner of the return. A piece of concrete is missing and rebar is exposed.

The transverse beam in the southern portion of the bay is in fair condition where it is recessed. There is one vertical crack that extends the height of the beam. The concrete matrix has suffered some deterioration as evidenced by the large aggregate. Lime spots or exudation of the concrete matrix is present.

At the north portion of the bay, the conditions are similar, horizontal, vertical and diagonal cracking are all present.

Pile Caps

The pile caps are suffering from top and bottom rebar corrosion. Vertical and horizontal cracking, and rust staining is evident though the entire bay, this extends to the top ledge of the pile caps. The south most corner is of concrete has spalled or been broken off by impact.

At the northern portion just beyond the dolphin, there is displacement of the concrete at the splash zone/ water line. There are parallel horizontal cracks running the length of the bay. The overall condition of the pile cap in this location is poor.

Visual Observations Bay 7

Soffit (Walkway)

The south most portion of the walkway, at the expansion joint between Bays 7 and 8 has an 11 inch piece of concrete which has broken from the soffit and is being held in place by the metal bumper. Additional spalls are 3 feet in length and vary between 6 and 12 inches in width. Rebar is slightly exposed. Additional cracks and spalls run the length and width of the soffit toward the front, where the through deck bolts are in place.

At the deck where the walkway recesses for the dolphin there is the typical horizontal cracking related to the corrosion of the chain supports (eyelet steel). At the north face to the right of the dolphin, a two foot length of concrete is missing at the deck level. Horizontal rebar is exposed.

At the walkway and soffit in the north portion of the bay, there is spalling at the deck top. This runs the length of the bay, with spalls coinciding with previous repairs. A portion of concrete from the soffit side, at the metal bumper has spalled. The spall is roughly 3 feet in length and 8 - 18 inches in width. Some of the concrete is being held in place by the metal.

Transverse Beam

At the south corner there is a large vertical crack the height of the bay. The transverse beam at the south portion of the bay has an extensive horizontal crack running the length of the bay half. In the middle of the span of the beam is a vertical crack that is exhibiting signs of displacement. In one location horizontal rebar is exposed, indicative of spalling and delaminated concrete. All returns of the transverse beams are cracked.

The transverse beam at the north is suffering cracking and slight displacement. Presently, there is not a significant amount of spalling in this location.

Pile Caps

The pile cap is suffering from cracking, staining and delaminations. In the southern portion there is not exposed rebar at the present time however, there is significant cracking indicative of delaminations which will soon lead to spalling.

The pile caps and bents at the northern portion of the bay have multiple horizontal and vertical cracks running the length of the cap. There is corrosion staining. in many locations and dissolution of the concrete at the water line. At the final bent on this bay, corrosion of the pipes that penetrate through the concrete are causing cracking which will lead to displacement and spalling.

Visual Observations Bay 8

Soffit (Walkway)

There are intermittent spalls at the outward soffit of the southern portion of the bay. These spalls range in size from 6 - 18 inches in length to 6 - 12 inches in width. There are cracks up to 1/8" in width 6 feet in length throughout the soffit. The face of the walkway is obscured with a steel bumper. There is significant staining and 'black rust" or liquid ferrous oxide at the soffit in line with rebar cracking.

Soffit cracking, exudation, and previous epoxy injections are present at the northern portion of the bay. There is a crazing pattern in concrete with delaminations at the soffit. At the walkway in this are there are cracks at previous repairs.

Transverse Beam

The transverse beam in this bay is in fair condition. There are smaller cracks in the beam at the mid height level. The recessed areas have hairline cracking and slight delaminations at the south. To the north there is diagonal cracking with evidence of patching at the returns and recesses.

Pile Caps

The pile caps in this bay are in fair condition. There are smaller cracks in the caps, but the majority of damage is located on the outward corners of the pile caps. These appear to have a parge coat on them, though the cracking damage extends beyond the surface with spalling and the loss of concrete at the right most corner. At the returns of the pile caps, the cracking extends from the face to the recessed pile cap. There is staining and displacement of the cracked concrete. These pile caps are higher than the water line, thus they are not fully in the 'splash' zone.

The last four feet of the pile caps are cracked, with some displacement, light spalling and corrosion staining. As all the damages are occurring at either the ends of the pile caps or at the outward projections, it could be presumed that the damages will increase with time as the corrosion cells move further away from the point of corrosion initiation.

Visual Observations Bay 9

The southern portion of Bay 9 is one full structure. There is no separation between walkway, transverse beam and pile cap. There is cracking and spalling at the top deck at the corners, with the spalls ranging from 20 to 52 inches in length. The cracking is occurring around the bolts and anchorage points, where metal is embedded into the concrete and in line with the reinforcing steel.

Soffit (Walkway)

The soffit to the left of the dolphin has cracked and previous epoxy repairs are present. There is a spall with is 32 inches in length, 22 inches in width with exposed rebar. There is corrosion staining on the soffit. The face is concealed with a steel bumper.

Transverse Beam

The transverse beam has horizontal cracking the length of the beam.

Pile Caps

The pile cap is damaged significantly at the recess. There is a spall which is 3 feet in length, 12 inches high with a 14 inches back to the recess. The bottom rebar is exposed in this location. The recess has cracking in line emanating from the rebar at the spall.

At the recessed areas of the pile cap, there is horizontal and vertical cracking in line with the center of the piles. This appears frequently and is most likely an early indicator of corrosion within the pile steel itself. As this recessed area of the pile cap is in better condition than other bays, early deterioration patterns where elements are in good to fair condition are present.

Inboard Elevation

Bays 9 and 8 are both in fair condition, with minor cracking and spalling. These bays are higher above the water line than the rest of the structure. One major crack is identified on Bay 9, at the pile cap under the south access bridge. The far end of Bay 9 has delaminations and spalls at the south face of the bay. Few delaminations have occurred, there is very little transverse beam damage, and some minor soffit cracking.

Bay 7 is exhibiting large cracks and delaminations at the walkway fascia, the transverse beams and the pile caps. There are delaminations and cracking extending the entire transverse beam that will soon lead to spalls. In one location, rebar is exposed. At the pile cap, parallel cracking is exhibited at the inboard elevation at the upper and lower rebar and are particularly severe at the splash zone. At the ledge of the pile cap, there are cracks that extend from the top to the side of the pile cap which are delaminated and will require removal and repair. It appears that a previous repair was carried out which has exceeded its useful life.

Bay 6 is a hurricane tie down bay. The delaminations in Bay 6 cover half of the transverse beam, and 80% of the pile caps, and 100% of the walkway fascia. At the transverse beam delaminations, the concrete is beginning to spall. This will eventually cause the vertical rebar to be fully exposed. At the pile caps, the cracking is in line with the top and bottom rebar. Surface delaminations have occurred at the ledge and the inboard face. Cracking of the pile cap is more severe to the north of the bay. Soffit cracking is present, primarily to the north of the bay.

The delaminations and cracking of the pile cap in Bay 5 extend the entire length of the bay. The cracking at the top portion of the pile cracks are crazed and appear to be a repair deterioration where as the lower cracking is indicative of lower rebar cracking at the splash zone/ water line. The top of the pile cap ledge is also delaminated. The delaminations at the transverse beam are limited to the southern half of the bay. The entire walkway soffit fascia is delaminated.

In Bay 4, the cracking and delaminations at the pile caps are limited to the lower half, at the splash zone. The entire transverse beam is delaminated at the lower half of the beam and the full soffit fascia is delaminated. There are full width soffit cracks at the southern portion of the walkway.

Bay 3 is a hurricane tie down bay. In this bay, the bottom half of the pile cap is delaminated with cracking at the lower rebar. At both ends of the bay, the concrete supports have vertical and horizontal cracking and delaminations. The damage at Bay 3 is more severe at the south portion of the bay, this is also where the majority of transverse beam delaminations have occurred. Due to the services at the soffit/ fascia of the walkway, it was not possible to hammer test some areas. At the far north there are delaminations, and there is soffit cracking to the south of the bay.

In Bay 2, the delaminations occur at the entire length of the soffit fascia, at the face of the pile caps at the lower rebar for the southern half, the entire pile cap at the northern half and at the entire top of the pile cap ledge. A large full bay crack extends the length of the bay at the splash zone/ water line.

In Bay 1, there is cracking and delaminations of the concrete at the lower half of the transverse beam, the lower half of the pile cap, and at the entire walkway fascia. The north most portion of the bay has vertical and horizontal cracking. This has been previously repaired and the outermost corner has spalled with exposed rebar.

Pre Stressed Slab

For the underside prestressed deck slab evaluation, the team was limited to Bays 9, 8, and 7. This was due to high tides and cross beams which obscured access to Bay 6 and beyond.

Each bay is constructed with a number of prestressed concrete panels which make up the main deck slab. The number of panels vary from bay to bay depending upon overall bay length. These panels are supported by the pile caps and piles.

Overall, the prestressed slab is in good condition in the three bays observed. Three minor spalls were noted in the prestressed panels, one at the 7th panel and 18th panel in Bay 9 near the inboard elevation, and one in Bay 8, near the outboard elevation, at panel 6. The spalls are no larger than 20 inches in length, width and are at the edges of the slabs. There is corrosion of a through slab pipe in Bay 9. There is very light corrosion staining which is present under the prestressed slabs.

General staining, and chemical residue, deterioration is present at the expansion joint between Bay 7 and 6. The backer rod is exposed and it appears that a chemical compound is exuding from the joint.

During this portion of the inspection, pile and pile cap defects were noted. The inboard elevation was in significantly better condition (pile caps) than the outboard elevation. Noted on the inboard side, were two cracks at the pile cap between slabs 11 and 13 and 18 and 20 at Bay 9. The largest of the two cracks spanned between piles and is greater than 6 feet in length, 1/4" in width. Hairline cracking was observed in the pile caps over the center of the piles, which appears as a typical condition throughout the dock facility as seen in Bay 8 at the pile located below panel 14.

Bay 7 has more extensive cracking on both the outboard and inboard elevations. This section of the dock has lower pile caps, which are partially submerged throughout the day. There is significant cracking at the lower rebar of the pile caps and delaminations of the concrete. Towards the outboard elevation there is cracking at both the upper and lower rebar. At the cross beam from Bay 7 to 8, there is delamination of the concrete with exposed vertical rebar.

In Bay 8, the damages toward the outboard pile caps are also more severe than the inboard elevation. At the double width pile cap there is a delamination of the concrete corner over 1 foot in diagonal where rebar is exposed.

The piles toward the outboard elevation have large spalls, exposed rebar, and cracking. This is a typical condition and is documented in detail with the dive report. As noted in the visual inspection, outboard piles are in poor condition. Fewer inboard piles were damaged and where damage was observed, it was less severe. For more detailed information on the piles, please refer to Dive Tech's video files.

Support Piles

The support piles consist of square 18” prestressed columns with the outboard series of piles going to a depth of 30ft below the water line. The inboard piles are only 4 to 5 ft in depth.

Our inspection was limited to three days on site utilizing Dive Tech an experience dive company familiar in the inspection of concrete piles.

Due to the time constraints limited inspection was carried out and we would recommend an additional survey to inspect all piles on the dock due to the construction type (prestressed) and the fact that these type columns from a corrosion perspective are viewed as having a zero chloride diffusion threshold.

Inspection of the columns in the main was good however as detailed within the drawings and documented on the divers video there are a number of defects to the piles.

The piles inspected are as follows with photographic log:

Bay 9

Photo 1- Hairline crack in pile 38

Photo 2- hairline through aggregate pile 38

Photo 3- possible spall or honeycomb pile 38

Photo 4- Missing Section on pile 38

Photo 5- pile 35 out of water spall

photo 6 - out of water spall pile 35

photo 7- metal piece, ground pile 35

photo 8- possible loss matrix pre grind pile 35 on south face at roughly 5' (1048 pn)

photo 9- photo 7 post clean pile 32

photo 10- pile 32 out of water damage

photo 11- out of water crack above crown line pile 32

photo 12 - pile 32 sw corner 2' down loss of material

photo 13- nw corner pile 32 possible loss material @ 3' (pn at 1221)

photo 14- out of water eminent spall pile 32

photo 15- Pile 52 out of water corner cracking se side

photo 16- out of water damage / repair se corner pile 26

photo 17- out of water crack west face pile 26

photo 18 - pile 48 s face cracks with bleeding rust out of water above crown term before crown

photo 19- pile 48 nw corner cracking

photos 20,21,22- pile 46 corner cracking with bleeding rust (a1,a2,a3)

photo 23- pile 24 spalling at the crown

photo 24- pile 24 spalling at the crown

photo 25-corner cracking pile 23

photo 26- corner cracking pile 23

photo 27- e face cracking pile 23

Bay 8

photo 30- crack on south west corner pile 38

photo 31- corner spalling pile 38

photo 32- Pile 37 west face corner cracking

photo 33- Pile 37 corner cracking

Photo 34- Pile 36 possible jacket bottom

photo 35- spall and crack pile 36

photo 36- pile 34 corner cracking

photo 37- pile 34 south face small spall

photo 38- corner spall north west corner pile 30

photo 39- east face of pile 30 spall

photo 40- spall out of water pile 28

photo 41(series)- large corner spall in pile 28

photo 42- pile 25 damage on east face of pile

photo 43- sw corner spall on pile 22

photo 44 & 45 - pile 26 discolorastion NW corner

photo 46- SW corner of pile 26 malformation corner spall less than 1" deep

Bay 7

Photo 47- Inside Beam of 65 and 63 Bleeding Rust in NW corner

Photo 48-Inside Beam of 65 and 63 South Side 6" from bottom of cap crack

Photo 49- BR crack on south east corner

Photo 50-overhead beam ver Pile 61 vertical crack

photo 51- cracking over pile 57

Photo 52- in between piles 57 and 55 cracking

photo 53- pile 61 ne corner spall roughly 6 inches down from cap

Photo 54- nw corner spall Pile 61

Photo 55- nw corner Pile 61 corner crack (inside spall from photo 54)

Photo 56- spall 3 feet from cap northeast corner Pile 44

Photo 57- sw corner pile 35 spall at roughly 5 feet

Bay 6

Photo 58- Bleeding rust above pile 87

photo 59 - eaten out gunite above pile 79

photo 60- nw corner spall on pile 70

photo 61- heavy cracking above 22

Bay 3

photo 65- small spall sw corner on pile 83

photo 66- nw corner spall at mudline pile 78

photo 67- west face hairline pile 78

photo 68- sw corner cleaned concrete is dark in color

photo 69- possible hairline crack west face of pile 60

Bay 2

photo 77- pile 50 west face light discoloration and light cracking

photo 78- pile 50 possible cracking with slight discoloration

Bay 1

photo 79- nw corner of pile 35 spall

Dolphins

The dolphins consist of 4 main "H" piles with a concrete bumper cap to take impact from the docked barges. Only a limited inspection was carried out to the dolphins and we would recommend extending the survey to the dolphins not inspected.

A full survey was carried out to the steel sections of the dolphins under the concrete cap. Significant damage was found on all piles inspected and at dolphin 2 one of the "H" piles was corroded completely through.

For full survey information please watch the enclosed video survey of Dolphin 1,2, 3 & 6.

The following is the photo log of some of the significant corrosion found.

photo 62- h beam under dolphin 6 knife edging

photo 63- h beam under dolphin 6 knife edge

photo 64-heavy h beam damage dolphin six

photo 70 and 71 and 72-north h beam of dolphin 3 @ cap

photo 73-same h beam 18 inches below cap

photo 74- west h beam dolphin 3 west flange

photo 75- possible bend in east h beam

photo 76- dolphin 2 full separation in south h beam

Section 04

Corrosion Models

SECTION 4

Corrosion Models

Chloride Diffusion

Chloride ingress (diffusion) is normally modeled using Fick's law of diffusion and the error function solution. The most common method of predicting chloride transport is Fick's second law of diffusion, as calculated by the error function equation:

(Cmax - Cx,t)/Cmax - Cmin) = erf[(x)/({4 Dct}1/2]

where:

Cmax	= the surface (or near surface) chloride concentration
Cmin	= the background chloride concentration
Cx,t	= the chloride concentration at depth x and at time t
Erf	= the error function
X	= depth
Dc	= the diffusion constant
T	= time of exposure

Diffusion is not the only process by which chlorides are transported into the concrete. For low concrete cover, poor quality of concrete and impingement of salt water onto dry concrete, other processes such as capillary action may be dominant in the first few months and the first few millimeters of the cover. This process can be extremely rapid and can reduce the effective cover to reinforcement by the depth of penetration. However, it is generally accepted that for reasonable quality concrete with reasonable cover after a few years exposure, diffusion appears to predominate for structures with greater than 20mm cover under a wide range of exposure conditions. If these requirements are not met then it is probable that the time to corrosion damage will be too short to warrant modeling.

The diffusion coefficient also varies with time, depending on the cement type Bamforth suggests:

Dca(t) = Dca(m)(t/tm)n

Where:

Dca(t)	= Apparent diffusion coefficient as a function of time
Dca(m)	= Apparent diffuction coefficient at time of measurement tm
T	= time
N	= -0.264 for Portland cement concretes,
	= -0.699 for pulverised fuel ash concretes,
	= -0.621 for ground granulated blast furnace slag concretes

The value of n above were derived from concrete in marine exposure and may vary for different curing and exposure conditions. Diffusion coefficients vary by approximately an order of magnitude in the first ten years and then a factor of 8 for the next 90 years for the blended cements. This ageing factor may be less significant for existing structure modeling from survey data on structures over 10 years old especially if the extrapolation period is short. Also, the variation in the coefficient is far smaller for Portland cements which applies to most older structures surveyed (other than highway structures where pfa has been routinely used for many years). The coefficient is generally smaller for increasing proportions of the addition. The effect of ageing has therefore been considered to be not significant within the accuracy of the model and has not been included in the validation and correlation work.

The Module works by taking an existing structure, analyzing the chloride distribution. It then determines the apparent diffusion coefficient Dca and the apparent surface chloride concentration Coa at the location of the chloride profile. The apparent (or effective) surface chloride concentration is calculated by extrapolating back the profile curve to the surface. The value of Coa is more stable than any measured surface concentration as it is not affected by the latest salt deposit or water run off which could temporarily increase or deplete the surface concentration at the time of measurement. From the calculated parameters, the future diffusion profiles can be predicted and the chloride build up rate at the reinforcement can be calculated. If the chloride concentration for the corrosion threshold is known then the time to corrosion initiation can be predicted.

The values of Dca and Coa are specific to the structure and its local conditions. In some cases they may be even more localized if there are microclimatic changes around the structure or at different levels. The engineer must decide how many profiles are necessary to provide a representative view of the structure, element or location that he is interested in.

The chloride concentration threshold for corrosion

The time to initiation requires a corrosion threshold for the chloride level at the reinforcing bar. A range of values has been used for the corrosion threshold for chlorides. They can also be given in different units. In Europe the most common quantity used is 0.4% chloride by mass of cement. This value depends on factors such as the environment, cement type etc.

The figure below shows one representation of the probabilistic nature of the corrosion threshold from field observations.

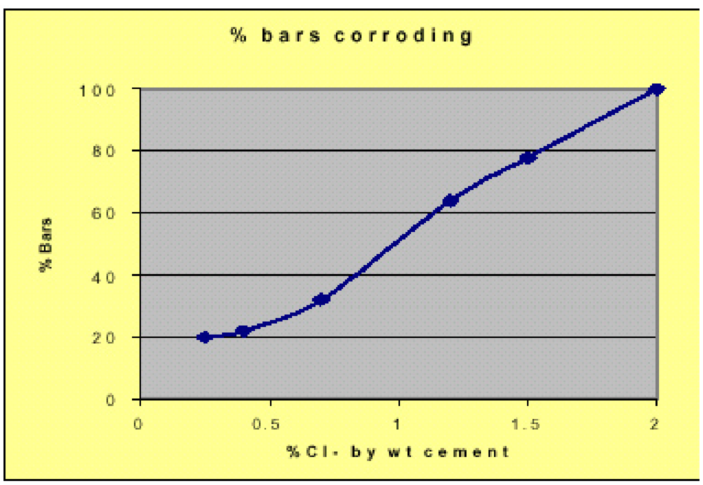


Figure 2 - Probability of corrosion vs. chloride level

The corrosion threshold is a function of:

- Anode and cathode formation – corrosion may not occur at an area that has been made cathodic by an adjacent, rapidly corroding anodic area of steel, despite the chloride level at the cathode being well above the corrosion threshold.
- Moisture level – adequate moisture (and oxygen see below) is required at the steel surface. However, saturated concrete may exclude oxygen and thus suppress the corrosion.
- Oxygen availability
- Free Alkali content – the chloride corrosion threshold has been found to be a function of the chloride hydroxyl ion contents.
- Chloride Binding – in principle, the higher the binding capacity of the cement (usually based on chloroaluminate content of the cement), the higher the corrosion threshold if measured in terms of total (acid soluble) chloride level in the concrete at the steel surface. Some blended cements also have a high chloride binding capacity.
- Carbonation – the hydroxyl ion content is lowered by carbonation. Chlorides bound in the cement phases can be released by carbonation.
- Temperature – chloride binding is lower at higher temperatures.

For many structures in the USA conditions subjected to deicing or sea salt ingress, the corrosion threshold is generally considered to be between 0.4% and 1%. The limit may be lower for blended cement concretes due to their lower free alkali content.

Time to Cracking Model

Cracking can be caused by a number of processes either during hardening of the concrete (e.g. plastic shrinkage), or after construction (e.g. settlement, thermal movement), or due to other deterioration processes such as alkali silica reaction.

The module should be solely concerned with modeling the effect of corrosion after initiation. The module should also offer different ways of estimating the actual corrosion rate, and hence calculating the rate of section loss of the reinforcement and the time to cracking.

The end of service life or time to intervention due to corrosion is generally a subjective and structure specific evaluation. However, clients are rarely interested in time to initiation or even time to first cracking, but do recognize that intervention is necessary once cracks and delaminations occur. The module should allow the engineer to choose a suitable end of service life or time to intervention based on the development of cracks to a critical crack size or based on section loss of critical components and is based on:

- A statistically and seasonally averaged corrosion rate which is measured assuming corrosion has initiated at some location on the structure.
- If the corrosion rate cannot be measured or if corrosion has not yet started, it can be estimated from:
- the chloride level at the rebar in the case of chloride induced corrosion or
- from the relative humidity in the case of carbonation or chloride induced corrosion.

Chloride Ingress Model

As stated earlier, the most widely used assumption is that chloride ingress approximates to Fick's second law of diffusion. It is therefore necessary to calculate an apparent diffusion coefficient Dca to represent the ability of the concrete to transport chlorides, and an apparent surface chloride concentration Coa to represent the chloride environment in order to predict the rate of development of chlorides at the reinforcing bar. In order to calculate Dca and Coa from field structures exposed to chlorides, the chloride profile through the concrete cover must be measured.

The aim of a chloride ingress model is to take chloride profiles and determine the apparent chloride diffusion coefficient Dca and an apparent surface chloride concentration Coa. This is then used to predict future chloride build up and time to initiation and corrosion.

A linear plot of $\sqrt{(Cx,t)}$ vs. depth x therefore yields the effective coefficient with an error check and a regression coefficient. As stated previously, the fixed value of Dca may be used as calculated, with no adjustment for its variation with time as it is assumed that it does not vary significantly within the variability of the data and the accuracy of the extrapolation, and given that:

Most of the change of Dca will occur within the first 10 years. Most surveys are taken after at least 10 years. However, the model does allow the input of the index n if the variation of Dca is considered significant.

It is then possible to determine the time for a chloride concentration to reach minimum, or mean cover depth or any given percentage of bars derived from the cover depth distribution. The following inputs are used to calculate the time to chloride initiation:

- Age of structure at time of survey.
- Background chloride level in concrete in percentage chloride by weight of cement.
- Cover depth in mm. (The actual cover at the profile location, the mean of covers for the structure, the minimum cover or the cover of the first few percent of bars, say 5% can be used as selected by the operator).
- Chloride threshold is percentage chloride by weight of cement (usually 0.4% but other values can be used).
- An absolute minimum of three chloride measurements at three depth intervals. However, five or more should be used where possible in percentage chloride by weight of cement.

The Chloride ingress Model calculates the following:

1. The apparent diffusion coefficient Dca in m^2/s.
2. The effective surface concentration in percentage chloride threshold.
3. The R^2 value for the fit of the profile to the diffusion coefficient calculation (a measure of the goodness of fit, where $R^2 = 1$ is a perfect fit).

	Ave Surface Chloride %Cl by wt of cement	Min Surface Chloride %Cl by wt of cement	Max Surface Chloride %Cl by wt of cement
Total	1.2	0.4	2.5
Car parks	1.2	0.4	2.5
Marine	1.6	0.6	2.4
Bridges	0.7	0.4	1.2

Table 9 - Average, maximum and minimum surface chloride levels by structure type

	Average Time to Corrosion (yr)	Min Time to Corrosion (yr)	Max Time to Corrosion (yr)
Total	67	1	>100
Car parks	46	1	>100
Marine	56	1	>100
Bridges	>100	19	>100

Table 10 –Average, maximum and minimum time to corrosion by structure type

Site Models

Bay 1 Chloride Diffusion Model

Diffusion coefficient Dca in m^2/s.	1.69^{-12}
Effective surface concentration in percentage chloride threshold.	0.6%
The R^2 value for the fit of the profile to the diffusion coefficient calculation (a measure of the goodness of fit, where R^2 = 1 is a perfect fit).	0.994
Time to exceed Threshold	6.4 Years

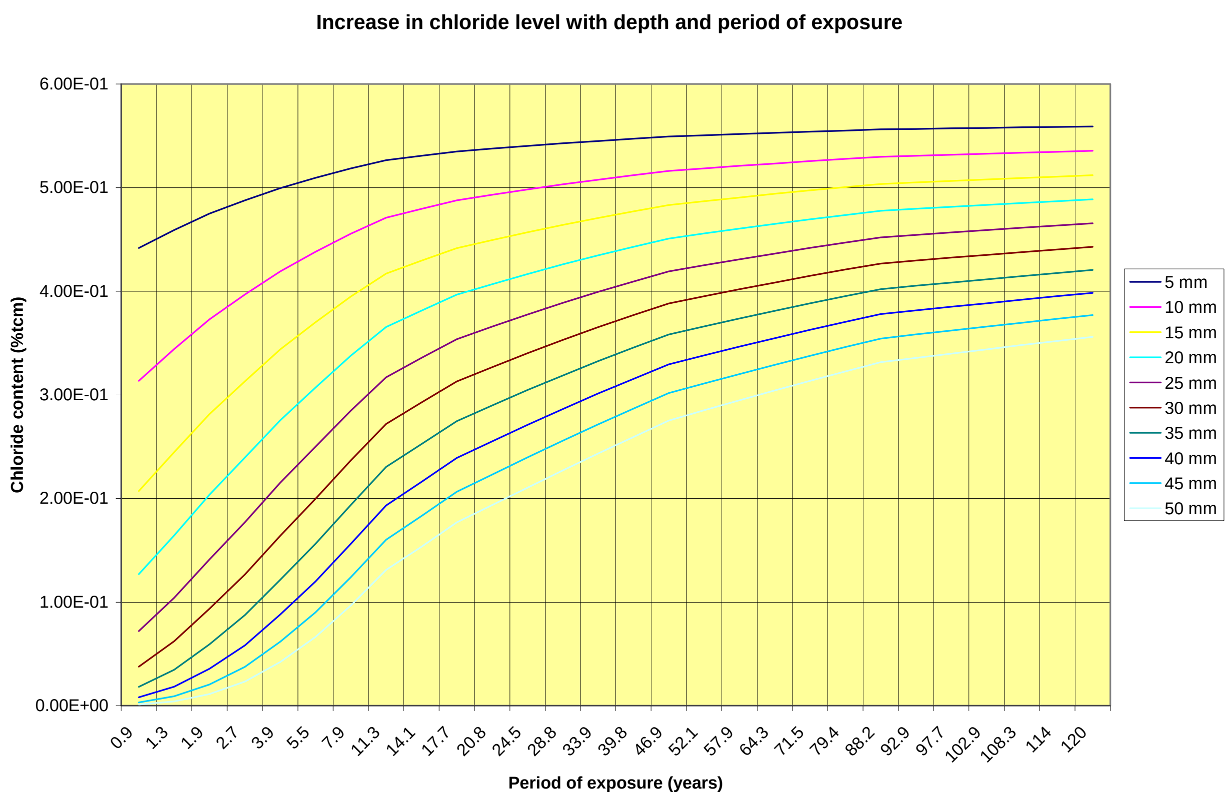


Figure 3 – Bay 1 Chloride Diffusion Model

Bay 1 Time to Cracking Model

Inputs		
Cover	37.5	mm
Bar Diameter	25	mm
Compressive Strength	40	MPa
Corrosion rate	0.68	uA/cm^2
OR		
Chloride at Rebar	0.36	% Cl- by wt of cement
Required Crack Width - w	0.8	mm for $w \leq 1$ mm
Required Section Loss	0.05	mm
Calculations		
w =	**0.05 + B x (X - Xo)**	
Xo =	**83.8 + 7.4 x c/d - 22.6 x Fcsp**	
RESULTS		
Tensile Strength	3.51	MPa
C/D	1.50	mm/mm
Corrosion rate from Chloride	0.35	uA/cm2
Section loss to first cracking - Xo	15.60	microns
Using Corrosion rate - Time to reach Xo	1.99	years
Using Chloride Level -Time to reach Xo	3.89	years
Section Loss to reach w	15.61	microns
Time to reach w using corrosion rate	2.00	years
Time to reach w using chloride level	3.90	years
Time to required section loss from Chloride	12.48	years
Time to required section loss from Rate	6.39	years

Figure 4 – Bay 1 Time to Cracking Model

Bay 4-01 Chloride Diffusion Model

Diffusion coefficient Dca in m^2/s.	9.24^{-12}
Effective surface concentration in percentage chloride threshold.	1.6%
The R^2 value for the fit of the profile to the diffusion coefficient calculation (a measure of the goodness of fit, where R^2 = 1 is a perfect fit).	1
Time to exceed Threshold	0.4 Years

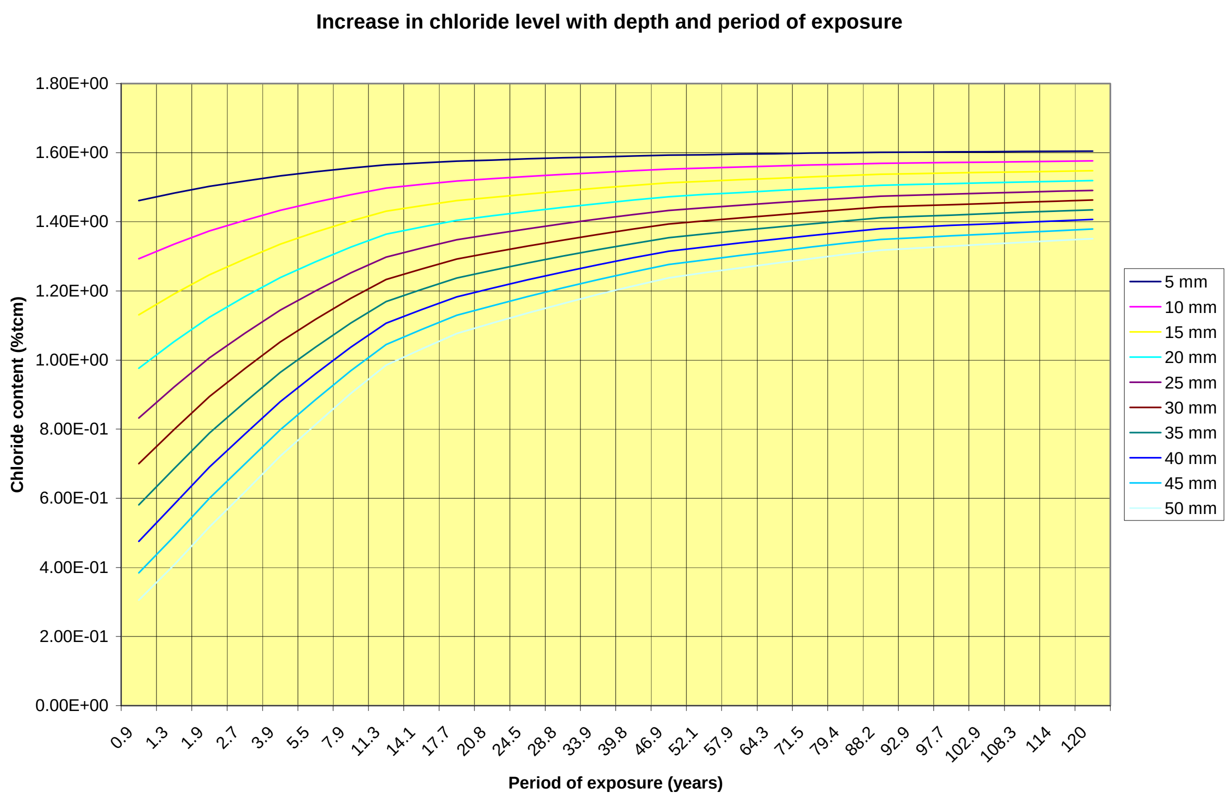


Figure 5 – Bay 4-01 Chloride Diffusion Model

Bay 4-01 Time to Cracking Model

Inputs		
Cover	37.5	mm
Bar Diameter	25	mm
Compressive Strength	40	MPa
Corrosion rate	0.56	uA/cm2
OR		
Chloride at Rebar	1.1	% Cl- by wt of cement
Required Crack Width - w	0.8	mm for w ≤ 1 mm
Required Section Loss	0.05	mm
Calculations		
	w =	**0.05 + B x (X - Xo)**
	Xo =	**83.8 + 7.4 x c/d - 22.6 x Fcsp**
RESULTS		
Tensile Strength	3.51	MPa
C/D	1.50	mm/mm
Corrosion rate from Chloride	0.65	uA/cm2
Section loss to first cracking - Xo	15.60	microns
Using Corrosion rate - Time to reach Xo	2.42	years
Using Chloride Level -Time to reach Xo	2.09	years
Section Loss to reach w	15.61	microns
Time to reach w using corrosion rate	2.42	years
Time to reach w using chloride level	2.09	years
Time to required section loss from Chloride	6.70	years
Time to required section loss from Rate	7.76	years

Figure 6 – Bay 4-01 Time to Cracking Model

Bay 4-02 Chloride Diffusion Model

Diffusion coefficient Dca in m^2/s.	2.56^{-10}
Effective surface concentration in percentage chloride threshold.	0.6%
The R^2 value for the fit of the profile to the diffusion coefficient calculation (a measure of the goodness of fit, where R^2 = 1 is a perfect fit).	1
Time to exceed Threshold	0.2 Years

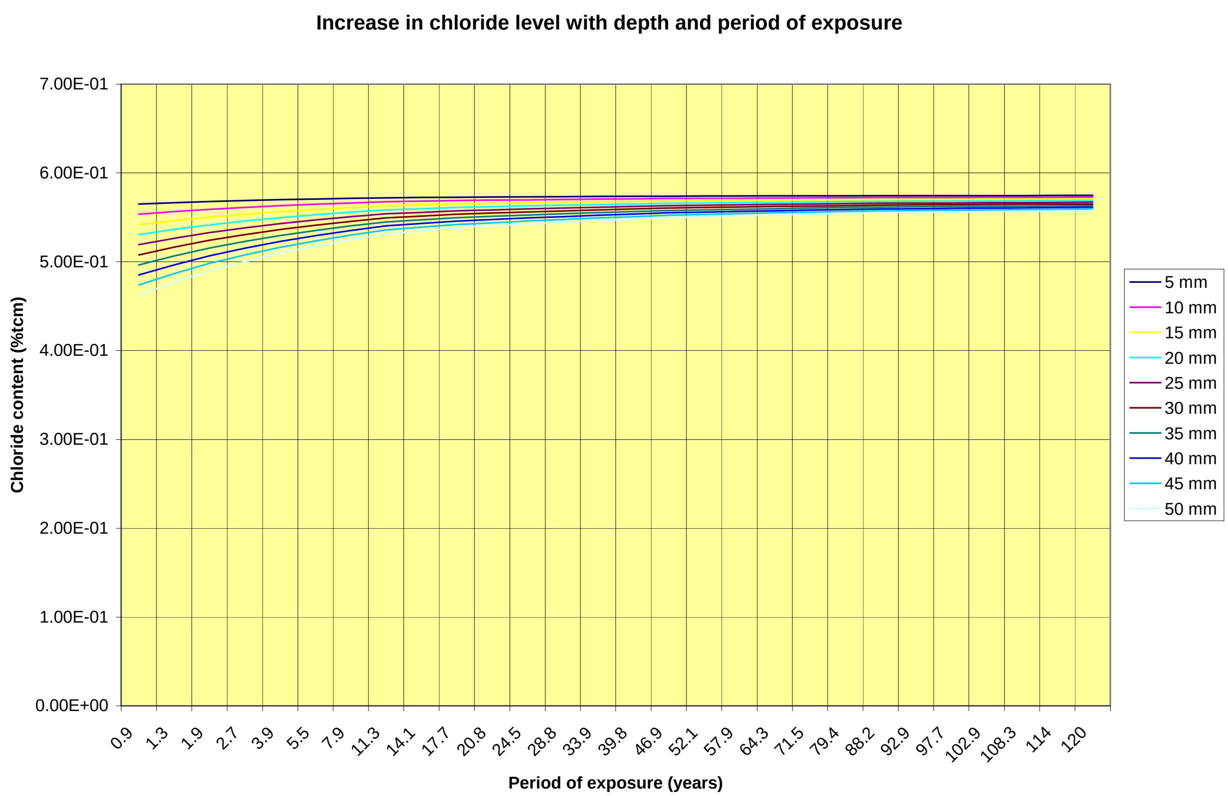


Figure 7 – Bay 4-02 Chloride Diffusion Model

Bay 4-02 Time to Cracking Model

Inputs		
Cover	37.5	mm
Bar Diameter	25	mm
Compressive Strength	40	MPa
Corrosion rate	0.35	uA/cm2
OR		
Chloride at Rebar	0.45	% Cl- by wt of cement
Required Crack Width - w	0.8	mm for w < 1 mm
Required Section Loss	0.05	mm
Calculations		
w =	0.05 + B x (X - Xo)	
Xo =	83.8 + 7.4 x c/d - 22.6 x Fcsp	
RESULTS		
Tensile Strength	3.51	MPa
C/D	1.50	mm/mm
Corrosion rate from Chloride	0.37	uA/cm2
Section loss to first cracking - Xo	15.60	microns
Using Corrosion rate - Time to reach Xo	3.88	years
Using Chloride Level -Time to reach Xo	3.69	years
Section Loss to reach w	15.61	microns
Time to reach w using corrosion rate	3.88	years
Time to reach w using chloride level	3.70	years
Time to required section loss from Chloride	11.84	years
Time to required section loss from Rate	12.42	years

Figure 8 – Bay 4-02 Time to Cracking Model

Bay 6 Chloride Diffusion Model

Diffusion coefficient Dca in m^2/s.	2.38^{-11}
Effective surface concentration in percentage chloride threshold.	1.7%
The R^2 value for the fit of the profile to the diffusion coefficient calculation (a measure of the goodness of fit, where $R^2 = 1$ is a perfect fit).	1
Time to exceed Threshold	0.2 Years

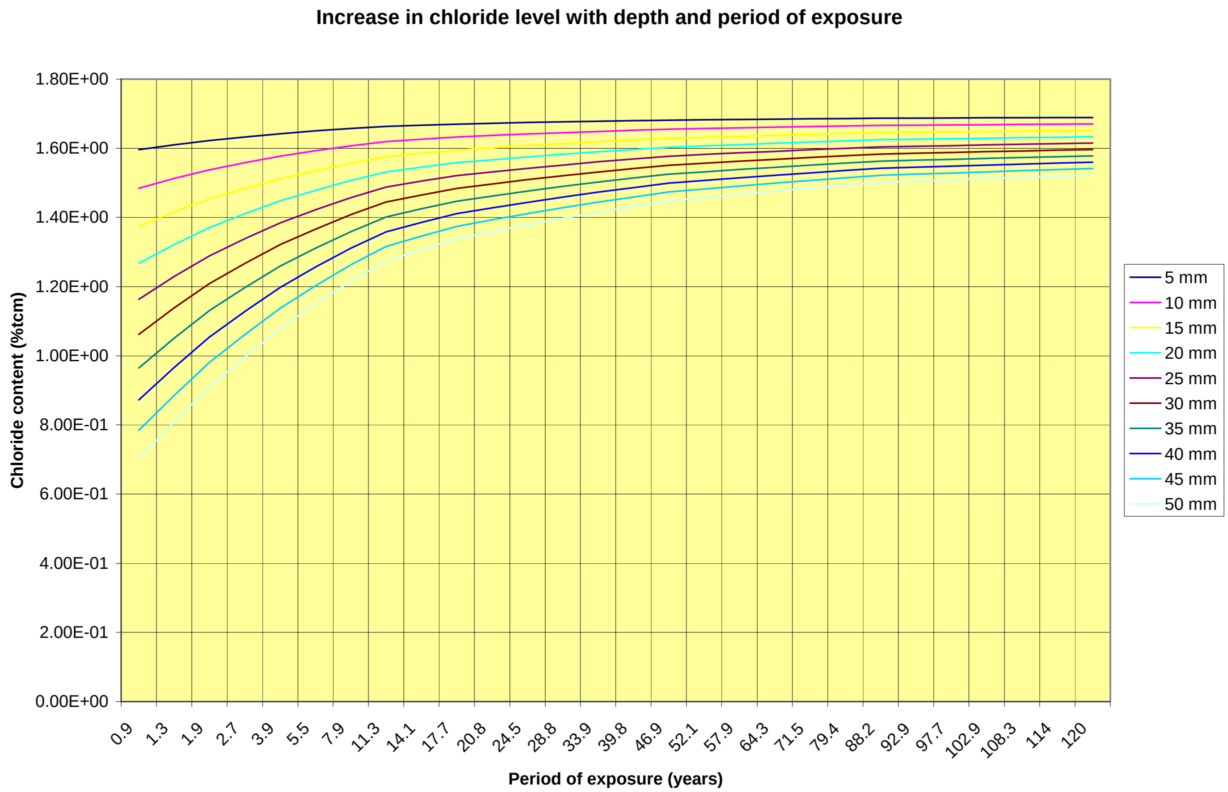


Figure 9 – Bay 6 Chloride Diffusion Model

Bay 6 Time to Cracking Model

Inputs		
Cover	37.5	mm
Bar Diameter	25	mm
Compressive Strength	40	MPa
Corrosion rate	1.56	uA/cm2
OR		
Chloride at Rebar	1.6	% Cl- by wt of cement
Required Crack Width - w	0.8	mm for w ≤ 1 mm
Required Section Loss	0.05	mm
Calculations		
w =	0.05 + B x (X - Xo)	
Xo =	83.8 + 7.4 x c/d - 22.6 x Fcsp	
RESULTS		
Tensile Strength	3.51	MPa
C/D	1.50	mm/mm
Corrosion rate from Chloride	1.04	uA/cm2
Section loss to first cracking - Xo	15.60	microns
Using Corrosion rate - Time to reach Xo	0.87	years
Using Chloride Level -Time to reach Xo	1.31	years
Section Loss to reach w	15.61	microns
Time to reach w using corrosion rate	0.87	years
Time to reach w using chloride level	1.31	years
Time to required section loss from Chloride	4.19	years
Time to required section loss from Rate	2.79	years

Figure 10 – Bay 6 Time to Cracking Model

Bay 9 Chloride Diffusion Model

Diffusion coefficient Dca in m^2/s.	1.19^{-12}
Effective surface concentration in percentage chloride threshold.	1%
The R^2 value for the fit of the profile to the diffusion coefficient calculation (a measure of the goodness of fit, where $R^2 = 1$ is a perfect fit).	1
Time to exceed Threshold	5.4 Years

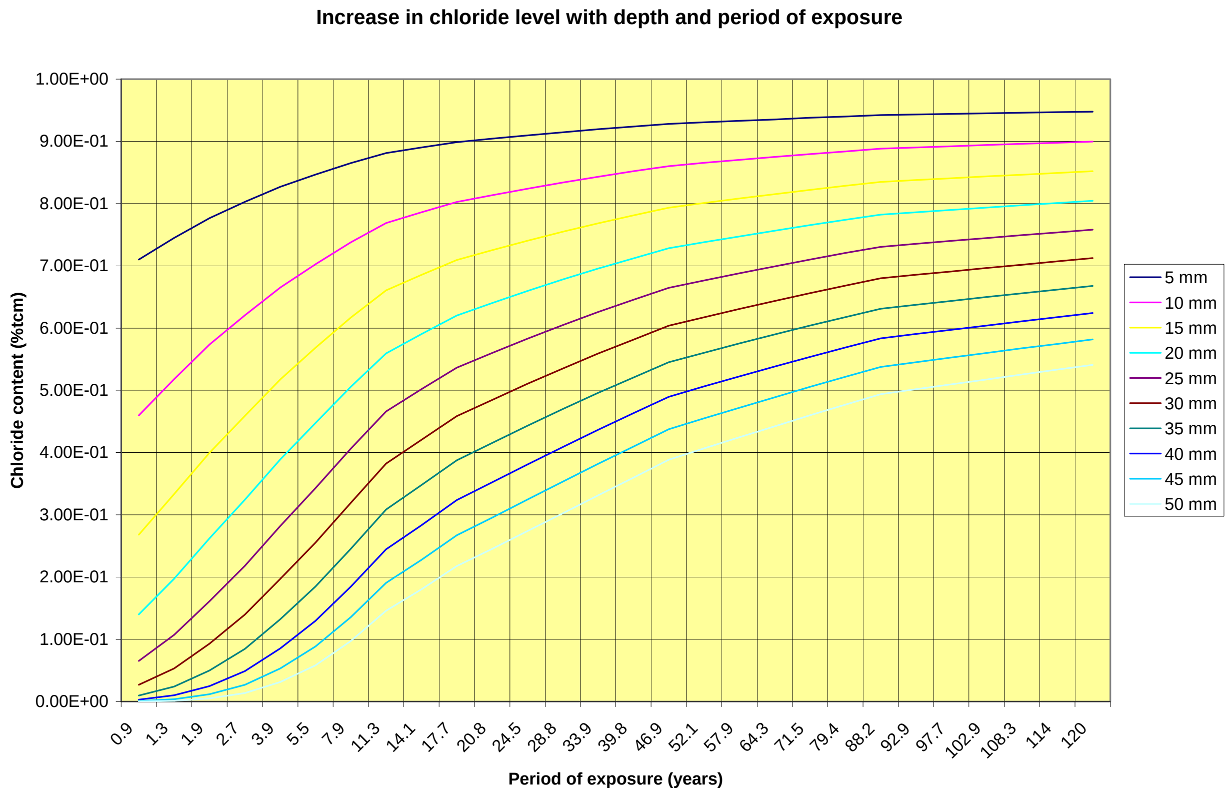


Figure 11 – Bay 9 Chloride Diffusion Model

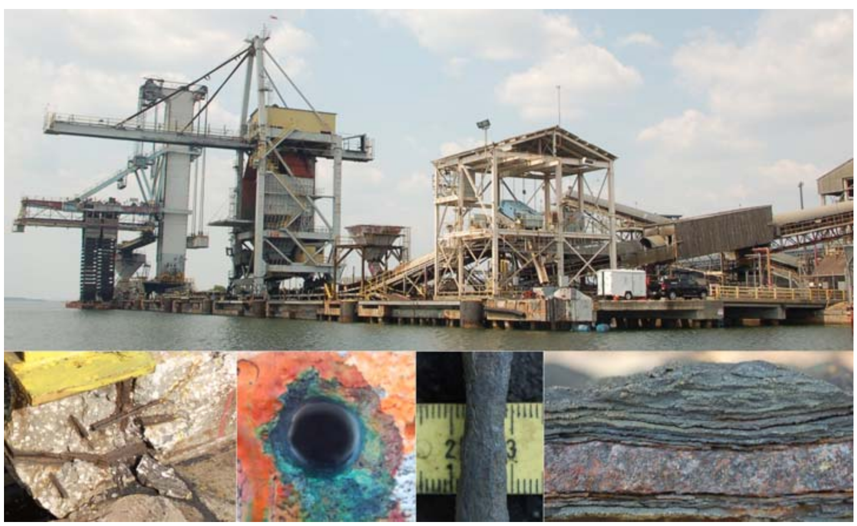
Bay 9 Time to Cracking Model

Inputs		
Cover	37.5	mm
Bar Diameter	25	mm
Compressive Strength	40	MPa
Corrosion rate	0.91	uA/cm2
OR		
Chloride at Rebar	0.32	% Cl- by wt of cement
Required Crack Width - w	0.8	mm for w ≤ 1 mm
Required Section Loss	0.05	mm
Calculations		
w =	**0.05 + B x (X - Xo)**	
Xo =	**83.8 + 7.4 x c/d - 22.6 x Fcsp**	
RESULTS		
Tensile Strength	3.51	MPa
C/D	1.50	mm/mm
Corrosion rate from Chloride	0.34	uA/cm2
Section loss to first cracking - Xo	15.60	microns
Using Corrosion rate - Time to reach Xo	1.49	years
Using Chloride Level -Time to reach Xo	3.97	years
Section Loss to reach w	15.61	microns
Time to reach w using corrosion rate	1.49	years
Time to reach w using chloride level	3.98	years
Time to required section loss from Chloride	12.73	years
Time to required section loss from Rate	4.78	years

Figure 12 – Bay 9 Time to Cracking Model

Section 05

Results



SECTION 5

Results

Covered in this section

Chloride Dust Sample Results Summary

The chloride dust samples are used to obtain diffusion models for the structure and can be summarized below:

Sample No	Depth mm	Chlorides by % by wt of Cement	Summary
Bay 1	5_20 20_50 50_75	0.411 0.295 0.164	Will Exceed upper threshold limit in 6.4 Years at the depth of the reinforcing steel
Bay 4 -01	5_20 20_50 50_75	1.370 1.233 1.055	Already at threshold
Bay 4- 02	5_20 20_50 50_75	0.425 0.548 0.534	Already at threshold
Bay 6	5_20 20_50 50_75	1.918 1.438 1.312	Already at threshold
Bay 9	5_20 20_50 50_75	0.301 0.397 0.199	Will Exceed upper threshold limit in 5.4 Years at the depth of the reinforcing steel

Table 11 - Chloride level Summary

As the results above are based on the diffusion of chlorides into the concrete, it can be seen little to no durability is left.

Potential Survey Summary

The following results provide a statistical analysis of the corrosion potential survey carried out on the structure. We have broken the summary down into various components to provide a more overall picture.

Deck

Min	Max	Average
mV w.r.t. $Cu/CuS0_4$		
75mV	-710mV	-182mV

Table 12 – Half Cell Potential Min, Max & Average Deck

Potential mV w.r.t. $Cu/CuS0_4$	No of Readings	%
>-200	5958	56
-200 to -350	2231	21
<-350	2411	23

Table 13 - Half Cell Potential ASTM C876 Deck

North Bridge

Min	Max	Average
mV w.r.t. $Cu/CuS0_4$		
30	-330mV	-27

Table 14 – Half Cell Potential Min, Max & Average North Bridge

Potential mV w.r.t. $Cu/CuS0_4$	No of Readings	%
>-200	434	99
-200 to -350	6	1
<-350	0	0

Table 15 - Half Cell Potential ASTM C876 North Bridge

Middle Bridge

Min	Max	Average
mV w.r.t. $Cu/CuSO_4$		
130mV	-315mV	28mV

Table 16 – Half Cell Potential Min, Max & Average Middle Bridge

Potential mV w.r.t. $Cu/CuSO_4$	No of Readings	%
>-200	453	98
-200 to -350	7	2
<-350	0	0

Table 17 - Half Cell Potential ASTM C876 Middle Bridge

South Bridge

Min	Max	Average
mV w.r.t. $Cu/CuSO_4$		
50mv	-255mV	26mV

Table 18 – Half Cell Potential Min, Max & Average South Bridge

Potential mV w.r.t. $Cu/CuSO_4$	No of Readings	%
>-200	434	99
-200 to -350	1	1
<-350	0	0

Table 19 - Half Cell Potential ASTM C876 South Bridge

Inboard Elevation

Min	Max	Average
mV w.r.t. $Cu/CuS0_4$		
194	-701	-300

Table 20 – Half Cell Potential Min, Max & Average Inboard Elevation

Potential mV w.r.t. $Cu/CuS0_4$	No of Readings	%
>-200	447	33
-200 to -350	341	25
<-350	582	42

Table 21 - Half Cell Potential ASTM C876 Inboard Elevation

Combined Results Inboard & Deck

Potential mV w.r.t. $Cu/CuS0_4$	No of Readings	%
>-200	6405	44.5
-200 to -350	2572	23
<-350	2993	32.5

Table 22 - Half Cell Potential ASTM C876 Inboard & Deck

Corrosion Rate Summary

The following results provide a statistical analysis of the corrosion rate survey carried out on the structure. We have broken the summary down into various components to provide a more overall picture.

Inboard elevation

Min	Max	Average
Metal Loss (mpy)		
0.002	1.46	0.18

Table 23 – Corrosion Rate Min, Max & Average Inboard Elevation

icorr ($\mu A/in^2$)	Metal Loss (mpy)	Percentage %	Severity of Damage
<0.031	0.0914	56	No corrosion damage expected
0.031 to 0.155	0.0914 to 0.45704	29	Corrosion damage possible in 10 to 15 years
0.155 to 1.55	0.45704 to 4.5704	15	Corrosion damage expected in 2 to 10 years
>1.55	4.5704	0	Corrosion damage expected in 2 years or less

Table 24 – Typical Section Inboard Elevation

Outboard Elevation

Min	Max	Average
Metal Loss (mpy)		
0.014	3.02	0.43

Table 25 – Corrosion Rate Min, Max & Average Outboard Elevation

icorr ($\mu A/in^2$)	Metal Loss (mpy)	Percentage %	Severity of Damage
<0.031	0.0914	27	No corrosion damage expected
0.031 to 0.155	0.0914 to 0.45704	39	Corrosion damage possible in 10 to 15 years
0.155 to 1.55	0.45704 to 4.5704	34	Corrosion damage expected in 2 to 10 years
>1.55	4.5704	0	Corrosion damage expected in 2 years or less

Table 26 – Typical Section Loss Outboard Elevation

Deck

Min	Max	Average
Metal Loss (mpy)		
0.0001	1.51	0.16

Table 27 – Corrosion Rate Min, Max & Average Deck

icorr (µA/in²)	Metal Loss (mpy)	Percentage %	Severity of Damage
<0.031	0.0914	37	No corrosion damage expected
0.031 to 0.155	0.0914 to 0.45704	60	Corrosion damage possible in 10 to 15 years
0.155 to 1.55	0.45704 to 4.5704	3	Corrosion damage expected in 2 to 10 years
>1.55	4.5704	0	Corrosion damage expected in 2 years or less

Table 28 – Typical Section Loss Deck

Combined Results All Areas

Min	Max	Average
Metal Loss (mpy)		
0.002	3.02	0.26

Table 29 – Corrosion Rate Min, Max & Average Combined All Areas

icorr (µA/in²)	Metal Loss (mpy)	Percentage %	Severity of Damage
<0.031	0.0914	40	No corrosion damage expected
0.031 to 0.155	0.0914 to 0.45704	42.5	Corrosion damage possible in 10 to 15 years
0.155 to 1.55	0.45704 to 4.5704	17.5	Corrosion damage expected in 2 to 10 years
>1.55	4.5704	0	Corrosion damage expected in 2 years or less

Table 30 – Typical Section Loss Combined All Areas

Only a minimal amount of corrosion rate readings were taken and this equated to less than 1% of the assessable areas surveyed.

In addition to this as chlorides are present the measurement of corrosion rate does not take into consideration pitting corrosion which is where a concentration of corrosion occurs. The measurement is based on a uniform area of 0.107639 which has not been adjusted in our measurements tabled above.

Cover Survey

The cover survey was limited to the areas where corrosion rate measurements were carried out on the same grid pattern. The corrosion rate node point was used for the measurement and this does not necessarily reflect the lowest cover in the area.

In general cover was good in all areas tested and is summarized in the table below:

Min	Max	Average
Inches		
1.7	2.56	2.17

Table 31 – Minimum, Maximum & Average Concrete Cover

Carbonation

Carbonation was tested at the 5 drilled areas used for chloride dust samples. All holes tested indicated a carbonation depth of less than ¼”.

Photographic records of the carbonation test are shown on the drawings.

Results Summary

The following tables summarize on a global basis the data collected during the inspection:

Test	Min	Max	Average	Comment
Chlorides	0.295	1.438	0.7822	Readings taken at average depth of reinforcing steel and measured in chlorides by weight of cement
Potentials	44.5%	32.5%	23%	The potential survey indicated that over half of the areas tested corrosion activity is present.
Corrosion Rate	0.002mpy	3.02mpy	0.26mpy	As only a small area of corrosion rates were tested and large delaminations are present the results would indicate further damage in less than 10 years
Concrete Cover	1.7”	2.56”	2.17”	Good average concrete cover depth.
Carbonation	<1/8”	¼”	¼”	Carbonation was not an issue at sound locations tested

Table 32 – Results Summary

Results Conclusion

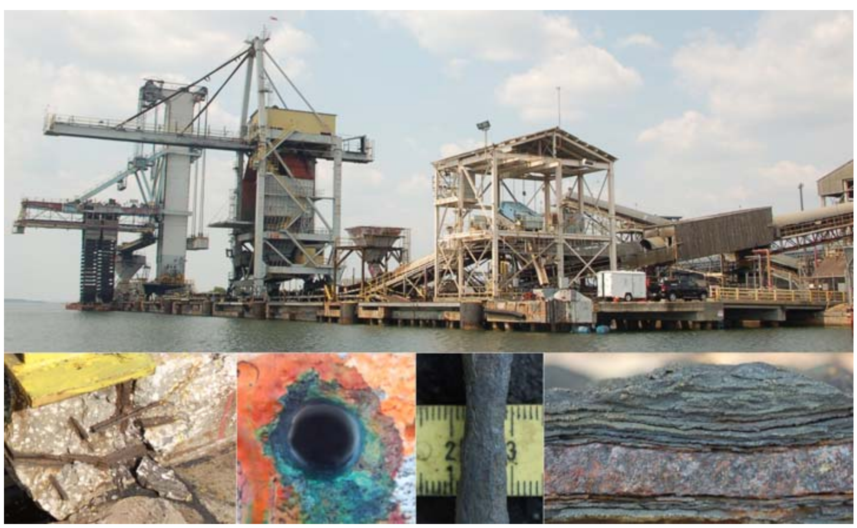
In conclusion the results of the testing have identified the following:

- All chloride levels exceed ASTM guidelines
- 50% of all areas tested showed active signs of corrosion
- Concrete Cover is good
- There is no carbonation at the depth of the reinforcing steel
- Corrosion rates indicated levels beyond corrosion initiation
- Corrosion models predict all chloride thresholds to be exceeded in less than 2 years.
- Visual inspection throughout the inspected areas has significant delamination of concrete.
- Dolphins are structurally compromised
- Access walkways around structure with defective top and bottom reinforcing steel.

All of the above key points indicate the dock requires repair within the next twelve months before additional repair cost are incurred. The increase in cost would be attributable to the continued deterioration of the structural components of the dock.

Section 6

Recommended Repairs



SECTION 6

Recommended Repairs

Structure Definitions

The dock facility comprises of a number of different structural components as follows:

- Bridges including supporting beam and piles.

  Three (3) number bridges defined as North, Middle and South

- Main Deck

  Pre-stressed deck slab and supporting deck slab wearing course surface

- Pile Caps

  XX Pile cap configurations across supporting piles

- Transverse Beams

  Load transfer beams from main deck into pile caps including overhanging walkways

- Main Dock 18" Pre-stressed Supporting Piles

  Main structural supports for entire dock

Critical Components

To enable a repair methodology to be developed we typically look at the key functional components and what requires repairs first. In order to achieve this we look at the durability of the components based on their current condition and their predicted deterioration against time.

The dock is broken into three key Critical components as follows:

1. Transverse Beams, Pile Caps and Piles
2. Bridges
3. Main Deck

Repair Options

Based on the critical components the current defects provide us with a number of options. These options are centered on the sequence of work items necessary to extend the life of the dock beyond 20 years.

Option 1 – 10 Years

This option is based on providing solutions where an extension of the dock is achieved for a minimum period of 10 years with minimal to no maintenance. Due to the current condition of the dock and its classification there are no options available to meet this criterion.

NOTE: Minimal maintenance cost would be deemed to be less than $250,000 over the 10 year period.

Option 2 – 20 Years

In order to repair the structure the following areas have been identified that require a corrosion control repair to achieve a 20 year extension with minimal maintenance.

Transverse Beams Inboard & Outboard Elevations

The transverse beams on the outboard and inboard elevations should be repaired using a galvanic mesh system incorporated in a fiberglass jacket (lifejacket).

LifeJacket®: Description:

Corrosion of reinforcing steel in concrete is an insidious problem. It significantly affects the functional characteristics of concrete by inducing cracks and spalls, which compromise the structural integrity of the structure itself and increases the threat to public safety. Today, engineers and owners are looking for new ways to solve the corrosion problem and are increasingly focused on the conservation of structures rather than replacement. To deal with this challenge, a system for the repair of concrete and corrosion control, known as the LifeJacket®, has proven itself as a simple, cost-effective, minimally-invasive solution.

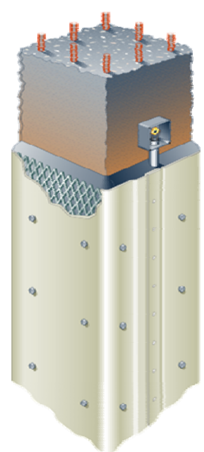


Figure 13 – Typical Lifejacket

Our evolution into the cathodic protection market began many years ago with the concept of using recycled products as an anode system for protecting bridge pilings in saltwater environments and the application of thermally applied coatings on concrete surfaces. Since that time, we have further refined our ideas and cultivated them into working "Systems" that have been laboratory tested, field installed and providing satisfactory performance for more than fifteen years. To date there has been more than 5,000 Lifejackets installed on more than 100 separate projects in eight states and in six countries.

The LifeJacket® Galvanic Protection System uses a proprietary zinc mesh anode placed directly against the inside face of a stay-in-place fiberglass form, and is proven to stop corrosion by providing an electrical current to the affected region. The simple fiberglass jackets can be customized to fit any type of structural component with minimal effort required in the field. One of the biggest advantages in using the LifeJacket® System is that it restores concrete section loss and provides structural strengthening to deteriorated conditions.

Figure 14 –Lifejacket Beam Application

Once installed, the System does not require additional maintenance or monitoring. In addition, it does not require post-installation adjustments to keep the system operating properly. There is no need for wiring and complex conduit systems for routing current to the source. The System operates maintenance free over its design life with no additional utility bills, consultant fees or reapplication costs - a significant benefit to owners.

<u>Piles</u>

The piles are to be protected by a galvanic system in the way of bulk zinc anodes. The dual anodes are to be designed for installation to each prestressed column and electrically bonded to the transverse beam to ensure total protection over the full length of the pile.

Deck Walkways

The deck walkway on the inboard and outboard sides of the dock should be repaired with an impressed current cathodic protection system. (ICCP) The system offers a life extension of greater than 20 years and would deal with the current condition of the dock.

Impressed Current Cathodic Protection: Description:

The cathodic protection of metal against corrosion was first demonstrated in the early nineteenth century. Since that time, this process has been used to stop the corrosion of metallic structures and components in a wide variety of environments.

Essentially, cathodic protection is the intentional application of a direct electric current in opposition to the naturally occurring electrochemical corrosion of metal. Cathodic protection is now a generally accepted and economical practice for reducing or eliminating the corrosion of metals, particularly steel. Steel structures as varied as underground storage tanks, ships' hulls, oil well casings, hot water heaters, gas pipelines, concrete reinforcing steel bridges, and offshore drilling rigs, are successfully protected by cathodic protection.

The natural corrosion of steel involves the formation of an electrochemical corrosion cell. This cell is made up of an anode and a cathode, typically at two different sites on the steel component, an electrolyte, and an electrical connection between the anode and cathode. The chemical reaction at the steel anode site is the oxidation of the metal, followed generally by oxide or hydroxide formation:

$$Fe \rightarrow Fe^{++} + 2\,e^{-} \qquad (1)$$
$$Fe^{++} + 2OH^{-} \rightarrow Fe(OH)_2 \qquad (2)$$

At the same time, an electrochemical reaction, generally the reduction of atmospheric oxygen, occurs at the steel cathode site:

$$\tfrac{1}{2}O_2 + H_2O + 2e^{-} \rightarrow 2OH^{-} \qquad (3)$$

For steel section embedded in masonry, the steel will corrode when the passive (non-corroding) steel surface is de-stabilized. The steel is de-stabilized when exposed to carbonation and in some instances chloride ions which de-stabilize the normal oxide film on the steel embedded in the alkaline environment. The chloride ions may result from the use of deicing salts on the sidewalk, exposure to sea water or marine fog. Carbonation is where Atmospheric CO2 dissolves in the pore water and forms carbonic acid. Acids react with alkali to form water and a neutral salt, so the carbonic acid reacts with the calcium hydroxide to form calcium carbonate. Once the calcium hydroxide is consumed the pH drops from 13 to 9 and de-stabilizes the oxide film. The steel then corrodes in the presence of the oxygen and water available in the masonry pores. The electrochemical corrosion cell is set up when two different parts of the steel, which are electrically bonded together, act as the anode and cathode.

The electrolyte in this case is the masonry, which will normally contain enough moisture to conduct the electrical corrosion current. Since the steel corrosion products, the iron oxides, occupy a larger physical volume than the un-corroded steel, the corrosion will exert tensile stresses on the surrounding masonry, with the stresses increasing until cracks, displacement or stone failure occur.

In order to reduce or stop the corrosion reactions shown above, the steel component must be made cathodic, so that reaction (3), and not reaction (1), will occur on the whole of the steel surface. Correspondingly, an anodic reaction must occur on the surface of an anode which is provided for the cathodic protection (CP) system.

In impressed current CP systems, the anode is generally a conductive material which is not consumed. A typical anode is a titanium substrate covered on its active surface by a noble metal or metal oxide catalyst. The anode reaction in this case will generally be the formation of oxygen from water:

$$2H_2O \rightarrow O_2 + 4H^+ + 4e^- \quad (4)$$

For impressed current CP systems, a separate source of DC current needs to be supplied, and reactions (3) and (4) will occur.

Dolphins

From the dolphins inspected repairs are necessary to strengthen the heavily corroded "H" piles before installing a galvanic bulk zinc anode. The repair scope would involve the use of underwater welding of new plates to replace the corroded sections after a full inspection of the piles is carried out.

All Other Areas without Additional Corrosion Control

The repair scope for the rest of the dock would be defined as full depth repairs and this would be limited to areas of exposed reinforcing steel on the deck or other areas not covered by a corrosion control solutions.

All of the defects in these areas would be defined as non essential and could be carried out after all critical works are complete.

The other area we have defined as critical would be the eight (8) expansion joints in the top deck. These joints need total replacement in the form of an Emseal Type DSM system or similar.

Section 7

Repair Scope

SECTION 7

Repair Scope

Covered in this section

Lifejacket Repair Scope

The scope of work proposed for the Lifejacket system is limited to the transverse beams on the outboard and inboard sections of the dock. The system shall be designed to provide adequate cathodic protection to the soffit of the beam, the sides and returns where applicable.

The following is a typical section of an installed lifejacket.

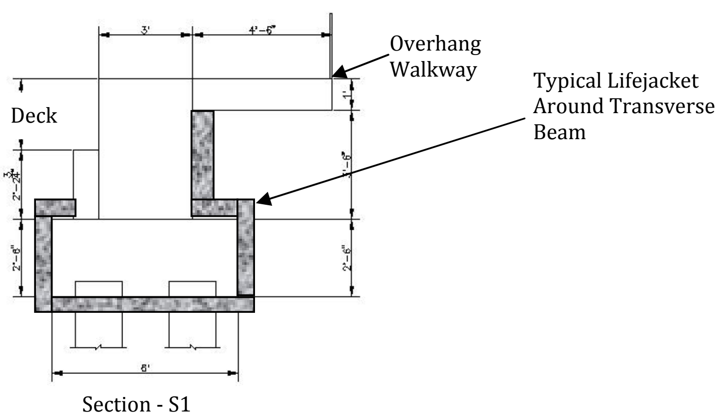


Figure 15 – Typical Lifejacket Installed to Transverse Beams

Bulk Anode Scope

The scope of work required for the bulk anode is all of the underwater components which include the prestressed piles and dolphins.

Calculations would need to be carried out to determine the size and shape of these anodes to provide a 20 year life extension.

Impressed Current cathodic Protection Scope

The impressed current scope is limited to the access walkways around the perimeter of the dock. This is due to the current condition and the long term durability of the twenty year extension.

A slotted anode system would be utilized on the top surface of the concrete which would be designed to provide full cathodic protection to the underside reinforcement.

Concrete Repair Scope

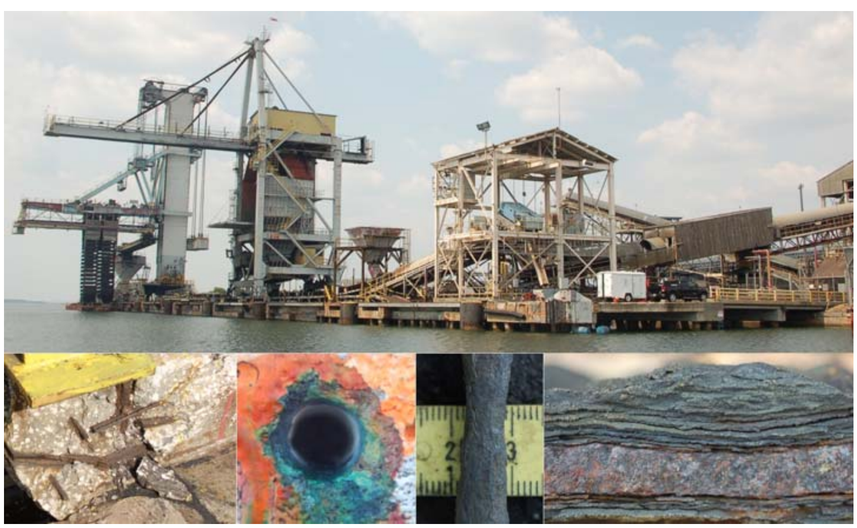
All defective concrete identified within the corrosion control areas will require repair as part of the corrosion system. In addition to this selective areas of the main deck and bridge decks where exposed reinforcement is shall also form part of the repair scope.

Expansion Joint Scope

All eight expansion joints require repair due to the potential contamination and subsequent damage to the main prestressed slab.

Section 8

Technical Specifications



SECTION 8

Technical Specifications

Covered in this section

- Lifejacket
- Bulk Anodes
- Impressed Current Cathodic Protection
- Concrete Repair

Lifejacket Specification

SERIES 2400 – LIFEJACKET SPECIFICATION

2410 GENERAL

1. The work under this section consists of supplying, installing and energizing a sacrificial/galvanic anode cathodic protection pile jacket system (Lifejacket System), including connection to the steel, materials, testing, and ensuring continuity between all embedded steel components on designated prestressed piles on the specified structures. The cathodic protection pile jacket system consists of continuous sheets of expanded zinc mesh anodes attached directly to the inside surface of FRP integral pile jackets. This "System" is manufactured by Jarden Zinc Products, Greeneville, TN or approved equal, and is installed on the piles at the elevation shown on the construction plans. It also includes bulk zinc anodes installed on each pile receiving a cathodic jacket. The elevation of the bulk zinc anodes shall be sufficient to provide immersion of the anode at all times. Any substitution materials, or alternative systems; must be submitted in advance for approval and have as a minimum ten (10) years of proven performance in similar applications.

2. The specification and installation of the complete system shall be of a standard to achieve a minimum life of 25 years use with minimum planned maintenance that require closures for inspection work or repairs of a minor nature.

2415 CONTINUTITY BONDING

3. Continuity of the prestressing steel, reinforcing steel, dowel bars, and spiral ties shall be provided by resistance welding or other approved method. On piles where a pile splice is detected during clean up, continuity shall be tested and provided if required between both pile sections.

2421 SUBMITTALS

4. Prior to commencing the cathodic protection pile jacket installation, the Contractor shall submit for approval shop drawings indicating equipment, materials, and the procedures for installing the jacket. Include details on the following: the negative connections to the steel, continuity check and correction procedures, data requirement to establish the mean high and low water elevations and anode system fabrication, including bulk anode and hardware, and the jacket.

2433 MINOR PILE REPAIRS OUTSIDE JACKET LIMITS

5. Minor Pile Repairs Outside Jacket Limits: The Contractor shall also restore to original dimensions minor concrete delaminations and/or spalls on the piles that may be present (above MLW line) outside the limits of the jacket as designated by the Engineer.

2440 CATHODIC PROTECTION SPECIALIST

6. The Contractor shall secure the services of a Cathodic Protection Specialist accredited by the National Association of Corrosion Engineers (NACE), a professional engineer with a minimum of five years of experience in the field of cathodic protection on concrete, or a corrosion engineering practitioner with no less than 12 years of certifiable experience in the field of cathodic protection on concrete.

7. The CP Specialist, or the technician under his direction, shall supervise the overall installation including participation with the Contractor in designing the construction sequence, performing random site visits to oversee every phase of the work. Additionally, the CP Specialist, or the technician under his direction, shall be responsible for all the continuity testing, testing all the continuity corrections, and performing the initial energizing on all piles including: current, static, and energized potential measurements.

8. The CP Specialist, or the technician under his direction, shall also check for shorts between the anode and the steel and notify the Contractor for correction as necessary prior to placing any jackets. Testing for shorts will be done before and after the filling materials are set. The CP Specialist's NACE certification shall be submitted to the Engineer for approval.

2443 QUALITY CONTROL

9. The Contractor shall submit a quality control plan for approval prior to commencing the jacket installation. Cost of a quality control and the CP Specialist services shall be considered incidental, and included in the cost of the jacket. Prior to commencing any work, the Contractor shall determine the scope and sequence of work so that the appropriate measures are taken to ensure proper quality control throughout the project.

2447 CERTIFICATION STATEMENT

10. The CP Specialist shall sign and notarize the following statement, and shall submit the original copy to the Engineer after completion of the project.

    "I hereby certify that the facilities constructed under (project number) have been completed to the point where the facilities are functionally complete. I further certify that construction on these facilities has preceded substantially in accordance with the contract plans and specifications or that any deviations, which are noted below, will not prevent the system from functioning in compliance with the intent of the contract when properly operated and maintained. These determinations have been based upon on-site observation of construction, scheduled and conducted by me or by a representative under my direct supervision, for the purpose of determining if the work proceeded in compliance with the contract documents."

2453 MATERIALS AND TESTING

11. The Contractor shall be responsible for the repair or replacement of any damaged private or public property resulting from his/her operation. Any testing required to assign responsibility of damage shall be secured by the Contractor at no cost to the Owner.

12. The Contractor shall use only approved materials from approved sources and shall furnish material certifications to the Engineer of Record for approval prior to placing any materials. Alternative materials must meet or exceed the specified requirements and be tested by a certified, third party testing laboratory.

13. Material certifications must be supplied and approved before commencing any work. Materials not meeting the specifications herein will be rejected for re-submittal. The "System" must be installed according to specification and in accordance to the Manufacturer's recommended procedures.

2461 JACKET SYSTEM INSPECTION

14. The Contractor shall inspect all piles and locate all deteriorated concrete areas on the pilings that are to receive the jacket "System". All areas to receive the jacket "System" and the surrounding concrete surfaces shall be sound tested by the Contractor to determine the actual dimensions of the deteriorated concrete to be removed.

15. Each jacket should encompass the entire problem area within the specified jacket limits. The Engineer reserves the right to add or delete piling repair and protection as necessary. Dimensions of the spalled areas shall be recorded by the Contractor and verified by the Engineer.

16. A final report detailing locations and size of the spalls and/or cracks shall be provided by the Contractor at the end of the project. Remove all delaminated, cracked or unsound concrete from the areas, which are hollow sounding when tested, or areas with visible cracks (up to 1/64 inch wide) may not need to be removed as directed by the Engineer.

2465 SURFACE PREPARATION FOR CATHODIC PROTECTION JACKET SYSTEM

17. Surface preparation for piles shall include the removal of all loose or delaminated concrete, to provide a minimum of 1 inch clearance between exposed steel and the surrounding concrete in the damaged area.

18. All reinforcing steel shall be maintained at original position and all exposed steel shall be sandblasted or hydro-blasted clean to a gray metal condition prior to concrete placement. No traces of rust, mill scale, epoxy or other contaminants shall be present after cleaning. Special attention shall be observed to ensure proper preparation of the backside of the reinforcing steel.

19. If the existing reinforcement has lost more than 15% of its cross section, new steel of equal area shall be installed a minimum of 12 inches beyond the deteriorated reinforcement. Additionally the Contractor is responsible for removing all residue or marine growth in the areas where the bulk anode will be installed.

2467 CONCRETE REMOVAL

20. Spalled concrete removal and clean up under this type of work is considered incidental to the jacket installation. Care shall be exercised as to contain falling debris from entering into the water. Debris includes but is not limited to scrap metal, demolition debris, concrete and concrete dust, zinc, etc. A containment plan shall be supplied by the Contractor for approval by the Engineer prior to commencing any work.

2470 CATHODIC PROTECTION PILE JACKET SYSTEM

21. The jacket "System" shall consist of stay-in-place fiberglass forms provided with a zinc mesh anode pre-installed against the inside surface in a continuous expanded metal sheet and filled with an approved sand-cement mortar for non-structural jackets (4000 psi minimum).

22. The jacket "System" also includes the installation of a zinc bulk anode at an elevation below the low water mark as shown on the contract drawings. The "Lifejacket System" is manufactured by Jarden Zinc Products Company and distributed by ElectroTech CP, LLC (561) 222-9037.

2473 ZINC MESH ANODE

23. The zinc mesh anode attached inside the jacket shall be continuous sheets of expanded zinc mesh placed in direct contact with the face of the fiberglass jacket and conforming to special A-190 zinc alloy and certified against ASTM B 69 test procedure. The zinc mesh must be tested and meet the following chemical composition (chemical analysis to be provided with the material submittals).

a.	Pb	0.003% wt. max.
b.	Fe	0.001% wt. max.
c.	Cd	0.003% wt. max.
d.	Al	0.001% wt. max.
e.	Ti	0.001% wt. max
f.	Cu	0.7-0.9% wt. max.
g.	Zn	balance

24. The mesh anode shall have the following physical properties:

a. Electrical conductivity = 28% min.
b. Solid zinc density = 0.28 lbs/in3
c. Weight of expanded zinc mesh = 1.60 lb/ft2 min.
d. Average open area = 53%
e. Solid sheet thickness = 0.90 inch

25. The expanded zinc mesh anode shall also conform to the following nominal geometry to allow proper mortar encapsulation:

a. 0.500 inch hex pattern
b. 0.125 inch strand width SWD
c. 0.563 inch strand bond width LWD
d. 0.313 inch short opening
e. 0.750 inch long opening

2477 ZINC MESH CONNECTION WIRES

26. The expanded mesh anode shall be provided with a connection wire which shall extend a minimum of 9 inches above the top of the jacket to perform the connection to the reinforcing steel inside the terminal box as shown in the plans.

2481 JACKET LOCATION AND LIMIT

27. The jacket "system" shall be installed on the designated transverse beams detailed in the construction drawings. The Contractor shall field verify all jacket lengths prior to placing any orders.

28. Jackets measuring six feet or longer will require staged pumping ports. Filling material shall not be dropped into the form from elevations greater than six feet.

29. Filling material for jackets extending below water level shall be pumped from the bottom upward using the staged pumping ports. The pumping process shall continue after initial filling until no water is present at the highest discharge point of the jacket and a uniform grout consistency is achieved.

30. The FRP forms (jackets) shall be composed of a durable, inert corrosion resistant material with an interlocking joint along two opposite sides that will permit the form to be assembled and sealed in place around the pile.

31. All joints need to be sealed for a grout-tight seal prior to placing any of the fill material. Forms shall be fabricated from fiberglass and polyester resins. The form dimensions shown in the plans are minimum dimensions permitted.

32. The minimum wall thickness for the jackets is 1/8 inch. Upon placing the forms around the pile, they should be grout-tight and capable of maintaining their shape without assistance or damage. Jacket stand-offs may require field fabrication after removal of unsound concrete to assure proper alignment of the jacket during fill material placement.

33. The inside-face of the form shall have no bond inhibiting agents in contact with the cement grout or mesh anode. The forms shall be provided with nonmetallic bolt stand-offs which will maintain the forms in the required position.

34. Assembly and jacket preparation shall be completed at the factory before delivery to the job site. The forms shall be properly sealed in the field to provide a positive seal of the annular space between the pile and the form.

2483 JACKET FORM MATERIAL

35. The material furnished for the jacket forms must meet the following physical property requirements:

    a. Water Absorption (ASTM D570) – 1% max.
    b. Ultimate Tensile Strength (ASTM D638)* - 15,000 psi min.
    c. Flexural Strength (ASTM D796)* - 25,000 psi min.
    d. Flexural Modulus of Elasticity (ASTM D790) – 700 ksi min.
    e. IZOD Impact (ASTM D256) – 15 ft-lb/inch min. (unnotched)
    f. Barcol Hardness (ASTM D2583) – 45 min.

* On original specimen whose flat surfaces are not machined to disturb fiberglass.

2483 JACKET FILL MATERIAL

36. The concrete fill material shall consist of a mixture of Portland cement, fine aggregate, water and approved admixture. The use of fly ash or slag in the mix is not allowed for this project. The filling material shall contain 1,000 pounds of cement per cubic yard with sufficient water to produce a workable mix. The fine aggregate shall be clean silica sand and there shall not be any additives containing chlorides or other salts corrosive to metals. A minimum 28-day compressive strength of 4,000 psi is required for nonstructural jackets. The use of fly ash or slag is not approved for this project.

37. Final acceptance of the filling material shall be made based on compressive strength results meeting or exceeding those established by the approved mix design.

38. After the filling material has cured for a minimum 72 hours, all temporary form support and/or bracing shall be removed from the piles and the exterior of the forms shall be cleaned of any filling material which may have been deposited. The top of the jackets shall be sloped as shown in the construction drawings.

Bulk Anode Specification

SERIES 2500 – BULK ANODE SPECIFICATION

2510 GENERAL

1. The Contractor shall furnish and install a bulk zinc anode along with each "System." The bulk anode shall be placed at an angle at the depth shown in the construction drawings. The Contractor shall be responsible for any necessary information search and surveying work to determine the correct elevation on each pile.

2. The bulk anode shall be a 50 lb, 99% pure zinc anode (hull type anode) with a steel strap core, conforming to Mil-A-18001K (1). The steel strap shall be hot-dip galvanized with a minimum thickness of .005 inches. A hole shall be drilled at each end of the strap for mounting. Such hole shall be fabricated prior to galvanizing. Location and size of the holes are shown in the construction plans.

3. The anode shall be clamped onto the pile using two; 2 inch galvanized steel channels with flange sides facing the concrete surface (as shown in the plans) using galvanized hardware.

2520 CONNECTION

4. A No. 6 AWG copper strand wire with HMWPE insulation shall be connected to the anode via a 3/8 inch diameter round steel bar welded to the anode strap. The No. 6 AWG wire shall be brazed to the bar, and the bar-wire connection shall be permanently encased in a 1 ¼ - inch diameter by 8-inch long PVC pipe filled with epoxy. The remainder wire shall be routed to the jacket inside a ¾ inch diameter PVC pipe. All required fabrication shall be done prior to the anode installation. The wire insulation shall be protected from heat during the welding and brazing operation.

5. Special precautions may be necessary to protect the wiring insulation and splice during anode installation. The ¾ inch pipe shall extend to an elevation of approximately 2 inches inside the bottom of the jacket. No conduit will be required on the portion of wire inside the jacket. The wire shall be routed upward along the closest corner and positioned between the fiberglass form and the zinc mesh anode.

6. At the top of the jacket, the wire shall be routed in conduit to the PVC terminal box located immediately above the jacket. At this location, the bulk anode wire shall be connected to the zinc mesh anode wires and routed via connection to a 5/16 inch diameter stainless steel bolt to the reinforcing steel connection wire as shown in the construction drawings.

7. Bulk anode installation shall be performed prior to placement of the filling material for the jacket.

2524 NEGATIVE CONNECTIONS (PRESTRESSED STRANDS)

8. General: The Contractor shall install an electrical negative connection on each pile receiving cathodic protection. The connection shall be performed by brazing two No. 10 AWG THWN copper strand wires to different areas of a spiral tie at the elevation shown in the construction drawings.

9. A sufficient length of wire shall be used such that the wires can be routed to the terminal box mounted on the pile without any splices.

10. This location shall be maintained constant at every pile unless otherwise approved by the Engineer and the CP Specialist. The brazed part of the negative connection wire (at the spiral ties) shall receive a coat of 100% solids, non-conductive epoxy such that no wire or brazing material will be in contact with the concrete when patching. The wire shall be brazed to a minimum length of the spiral tie of 1 inch.

11. All connection lead wires shall be routed to the PVC terminal box located immediately above the jacket as shown in the drawings. The negative lead shall be connected to the wire originating at the mesh anode and to the bulk anode wire in the terminal box.

12. Soldered electrical ring connectors shall be used for the connection. Connection between the ring connectors shall be made using 316 stainless steel bolts, nuts and washers. The connection shall be properly insulated after completion.

13. Wire splices and connections insulating method and materials shall be submitted to the Engineer for approval prior to performing this work. In the case of structural jackets that require a steel reinforced cage; two negative connection wires will be required to provide protection to the new steel and routed separately to the terminal box.

2532 TERMINAL BOX

14. Terminal Box: The terminal box is placed above the jacket and fastened either directly to the pile or on the pile cap. The terminal box shall measure 4 inch by 4 inch by 2 inch or other suitable size with weather tight cover and shall be attached to the concrete with four 316 stainless steel fasteners per box.

15. All PVC components shall be schedule 40, sunlight resistant. All hardware for installation of the PVC conduit and terminal box shall be 316 grade stainless steel.

16. Elevation of the terminal box shall be maintained constant throughout the project. A 0.1 ohm shunt shall be placed inside the terminal box and wires for measuring the current shall be routed to two, ¼ inch diameter stainless steel bolts that shall extend outside the terminal box.

2541 CONTINUITY

17. Concrete excavation to expose the spiral tie shall be performed inside the jacket limits. Dimensions of the excavation shall be kept to a minimum but not exceed 4 inch by 4 inch. Routing wires outside the excavation to the conduit system shall be performed inside the jacket conduit attached to the terminal box.

18. The Contractor shall submit details of the intended method for this operation and materials specifications for approval by the Engineer. The Contractor shall verify continuity between the connections and the spiral tie prior to coating with epoxy. Repair any connection testing discontinuous at no extra cost. After connection is approved, the excavation shall be filled with an approved mortar prior to the jacket installation.

19. Prior to installing the jacket, the CP Specialist or technician, shall perform an electrical continuity test between all strands, spiral ties, and dowel bars (if present) on all piles receiving cathodic protection. The CP Specialist shall certify such tests correct and a detailed report shall be provided to the Engineer at the end of the project. Strands and dowels for continuity test shall be exposed by drilling a ¾ inch diameter hole to each strand and/or dowel in the concrete and measuring inter-strand (or dowel) voltage using a high impedance voltmeter.

20. Drill holes in a staggered pattern within the limits of the jacket. Using existing exposed steel for continuity testing when possible. Some additional chipping may be necessary to expose the stands and/or dowels.

21. Where continuity correction is required, additional concrete excavation will be necessary. Size of continuity correction excavation shall be maintained at the minimum required to expose the discontinuous to a continuous adjacent strand as shown in the construction plans or advised by the Engineer.

22. On piles where discontinuous strands are found on two or more faces of the pile, saw-cut a 3-inch wide groove at an elevation no less than 6 inches below the top of the jacket. Continuity shall then be provided to all strands inside the groove.

23. Any hole and/or excavation for continuity testing shall be filled with an approved concrete repair mortar prior to placing the jacket. Special care shall be observed to avoid cutting any of the strands or spiral ties during the drilling or saw cutting operation.

24. Provide continuity by resistance welding two continuous solid steel wires to each strand requiring continuity correction inside the excavation. Re-test continuity on all strands after this operation is completed. All welds shall be approved satisfactory by the Engineer, appointed inspector or CP Specialist before coating with epoxy. Continuity welds shall receive a coat of 100% solids, non-conductive epoxy such that no welded wire shall be in contact with the concrete when patching. Intended resistance welding equipment and procedure shall be included and submitted for approval in the shop drawings prior to performing this work.

2541 COMMISSIONING

25. The CP Specialist shall submit a report to the Engineer detailing: continuity testing and correction, anode to steel resistance, initial current, and static and energized on and off potentials for each pile. The commissioning report and the Contractor's spall size log shall be submitted to the Engineer at the completion of the project.

Impressed Current Cathodic Protection

SERIES 2600 – MISCELLANEOUS

2610 GENERAL

1. The Contractor shall install commission and maintain a impressed current cathodic protection system on the walkway areas of TECO Coal dock as detailed within the contract drawings.

2. The specification and installation of the complete system shall be of a standard to achieve a minimum life of 25 years use with minimum planned maintenance that require closures for inspection work or repairs of a minor nature.

3. The installation, energizing, commissioning, long term operation and the documentation of all elements of the cathodic protection systems shall be fully documented. The Contractor shall submit details of his proposed Quality Management system of record keeping to the Project Manager for approval. Subsequent approval of any work may be withheld in the absence of acceptable written records.

4. The Contractor shall submit details of the following to the Project Manager 2 weeks before starting work on site or ordering of materials:

- Materials and specifications
- Method statements or specifications for the installation, testing and energizing
- Schedule of materials used
- Detailed plan for monitoring and final assessments

5. All works relating to the installation and commissioning of the cathodic protection system shall be supervised by a cathodic protection specialist with a minimum qualification of Professional Member of the Institute of Corrosion, a post graduate qualification in corrosion engineering or corrosion science, or a certified Corrosion Specialist of NACE International with practical experience in the installation and operation of cathodic protection systems to reinforced concrete. The specialist shall have at least 3 years experience in installing at least two completed cathodic protection systems to atmospherically exposed reinforced concrete structures or shall have demonstrated equivalent expertise to the satisfaction of the Project Manager.

6. The Contractor shall supply evidence of qualifications and experience of the proposed specialist cathodic protection personnel. The Project Manager shall have the authority to approve or reject the nominated personnel or organization. No changes to the personnel involved in the project shall be made without approval of the Project Manager.

7. Any material or equipment whose standard is not stated within this specification shall be in accordance with the current Standard, along with subsequent amendments to that standard, relating to that material or equipment and shall be to the standard current at the time of inviting tenders. Equivalent materials or products not covered by current Standard may be permitted subject to the approval of the Project Manager, provided the Contractor can demonstrate successful performance on similar installations.

The Contractor shall provide the Project Manager with the following details, as a minimum, of all materials or products prior to approval for use:

(i) Manufacturer

(ii) Technical details and specification

(iii) Test results

(iv) Examples of successful installation

8. All operations comprising repair shall be carried out in accordance with the concrete repair specification except where otherwise stated in this paragraph.

Concrete Removal

i) All repair materials from previous installations with an electrical resistivity outside the range 50% to 200% of the nominal parent concrete electrical resistivity shall be broken out to achieve a clean concrete surface.

ii) Any tying wire, nails or other metal components visible on the concrete that might contact the anode system or might be too close to the anode for optimum anode/cathode spacing shall be cut back and the concrete shall be repaired.

iii) Existing loose delaminated concrete shall be broken out to sound concrete, except on soffit areas where the concrete shall be removed from behind the reinforcement for a distance of 25mm beyond the rear face of the reinforcement.

Reinforcement Preparation

iv) Any loose corrosion product particles shall be removed from exposed reinforcement to ensure good contact between the steel and the repair material. Neither primers nor coatings on the steel nor insulating/resistive bonding agents shall be used.

Concrete Reinstatement

v) All repair materials shall have an electrical resistivity within the range 50% to 200% of the nominal parent concrete electrical resistivity.

vi) All repairs shall have a smooth, level trowelled finish or equivalent to give an acceptable visual appearance and minimize the accumulation of deposits on the surface.

9. The Contractor shall comply with all current Health and Safety Standards throughout the duration of the works. Particular attention should be given to the containment of debris, dust noise and other forms of hazard and nuisance to users of the structure resulting from the works.

10. On completion of the installation and commissioning of the cathodic protection system the Contractor shall operate the system for a period of 52 weeks. The Contractor shall provide documentation consisting of Record Drawings, a Commissioning Report, an Operating and Maintenance Manual and a Report of the System Review.

11. At the end of the maintenance period the cathodic protection system, the contents of the main cabinet including control computer and other equipment necessary for the ongoing operation of the cathodic protection system, shall be handed over to the Project Manager in working order and shall become the property of TECO Energy.

2620 DESIGN, PERFORMANCE AND PROTECTION CRITERIA

1. The cathodic protection design, prepared by the Project Manager, is based on the information provided by the reinforcement drawings and this technical specification.

Protected Steelwork

2. The cathodic protection anode system(s) is designed to provide protection to the reinforcement as shown on the contract drawing.

Anode Placement

3. All anodes shall be placed within or on the surface of the concrete as shown on the contract drawings.

Power Supply System

4. The power supply system shall be designed on the basis of a control system located on the main deck of the dock. The system must be capable of remote monitoring and control with an adjustable output of 1mA steps.

Current Density and Operating Voltage

5. The current density for the system shall be 1.85mA/ft^2.

Anode Zones

6. The Contractor shall install a minimum of nine anode zones as shown on the contract Drawings.

Protection Criteria

7. The cathodic protection system shall deliver sufficient current to satisfactorily meet the requirements within NACE SP0290-2007.

2625 Standards

1. The work shall be carried out in accordance with the following standards and codes, which form the basis of this technical specification:

    i) BSEN12696:2000; Cathodic Protection of Steel in Concrete.

    ii) Concrete Society/CEA Technical Report No. 36 & 37; Cathodic Protection of Reinforced Concrete and Model Specification for Cathodic Protection of Reinforced Concrete.

    iii) NACE SP0290-2007: Cathodic Protection of Steel in Atmospherically Exposed Concrete Structures.

    iv) NACE TM0294-01, Item No. 21225 "Standard Test Method - Testing of Embeddable Impressed Current Anode for Use in Cathodic Protection of Atmospherically Exposed Steel in Reinforced Concrete"

2. The Contractor shall ensure that all aspects of the cathodic protection installation are carried out such that the completed installation is to the satisfaction of the Project Manager.

3. The components of the cathodic protection system shall be installed in strict compliance with the component manufacture's instructions unless otherwise instructed by the Project Manager.

4. All electrical equipment shall be installed in accordance with the National Electrical Code (NEC).

2634 Materials

Anode System

1. The ribbon anode systems described below shall be installed at locations shown on the contract drawings.

2. The Contractor shall provide the Project Manager with the following details of the anode materials prior to approval for use:

    (i) Manufacturer;

    (ii) Technical details and specifications;

    (iii) Examples of successful applications;

    (iv) Operating voltage limits;

    (v) Operating current output limits;

    (vi) Theoretical predicted anode life.

3. Alternative anode system(s) other than those described below will be considered providing the Contractor can demonstrate their successful application on at least five similar structures and have a track record of a minimum of three years. Supporting laboratory research evidence of accelerated tests to demonstrate satisfactory performance and achievement of design life may also be required.

Ribbon Anode System

4. The ribbon anode material shall be catalysed, expanded titanium mesh. The mesh shall have a minimum guaranteed working life (at maximum output) of at least 25 years, and it shall have a proven track record, details of which shall be submitted to the Project Manager for approval.

5. The anode grout shall be a BASF 928 or equivalent cementitious mortar with a track record of application to at least two ribbon anode systems on soffit or vertical surfaces on marine or highway bridge structures suffering from chloride attack. The names of the owners of the structures shall be required.

Monitoring Sensors - Reference Electrodes

6. The Contractor shall provide and install reference electrodes as shown on drawing no. the contract drawings to enable the polarised level of the steel to be determined. This will enable the Contractor to demonstrate the performance of the system in line with the requirement under the protection criteria.

7. Reference electrodes shall be silver/silver chloride/potassium chloride electrodes (Ag/AgKC1) or manganese/manganese dioxide (Mn/MnO2/0.5M NaOH) gel electrodes of robust construction cased in a material, which is electrically insulating and suitable for permanent embedment and exposure in highly alkaline conditions, or acid conditions if close to the anode system.

8. The Contractor shall provide written confirmation to the Project Manager 7 days prior to the start of the monitoring works that the reference electrodes are compatible with the Contractors cathodic protection system and remote sensing.

9. Reference electrodes shall have a theoretical life expectancy in excess of 25 years based upon encapsulation in concrete and interrogation of up to once per week on average for 1 hour with instrumentation of 10M.ohm input impedance or greater.

10. They shall have a predicted accuracy of ±l0mV for the 25 year electrode life expectancy and shall have an accuracy of ±3mV over any 24 hour period. Accuracy of the electrode is defined as the potential of the electrode measured with respect to a calibrated laboratory Saturated Calomel Electrode (SCE) at 20 °C; the measured potential shall be within ±3mV of the theoretical or manufacturer stated potential with respect to SCE.

11. The Contractor shall offer a reference electrode for inspection and approval by the Project Manager and shall submit a manufacturers test certificate to prove compliance with the specification. The Contractor shall submit certificates of accuracy and predicted life expectancy, and also a material specification for the electrode and cabling.

Power Supply Unit

12. The AC powered supply unit shall be designed by the Contractor on a centrally controlled rectifier system with individual outputs for each zone. The unit must be capable of full remote control and operation. The system shall comprise a main power supply unit, a remote monitoring unit and control software.

13. The AC power supply (Low voltage 120 A.C, 60Hz) is to be obtained from the AC electrical panel on the dock as shown on the contract drawings.

14. The power supply unit shall have LEDs or other means of indicating a.c. power 'on' and d.c. output 'operating' shall be provided.

15. The power supply units are to have their own microprocessor for control with a back up facility for the output settings should there be a power loss.

16. The power supply units shall conform to the following requirements as a minimum.

Internal Impedance:	>10M ohms
Ambient Temperature:	-10 to +85ºC
Regulation Steps:	10mV/5mA
Measuring Accuracy:	0.5mV
Efficiency:	>85%
Linearity:	Less than 10mV
Output Ripple:	10 mV at 100 Hz

Control Unit and Computer

17. The AC operated control unit shall be housed within the same groundside cabinet as the power supply units. The control unit shall read and set the operating parameters of each of the local distributor units, process any alarms and treat collected data for reporting purposes etc.

18. The control unit, which shall act as the interface between the site and the control computer, and shall enable full remote control of the following functions.

    - Turn each power supply unit ON/OFF
    - Read and Set outputs to each power supply unit
    - Carry out Depolarisation Test using embedded reference cells
    - Retrieve and store depolarisation test data and routine monitoring data
    - Monitor individual power supply units in real time
    - Print graphical data of logged samples
    - Carry out instant off potential measurements between 0.1 and 0.4 seconds.
    - Control and record Relative Humidity and Temperature Probes.

19. The Contractors control unit shall communicate remotely with the control computer. If this is via a modem link in a separate land line, the Contractor shall provide the dedicated telephone line required by the system and shall:

    - Co-ordinate and programme the telephone line supplier's works within the contract period taking into account testing and commissioning
    - Place orders and arrange for application forms and any test certificates to be completed
    - Monitor the telephone line supplier's progress and advise the Project Manager of any potential delays that arise
    - Provide attendances on the telephone supplier whilst he carries out his works, providing all necessary cable conduit, fixings, boards and enclosures to accommodate his equipment as may be necessary
    - Maintain the permanent land line for the duration of the works
    - Arrange for the transfer of responsibility for the telephone supplier's incoming service account to the Project Manager at the completion of commissioning

Cables

20. All cables shall be either HMWPE or have a minimum of a single layer of insulation and single layer of sheathing which shall confirm to IEC 60502. The selection of insulation and sheath shall take due account of the proposed installation and functional requirements. Cable to be installed in contact with anode material shall be suitable for long term exposure to acidic conditions, typically pH = 2 and those to be installed in concrete for long term exposure to alkaline conditions, typically pH = 13.

21. Cables shall be stranded copper and shall have a minimum of seven strands. They shall have a minimum of one layer of insulation of XLPE and an outer layer of PVC or be a single insulation of HMWPE.

22. Single core cables shall be coloured coded as detailed below and have a minimum of 14AWG cross sectional area. Cables shall be colour identified as follows:

Positive Cables	Red
Negative Cables	Black
Reference electrode	Blue
Reference connection to steel reinforcement	Yellow

Junction Boxes

23. Each Bay shall be supplied with a junction box, which shall provide a permanent facility for housing the electrical connections of the reference electrode signal and the DC system positive and system negative cables.

24. The junction boxes shall be constructed from non-metallic material with non-removable lid. The junction box shall be rated to NEMA 4X.

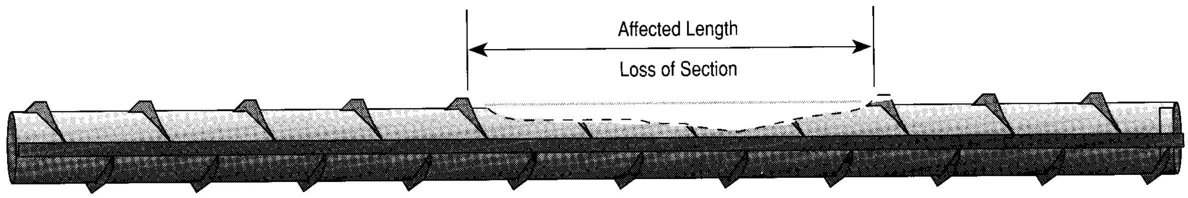
25. Cable entry shall be via cable glands or conduit and shall not compromise the IP rating of the box.

Concrete Repair

The following pages are inserted into the final revision.

TAMPA ELECTRIC COMPANY
DOCKET NO. 130040-EI
HUA'S THIRD REQUEST FOR PODS
FILED: AUGUST 21, 2013

Reinforcing Steel Repair (from Section Loss)

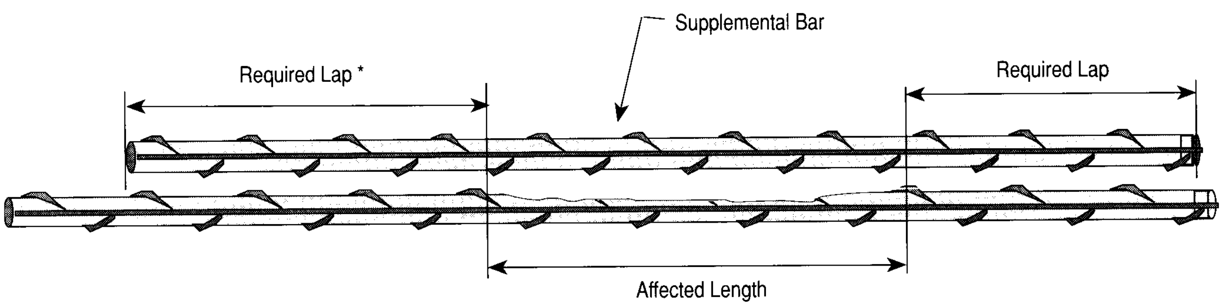


If the reinforcing steel has lost more than 25% of its cross section (or 20% if two or more adjacent bars are affected), then reinforcing steel repair is generally required.

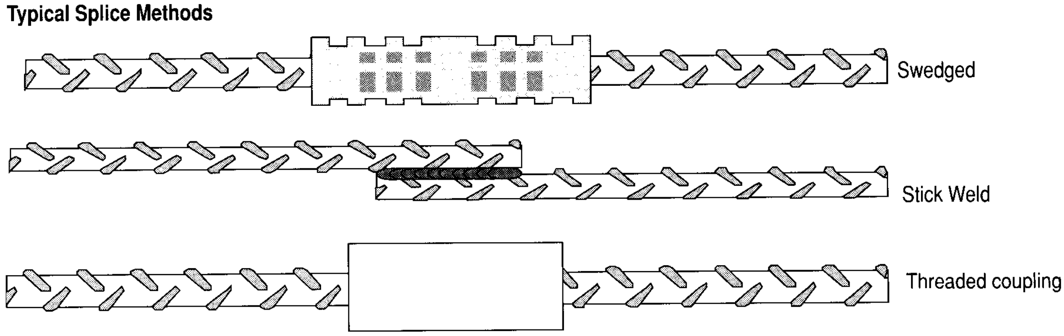
Note: When damage to reinforcing steel is uncovered, it is good practice to perform a structural review of situation.

If repairs are required for the reinforcing steel, one of the following methods should be used:

1. Supplemental bar over affected length. New bar may be mechanically spliced to affected bar or placed parallel to existing bar.
2. Complete bar replacement.



Typical Splice Methods



* Lap length shall be determined in accordance with ACI 318; also refer to AASHTO and CRSI Manuals.

Adapted from IACRS—*Surface Preparation Guideline* 3730, 31, 32, 34 dated 10/15/89.

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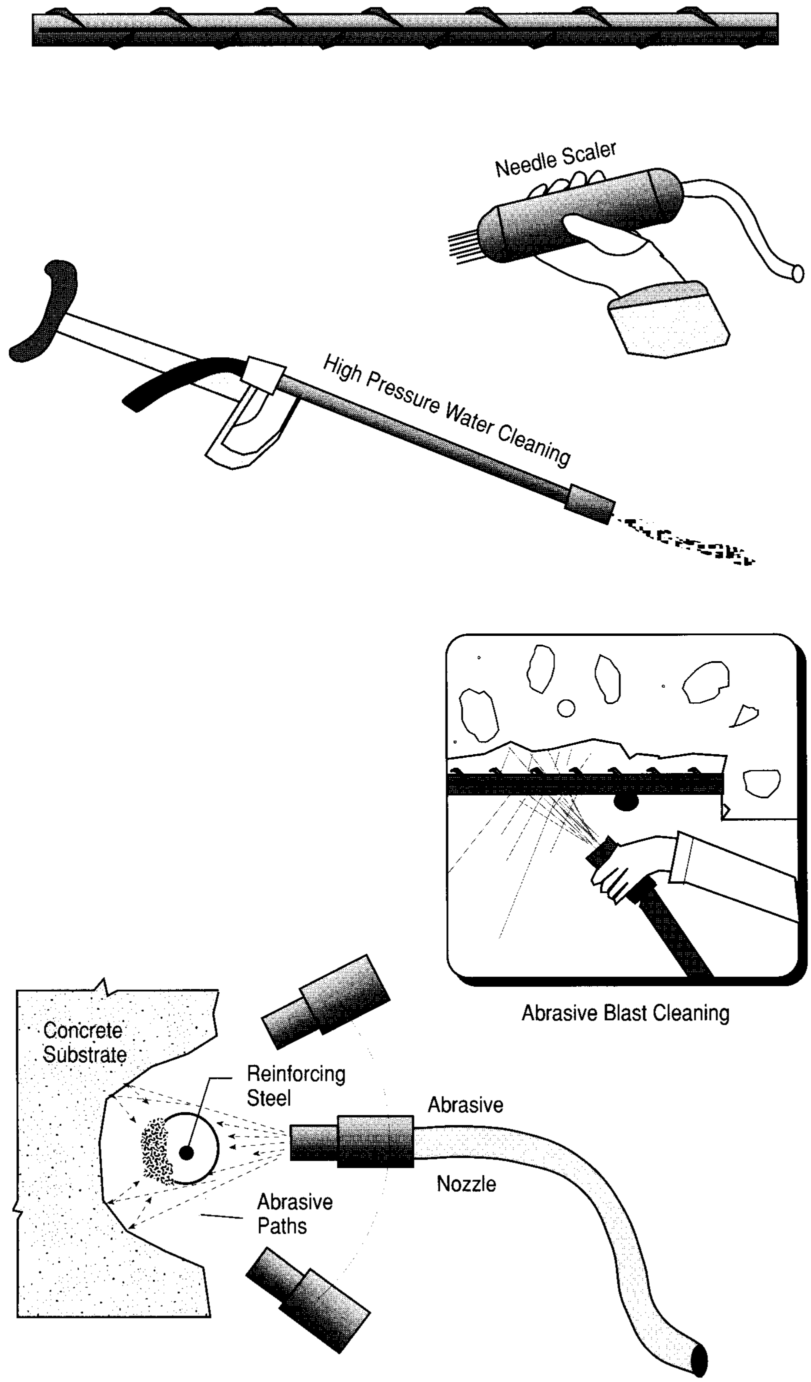
FILED: AUGUST 21, 2013

Reinforcing Steel Cleaning



Adapted from IACRS—*Surface Preparation Guideline* 3731, October 15, 1989.

General Procedure

All heavy rust and scale should be removed from the rebar to promote maximum bond with repair materials. A tightly bonded light oxide build-up may develop after cleaning. This is usually not detrimental to bond. If a protective coating is being applied to the rebar, the manufacturer's recommendations for surface preparation should be followed.

Needle Scalers

Needle scalers are pneumatic tools utilizing a group of small diameter steel rods powered by an internal piston. The steel rods hit the intended surface, causing removal of surface materials. Needle scalers are effective tools for removal of heavy oxide layers, as well as for surface cleaning of small areas of concrete.

High Pressure Water Cleaning

High pressure water (3,000 to 10,000 psi(20.7 to 69 MPa)) cleans concrete and steel surfaces, removing unsound materials. Water mixed with sand cleans faster and results in a roughened surface which will promote a better bond with coatings or with repair materials.

Abrasive Blast Cleaning

Abrasives mixed with pressurized air and projected through a nozzle are the best method of providing steel or concrete surfaces with a clean profiled surface. Airborne debris (dust) is an environmental concern when using this method. Water can be injected at the nozzle to reduce dust in this process.

Power Wire Brushing

A power wire brush is an effective tool for removing unwanted oxide from steel surfaces. Wire brushing is a very slow and ineffective operation when rebar has to be cleaned on the back side.

TAMPA ELECTRIC COMPANY
DOCKET NO. 130040-EI
HUA'S THIRD REQUEST FOR PODS
FILED: AUGUST 21, 2013

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RAP-3

Spall Repair by Low-Pressure Spraying

Reported by ACI Committee E 706

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The committee would like to thank Brandon Emmons for his illustrations in these bulletins.

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Structural Disclaimer
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TAMPA ELECTRIC COMPANY
DOCKET NO. 130040-EI
HUA'S THIRD REQUEST FOR PODS
FILED: AUGUST 21, 2013

ACI RAP Bulletin 3

FIELD GUIDE TO
CONCRETE REPAIR
APPLICATION PROCEDURES

Spall Repair by Low-Pressure Spraying

by Patrick "Doc" Watson

Introduction

Similar to wet-mix shotcrete but sprayed at a much lower velocity, low-pressure spall repair spray comes in the form of prepackaged mortar. The spray is applied using small concrete pumps or heavy-duty grout pumps to force the low-slump mortar through a hose. Air is added at the nozzle to impel the mortar. Bond with the prepared substrate is achieved through a combination of proper surface preparation, low-velocity impact, and the material properties of the prepackaged mortar.

Compared with either wet- or dry-mix shotcreting, this method allows the spray nozzle to be much closer to the repair surface. This means it can be used in tight spaces. Due to the viscous, sticky nature of the mixture and the low pressures involved, there is considerably less rebound than with high-velocity shotcreting.

Before any concrete repair is carried out, the cause of the damage must be assessed and the objective of the repair must be understood.

Typical causes of concrete deterioration include steel corrosion, sulfate attack, alkali-aggregate reactions (AARs), excessive deflection, and freeze-thaw damage. Poor practices during the original construction can lead to premature deterioration. Improper joint spacing and load imbalances also contribute to cracking and spalling.

What is the purpose of this repair?

Depending on the mortar mixture selected, low-pressure spray is used for surface repairs, structural repairs, or cosmetic renovation. The spray can be formulated for freeze-thaw durability, sulfate resistance, low permeability, and other desired or specified characteristics.

When do I use this method?

Low-pressure spray is typically used for vertical and overhead repairs. Successful applications have included structural repairs to bridges, bridge and building piers, structural slab undersides, tank walls (interior and exterior), stadiums, tunnels, and retaining walls. Structural repairs utilizing low-pressure spray are best done under the guidance of a qualified engineer.

The placement thickness can be 1/2 to 4 in. (13 to 100 mm) in a single lift. Thicknesses greater than 6 in. (150 mm) are possible in multiple lifts. If the repair application requires more than a 4 in. (100 mm) thickness, other methods may be more economical (see ICRI Guideline No. 03731, "Guide for Selecting Application Methods for the Repair of Concrete Surfaces").

The ingredients that make up the mortar vary widely, and the ingredients selected will depend on the specific repair situation. Formulas may contain ingredients such as corrosion inhibitors, air-entraining agents, and bonding additives.

The initial material costs are generally higher with this method than for a typical shotcrete application, but in-place costs are often lower or comparable because this method produces less rebound and requires less cleanup. Certified nozzle operators are not required.

How do I prepare the surface?

Consult the recommendations of ICRI Guideline No. 03732, "Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays," or ICRI Guideline No. 3730, "Surface Preparation for Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion."

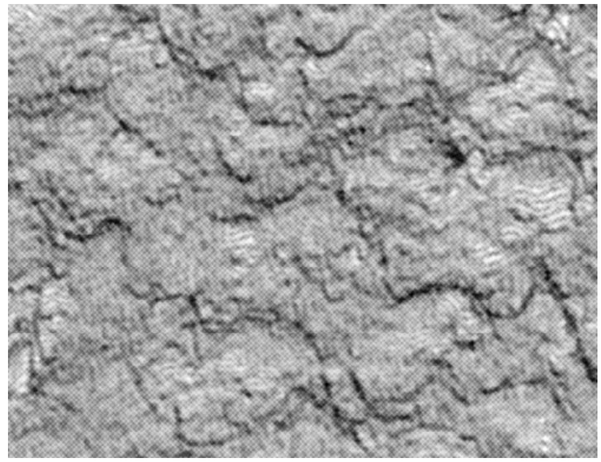

Fig. 1—Typical plus or minus 1/4 in. profile. (From ICRI Guide No. 03732 Profile No. 7). This is the standard recommended surface profile for low-pressure spray-applied mortars.

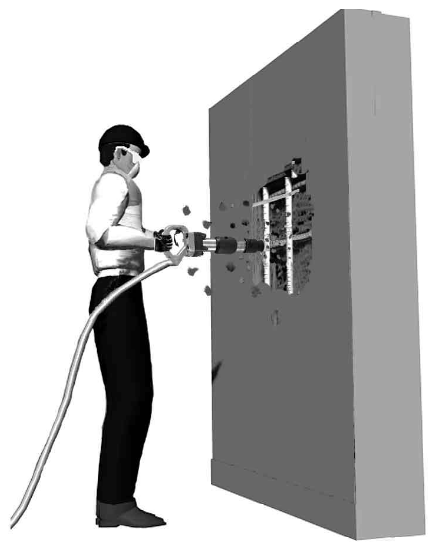

Fig. 2—Chipping surface.

Because many of the materials applied using low-pressure spray are prepackaged, the manufacturer's recommendations should also be consulted.

A typical roughness or profile recommendation for this repair method is ICRI Concrete Surface Profile (CSP) No. 7 or higher, as per ICRI Guideline No. 03730. A CSP No. 7 is equivalent to an amplitude of approximately 1/4 in. (7 mm) (see Fig. 1).

Factors that will influence the specific surface preparation include, but are not limited to:

- Desired roughness profile of the prepared surface (CSP);
- Method of preparation. These may include, but are not limited to, hydrodemolition, sandblasting, and use of pneumatic hammers (see Fig. 2);

Fig. 3—Presaturate prepared surface with water.

- Possible contamination of the surface by chemicals, oils, soaps, or carbonation. Test for carbonation with a pH indicator. The pH should be 11.5 or higher;
- Methods of treatment for contaminated surfaces.
- Required substrate saturation (see Fig. 3);
- Reinforcing requirements from the mortar manufacturer, the engineer, or the owner; and
- Treatment of existing cracks and joints: Repair the cracks? How? Fill the joints? If so, how and with what?

How do I select the right material?

Low-pressure spray-applied repair materials are proprietary, prepackaged cementitious products. Specifiers, applicators, and owners should consult ICRI Guideline No. 03733, "A Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces." Consult the American Concrete Institute publications on concrete repair by ACI Committee 546, Repair of Concrete. Refer to manufacturers' data sheets for material properties.

Physical property requirements such as bond strength, freeze-thaw durability, permeability, and flexural strength will vary from project to project.

When low-pressure spray-applied materials are used, some repair applications may require that the material be coated with a protective barrier system. When this is the case, confirm the required curing and drying time before installing the coating.

What equipment do I need?

Be sure that all necessary equipment and tools are on site and in proper working order. Have backup equipment or alternate methods planned and available. Typical equipment needed for low-pressure spray application of repair mortars includes, but is not limited to:

- Concrete or grout pumps suitable for low-pressure spray. Field experience has shown that ball valve pumps are not suitable. Short stroke, swing-type piston pumps or heavy-duty rotor-stator pumps perform well;
- If using a rotor-stator or "moyno" type pump, have a backup rotor on hand at the job site;

(a)

(b)

Fig. 4(a) and (b)—Application of mortar with pressure equipment and nozzle.

- Air compressor with pressure gages and controls. (Some pumps come equipped with built-in air compressors and controls.);
- A suitable mixer for mixing the mortar is needed. A backup mixer is recommended in case of breakdown. (Some repair type pumps come equipped with mortar mixers.); and
- A water measuring device, preferably a meter. (Many repair-type pumps equipped with mixers include built-in water meters.).
- A means of communication between the pump and mixer operators and the nozzleman.

Fig. 5—Skilled worker applying final finish to surface.

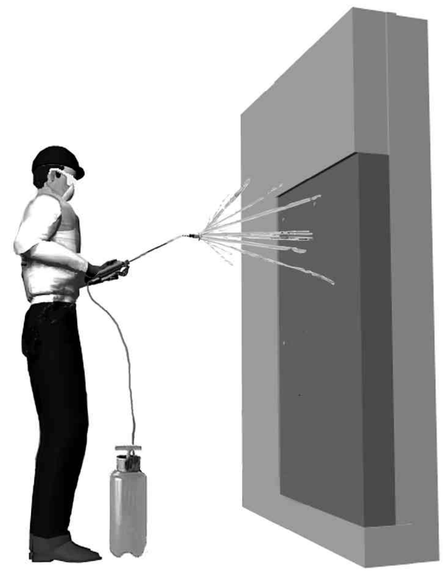

Fig. 6—Application of curing compound.

- All finishing, handling, and testing tools required by specification or good concreting practices.

What are the safety considerations?

Prepackaged mortars are hazardous materials and should be treated as such. Job-site safety practices should include, but are not limited to, the following where applicable:

- Material Safety Data Sheets (MSDS) should be on hand;
- Safety equipment: all machinery and equipment being used must have the correct safety equipment, guards, and warnings in place;
- Protective clothing: protective gloves for workers in contact with wet, cementitious material;
- Protective eyewear: safety glasses or face shields will be needed for all workers;
- Eyewash facilities should be provided;
- Respirators: dust masks will be needed for workers operating the material mixer;
- Ventilation of closed spaces: confirm that adequate ventilation is available before operating equipment that emits dangerous exhaust;
- Secured storage should be available for all hazardous materials;
- Fuel for equipment operation needs a safe storage area, well marked and visible; and
- A safety meeting with all involved should be held and led by the prime contractor's safety manager prior to beginning repair operations.

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Preconstruction meeting

Prior to proceeding with the repair, a preconstruction meeting is recommended. The meeting should include representatives from all participating parties (owner, engineer, contractor, materials manufacturer, etc.), and specifically address the parameters, means, methods, and materials necessary to achieve the repair objectives.

Repair procedure

1. The prejob (preconstruction) meeting agenda might include:

- On-site availability of power;
- On-site availability of water;
- Site accessibility;
- Debris removal and disposal;
- Dust, odor, and emissions control;
- Confirmation that all materials documentation is on hand—for example, MSDS sheets;
- Methods of curing and time required for curing;
- Possible emergencies and breakdowns—what to do if they occur;
- Finish requirements;
- Testing required; and
- All other concerns that could affect the progress of the repair.

2. Apply the repair.

- Inspect and approve the surface preparation. (See ICRI and ACI references);
- Presaturate the prepared substrate. Twenty-four hours is standard. Prepared saturated surface should be saturated surface-dry when the repair mortar is sprayed;

- Install the specified reinforcement;
- Install/apply bonding agents, corrosion inhibitor sacrificial anodes, if called for;
- Mix the repair mortar and load it into pump hopper.
- Begin spray operations with pump, compressor, and suitable spray nozzle; and
- Apply the mortar at the thickness recommended by the material manufacturer. Most low-pressure spray materials require application in lifts when the thickness of the total application layer exceeds 3 in. (76 mm).

3. Finish the repair.

Confirm the final finished appearance of the repair with the owner. This may vary from rough as-sprayed to smooth troweled. If smooth troweling is specified, production may be reduced and additional labor may be required. One nozzle operator may require multiple finishers to keep pace. This will be influenced by such factors as:

- The installed thickness of the material being applied;
- The drying conditions caused by ambient and substrate temperatures;
- The setting characteristics of the repair mortar; and
- Whether the repair is vertical or overhead.

Because of the non-bleeding, sticky nature of these materials, use of an evaporation control film when finishing is recommended.

Proper curing is important and should be conducted in accordance with ACI 308.1-98, “Standard Specification for Curing Concrete.” Additional curing information is available from ACI 308R-01, “Guide to Curing Concrete.”

For most cementitious low-pressure spray-applied mortars, application of a curing compound that complies with the moisture retention requirements ASTM C 309 is satisfactory. As an alternate, moist cure for 7 days.

Always refer to the mortar manufacturer’s instructions for specific curing methods and materials recommenced for the product selected.

How do I check the repair?

Requirements may include:

- Before and after photos;
- Confirmation of acceptable surface preparation. This can include the prepared surface profile and the pH of the prepared surface. A pH of 11.5 or greater is recommended;
- Confirm repair depth;
- Material property tests performed by a qualified testing agency. This is usually done to confirm the material manufacturers’ published material properties;
- In-situ direct tensile strength tests of the prepared surface;
- In-situ bond direct tensile tests of the hardened, cured repair; and
- Confirmation that all materials used were as specified, cross checking material purchase orders with quantities estimated and with actual quantities billed.

Sources for additional information

ACI Committee 308, 1998, “Standard Specification for Curing Concrete (ACI 308.1-98),” American Concrete Institute, Farmington Hills, Mich., 1998, 9 pp.

ACI Committee 308, 2001, “Guide to Curing Concrete (ACI 308R-01),” American Concrete Institute, Farmington Hills, Mich., 2001, 31 pp.

ACI Committee 503, 1993, “Use of Epoxy Compounds with Concrete (ACI 503R-93),” Appendix A, Farmington Hills, Mich., 28 pp.

ACI Committee 506, 1995, “Guide to Shotcrete (506R-90 (Reapproved 1995)),” American Concrete Institute, Farmington Hills, Mich., 41 pp.

ACI Committee 546, 1996, “Concrete Repair Guide (ACI 546R-96),” American Concrete Institute, Farmington Hills, Mich., 1996, 41 pp.

“Guide for Selecting Application Methods for the Repair of Concrete Surfaces,” *ICRI Guideline* No. 03731, 1996.

“Guide for Selecting and Specifying Materials for Repair of Concrete Surfaces,” *ICRI Guideline* No. 03733, 1997.

“Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Membranes,” *ICRI Guideline* No. 03732, 1997.

“Surface Preparation for Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion,” *ICRI Guideline* No. 03730, 1995.

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RAP-6

Vertical and Overhead Spall Repair by Hand Application

Reported by ACI Committee E 706

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ACI RAP Bulletin 6

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FIELD GUIDE TO
CONCRETE REPAIR
APPLICATION PROCEDURES

Vertical and Overhead Spall Repair by Hand Application

by J. Christopher Ball

Introduction

One of the most common application methods for repairing concrete is by hand troweling mortars. This method can be used to repair spalled or deteriorated concrete (Fig. 1) or to resurface vertical, overhead, and horizontal concrete surfaces. Applying repair materials by hand does not require significant equipment and is ideal for shallow surface repairs, especially in areas with limited or difficult access. While both portland cement-based and resin-based repair mortars have been used for trowel-applied vertical and overhead repairs, this field guide focuses on the application of portland cement-based repair materials.

Before any concrete repair is initiated, the root cause of the damage should be determined with a thorough condition survey of the structure. Typical causes of concrete damage can include corrosion of embedded metals from exposure to chloride ions from deicing salts or sea spray in coastal areas; disintegration from freezing-and-thawing cycles when the concrete is saturated with water; or deterioration from chemical attack. Understanding the cause of the deterioration, the owner's repair objectives, and the in-service environment of the concrete structure will help in the proper selection of repair materials and application methods.

The technique of hand troweling repair mortars requires the selection of a sag-resistant mortar and attention to detail during application to achieve an adequately consolidated repair that is well-bonded to the concrete substrate. The successful installation is a function of good surface preparation, application techniques, curing procedures, and properties of the repair material selected.

What is the purpose of this repair?

Hand-applied repair mortars replace damaged concrete and are generally recommended for thin repairs that are cosmetic in nature. Thin overlays of mortar can also be applied to renovate deteriorated vertical and overhead concrete surfaces. This technique, when properly executed, improves the appearance of the deteriorated structure and provides additional protection to the concrete surface.

When do I use this method?

Structural repair projects generally require other repair methods such as form and cast-in-place, grouted pre-placed aggregate repair, or shotcrete. Experienced workers using wood floats, sponges, or steel trowels can achieve a variety of finishes with trowel-applied mortars. Hand application has been used to repair vertical and overhead surfaces including walls, columns, beams, soffits, and building facades.

Placement thickness can vary depending on the type of materials selected and the size, depth, and orientation of the repair cavity. Placement thickness can range from 1/8 to 2-3/4 in. (3 to 70 mm) on vertical surfaces, and 1/8 to 1 in. (3 to 25 mm) on overhead surfaces in a single layer. Deeper placements may require repair material to be placed in additional layers.

How do I prepare the surface?

The recommended steps in properly preparing the surface to receive a hand-applied mortar are as follows:

Fig. 1—Concrete delamination.

Fig. 2—Bulk concrete removal.

1. *Bulk concrete removal and edge conditioning*— Loose, delaminated concrete should be removed until the substrate consists of sound concrete (Fig. 2). Where corrosion of the reinforcement exists, continue bulk removal along the reinforcing steel and adjacent areas with evidence of corrosion-induced damage that would inhibit bonding of repair materials. Bulk concrete removal should include undercutting the corroded reinforcing steel by approximately 3/4 in. (19 mm). The shape of the prepared cavity should be kept as simple as possible–generally square or rectangular in shape. The edges of the patches should be sawcut perpendicular to the surface to a depth of 1/2 in. (13 mm) to avoid feather edging the repair material (Fig. 3).

2. *Final surface cleaning*— Use abrasive blasting (Fig. 4) to remove residual dust, debris, fractured concrete, and contaminants that prevent proper bonding. If abrasive blasting is not feasible, pressure washing using a minimum 3000 psi (250 MPa) may be acceptable depending on the bond strength required. Blowing with oil-free compressed air or alternately, the use of a vacuum, may be appropriate if dust is still present after the blasting. The final surface texture should be rough, with approximately a 1/4 in. (6 mm) amplitude (Fig. 5);

Fig. 3—Edge conditioning.

3. *Treatment of exposed reinforcement*—Bond-inhibiting corrosion should be removed from the reinforcing steel by an abrasive blasting wire wheel or needle scaler. If the cross-sectional area of the reinforcing steel has been significantly reduced, a structural engineer should be consulted. If a reinforcing steel coating has been specified, apply the coating after the reinforcing steel has been cleaned (Fig. 6).

4. *Substrate saturation*—Most portland cement-based materials require the base concrete to be in a saturated, surface dry (SSD) condition prior to application to prevent a rapid loss of moisture from the repair material and into the substrate. An SSD condition is achieved when the body of the concrete is saturated and free surface water and puddles have been removed from the surface of the concrete. An SSD surface is not recommended if a polymer bonding agent is to be used. When using polymer bonding agents, follow the manufacturer's recommended surface preparation requirements. The general recommendations previously given may be influenced by several factors, including:

- Desired roughness profile of the prepared surface (This may be specified by the manufacturer of the repair product);
- Method of surface preparation, including chipping hammers, abrasive blasting, high-pressure water-blasting, or hydrodemolition;
- Possible contamination of the surface by chemicals, oils, or grease; possible carbonation; and methods of removing contaminants or carbonated concrete;
- Repair material manufacturer's recommendations (Ask for technical data sheets and installation bulletins and read the printed instructions on the packaging.); and
- Treatment of existing cracks and joints.

For additional information, consult the recommendations of the International Concrete Repair Institute (ICRI) Guidelines No. 03732, "Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Overlays," or No. 03730, "Guide for Surface Preparation for Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion."

How do I select the right material?

Hand- or trowel-applied repair materials are generally proprietary, prepackaged, cementitious products. Portland

Fig. 4—Final surface cleaning.

Fig. 5—Properly prepared surface.

cement-based materials designed for hand application may also include polymers, silica fume, shrinkage-compensating materials, and other additives for enhanced physical properties and improved handling.

Specifiers, applicators, and owners can consult ICRI Guideline No. 03732, "Selecting and Specifying Materials for Repair of Concrete Surfaces," for a useful checklist for prioritizing desired material properties. Manufacturers' technical data sheets should be consulted for material properties.

The physical property requirements such as drying shrinkage, permeability, freezing-and-thawing resistance, and mechanical properties vary from project to project depending on the expected service conditions. The properties critical to the long-term success of the repair should be determined during the evaluation phase and be specified.

Other factors that may influence the selection of repair materials include desired application thickness, rate of strength gain, ease of application, color, and in-place cost.

For some hand-applied repairs, sealers or decorative or protective coatings may be used to provide additional protection to the base concrete, to enhance aesthetics, or both. When this is the case, confirm the required curing and drying

Fig. 6—Treatment of exposed reinforcement.

Fig. 7—Typical equipment to mix materials.

time (or maximum moisture content) with the sealer or coating manufacturer before application commences. For more information, consult ACI 515.1R, "Guide to Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete."

What equipment do I need?

Typical equipment needed for hand-applied repair mortars includes:

- A suitable mixer unit such as a drill/paddle/pail combination for small repairs (Fig. 7), or paddle-type mortar mixers for larger applications;
- Air compressor, sawcutting equipment, blades, abrasive blast equipment;
- Water-measuring device to ensure that proper amounts of mixing water are used; and
- Finishing, handling, and testing tools required by the specification or good concreting practices.

Be sure that necessary equipment and tools are on site and in proper working order. Have backup equipment or alternate methods planned and available.

What are the safety considerations?

Concrete repair mortars are hazardous materials and should be treated as such. Job-site safety practices should include the following where applicable:

- Applicable material safety data sheets (MSDS) should be on hand;
- Machinery and equipment used must have the correct safety guards and warnings in place;
- Workers should wear protective gloves and other clothing needed to prevent skin contact with wet, highly alkaline cementitious materials;
- A face shield or safety glasses are needed to provide eye protection;
- Eye wash facilities should be available on the job site.
- Dust masks are needed for workers operating or working near the material mixer and forced-air respirators used for abrasive blasting;
- Hearing protection must reduce sound levels reaching the inner ear to limits that are specified by the United States Occupational Safety and Health Administration (OSHA); and
- Confirm that adequate ventilation is available in closed spaces before operating equipment that emits dangerous exhaust fumes.

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Preconstruction meeting

Prior to proceeding with the repair, a preconstruction meeting is recommended. The meeting should include representatives for the owner, engineer, contractor, materials manufacturer, and any other parties needed to explain the means, methods, and materials necessary to achieve the repair objectives. See ICRI Guideline No. 03733, "Guide to Selecting and Specifying Materials for Repair of Concrete Surfaces."

Repair procedure

1. *Apply the repair material.*

- Mix the material following the manufacturers' recommendations;
- Scrub a thin bond coat of the repair mortar into the SSD substrate, thus filling pores to ensure intimate contact and to help prevent sloughing or sagging of repair materials on vertical and overhead surfaces. Alternatively, apply a bonding agent if required by the manufacturer or the repair specification;
- Apply the material with adequate pressure before the bond coat dries (Fig. 8). Thoroughly consolidate the repair material into the corners of the patch and around any exposed reinforcement in the repair zone. Full encapsulation of the reinforcement is important for

Fig. 8—Hand application of repair material.

Fig. 9—Spray application of curing compound.

long-term durability; and

- If a second lift is required, thoroughly roughen the surface of the first lift by scoring the soft mortar to achieve an aggressive finish, similar in profile to the prepared concrete substrate. This process will promote additional mechanical bond between lifts. If the second lift will not be immediately applied, keep the first lift moist until application of the second lift. After the first lift has reached final set, moisten the surface of the first lift, scrub in a thin layer of fresh mortar, and apply the second lift of material. Once the desired thickness has been achieved, strike off level with the adjacent concrete.

2. *Finish and cure the repair.*

- Finish the repair material to produce a final finished appearance as required by the project specifications. Because of the nonbleeding, "sticky" nature of many of these materials, the use of an evaporation control film may be helpful; and
- As with all portland cement-based materials, proper curing will provide enhanced physical properties. Good curing procedures prevent rapid moisture loss at early ages. Consult the product manufacturer for curing instructions. Curing will generally be conducted in accordance with ACI 308R, "Guide to Curing Concrete." The use of curing compounds (Fig. 9) that comply with the moisture retention requirements of ASTM C 309, or moist curing are common curing methods.

How do I check the repair?

Requirements may include:

- Before and after photos;
- Confirmation of acceptable surface preparation. This may include observing the surface amplitude profile. Alternatively, direct tension testing of the prepared surface will provide quantitative data regarding the level of surface preparation achieved;
- Material testing performed by a qualified testing agency;
- Sounding the cured repair for delaminations;
- In-place direct tensile bond testing of the hardened, cured repair to the base concrete using methods similar to those described in ICRI Technical Guideline No. 03739, "Guide to Using In-Situ Tensile Pull-Off Tests to Evaluate Bond of Concrete Surface Materials," published by the International Concrete Repair Institute. Important observations include maximum stress, expressed in psi or MPa, and failure mode (base concrete, bond line, or cohesive failure of the mortar).

References

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RAP-5

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Surface Repair
Using Form-and-Pump Techniques

Reported by ACI Committee E 706

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The committee would like to thank Brandon Emmons for his illustrations in these bulletins.

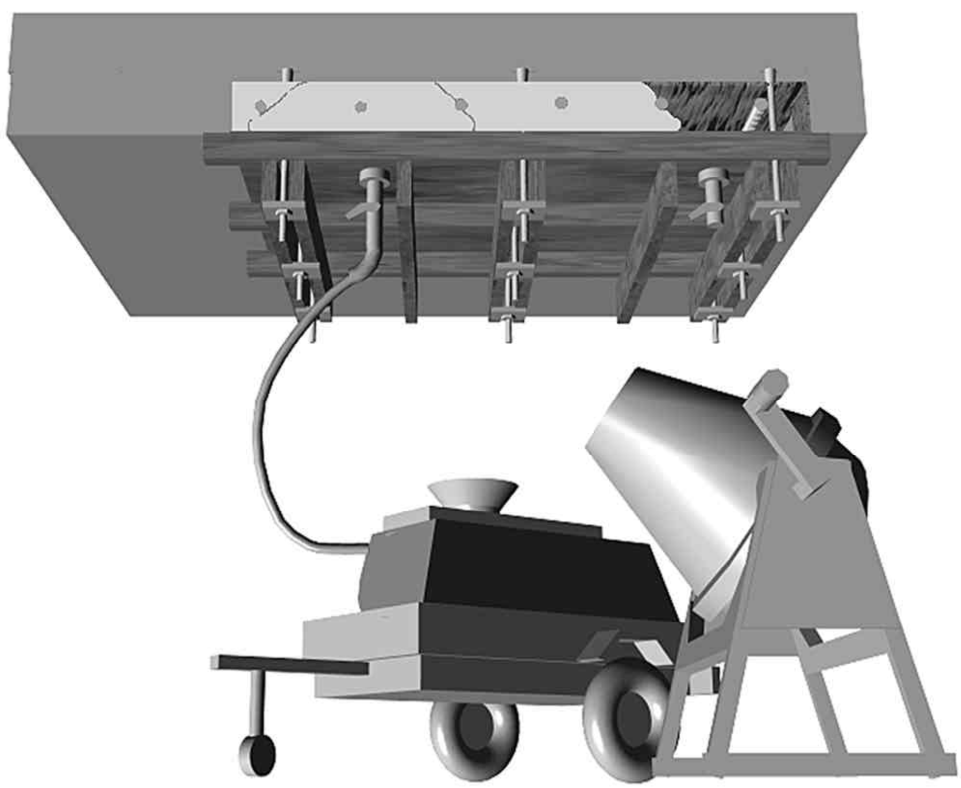
ACI Repair Application Procedure 5.

ACI RAP Bulletin 5

FIELD GUIDE TO
CONCRETE REPAIR
APPLICATION PROCEDURES

Surface Repair Using Form-and-Pump Techniques

by Peter Emmons



Introduction

The form-and-pump repair technique is a multi-step process of preparing and constructing formwork, and pumping repair material into the cavity confined by formwork and existing concrete. The form-and-pump technique allows the use of many different repair materials. The necessary requirement for material selection is pumpability. Various pumps are used, depending on the mixture design with focus on aggregate size. Prior to construction of formwork, any surfaces that may cause air to become trapped during the pumping process must be trimmed, or vent tubes installed.

Repair materials are mixed and pumped into the confined cavity. The sequence of pumping is from low points to high points and when performed overhead, from one extremity to the other. Large areas may require bulkheading to separate placements into manageable areas. When the cavity is full, pump pressure is exerted on the form, causing the repair material to consolidate and make intimate contact, and effect bonding with existing concrete surfaces. The form-and-pump technique offers many advantages to alternative techniques, such as shotcrete, hand placement, and preplaced aggregate.

Advantages include:

- Placement is not limited by thickness of repair or by size or density of exposed reinforcement;
- Repair materials are premixed and placed to provide a uniform cross section without segregation or intermediate bond lines;
- Sagging or dropouts of freshly placed materials aren't problems; all materials are supported by formwork during the placement and curing process;
- The pressurization process consolidates the repair material, providing for full encapsulation of exposed reinforcing steel;
- The formwork protects the repair material during the curing process;
- The process is less subject to individual operator error; and
- Quality assurance of the in-place repair is easier to provide.

What is the purpose of this repair?

The primary purpose for this type of repair is to restore the structural integrity, concrete cover requirements, or both, for the damaged element.

When do I use this technique?

This technique is commonly used on vertical surfaces such as walls, columns, and other combinations such as beam sides and bottoms. Separate bonding agents such as grouts or epoxy are not commonly used with this technique. It is highly recommended that for each project, a trial installation be performed to verify the preparation, material, and placement technique using quality-control procedures outlined at the end of this document.

How do I prepare the surface?

Regardless of the repair method, surface preparation is essentially the same. Concrete is removed until good quality concrete is located. Exposed bars are undercut, and surfaces are cleaned with high-pressure water or are abrasively blasted.

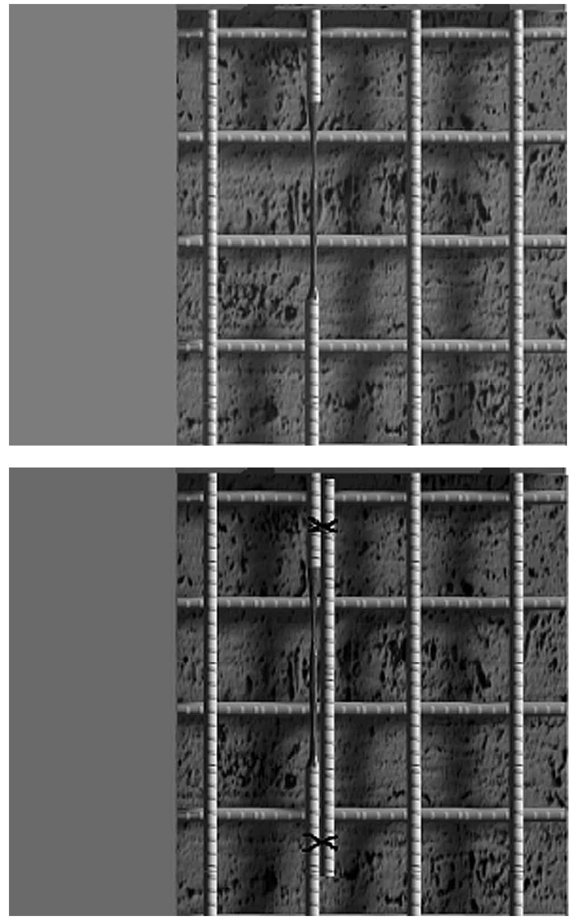


Fig. 1—Lapping of supplemental reinforcing.

With form-and-pump techniques, it is important to understand how the existing surfaces will permit the repair material to penetrate and flow. Surfaces that might trap air need to be trimmed, or vent pipes may be provided in the formwork. Profile roughness from hand-chipping or hydrodemolition is not generally a problem for entrapping air. Flow of the repair material (while flowing within the formed cavity) will most likely remove air from the profile.

Steps in surface preparation include the following:

Step 1—Sounding or other appropriate nondestructive concrete testing to locate areas of delamination.

Step 2—Marking of the perimeter of the repair area. Layout should be simple square or rectangular shapes. There should be no acute angles between boundary lines defining the repair area.

Step 3—Removal of concrete with a 15-lb chipping hammer. Hammers larger than a 15-lb class may cause damage to substrate and reinforcement.

Step 4—Sawcutting perimeter of the repair. Note: sawcut should not be deeper than the cover over the reinforcement.

Step 5—Reinforcement repair. When reinforcing steel is heavily corroded and the diameter is reduced, consult a structural engineer for repair procedures. For many applications, supplemental reinforcement can be lapped to adjacent damaged bars, as shown (see Fig. 1).

Step 6—Cleaning of reinforcing steel and concrete with abrasive blasting.

How do I select the proper repair material?

Constructibility requirements for materials used with the form-and-pump technique are limited only by their ability to be pumped and their flow characteristics. More important than constructibility are the materials' in-place properties, such as low drying shrinkage, compatible strength, thermal and elastic properties, and durability requirements. While constructibility of the repair materials requires good pumpability and flowability, these required characteristics should not sacrifice the requirement of low drying shrinkage. Drying shrinkage can cause cracking, delamination, inability to carry loads, and reduced durability. Pumpability and flowability can be brought to the material with aggregate shape and chemical admixtures that preserve low water-cement ratio, yet provide a pumpable mixture. Prepackaged repair materials, which are designed for pumping and incorporate shrinkage-compensating additives, are appropriate for many applications. Materials should be reviewed for effects on drying shrinkage to find those with low shrinkage.

Shrinkage testing in accordance with ASTM C 157 (modified in accordance with ASTM C 928 and measured over a 120-day period) will provide meaningful and comparable shrinkage properties.

What equipment do I need?

Pumping equipment—Pumping equipment is generally matched to the type of repair material and the size of the repair project. The specified repair material requires pumping through a pump line to the formed cavity. Cementitious repair materials have various aggregate contents and aggregate sizes. Fine-grained repair mixtures with very fine aggregate and little or no coarse aggregate can be pumped with mono-type pumps or piston/ball valve pumps.

Repair materials with large aggregates (larger than 3/8 in. [10 mm]) are best pumped with hydraulic/swing valve pumps. All pumping equipment must have adequate controls to regulate flow rates and pressures.

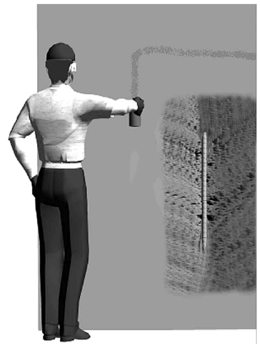
STEP 1

Sounding of concrete to locate areas of delamination.

STEP 2

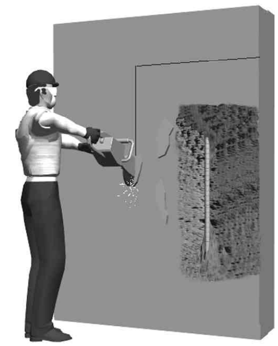
Unsound concrete removed with 15-lb chipping hammer. Hammers larger than 15-lb class may cause damage to substrate and reinforcement.

STEP 3



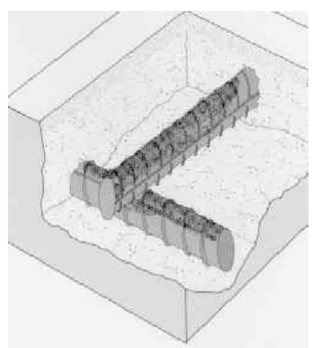
Mark perimeter of repair area. Layout should be simple geometric shapes.

STEP 4



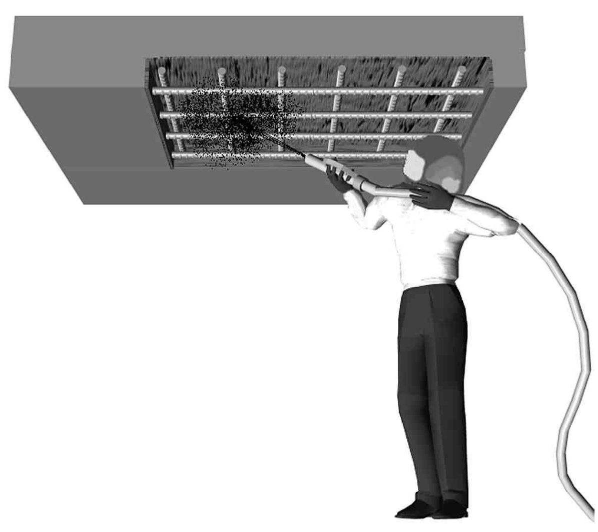
Sawcutting perimeter of repair. Note: sawcut should not be deeper than cover over reinforcement.

STEP 5: *Reinforcement repair. When reinforcing steel is heavily corroded and the diameter is reduced, consult a structural engineer for repair procedures. For many applications supplemental reinforcement can be lapped to adjacent damaged bars, as shown.*

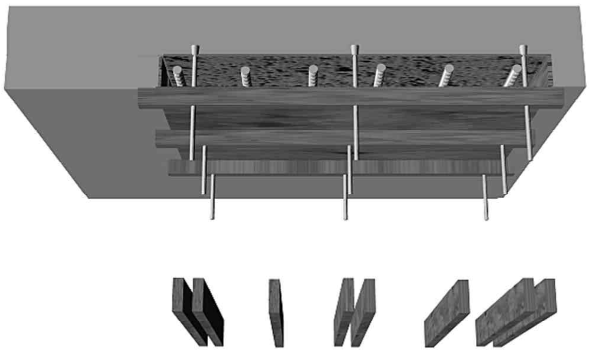


Important Note*: If corroded reinforcing bars are encountered in the preparation process, then concrete surrounding the bars must be removed to fully expose the full circumference. Clearance under the bar should not be less than 3/4 in. (19 mm), or 1/4 in. (6 mm) greater than the largest aggregate size of the repair mixture, whichever is greater.*

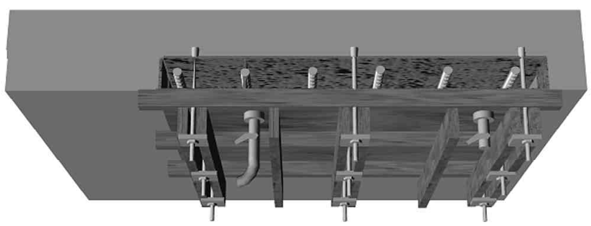
STEP 6



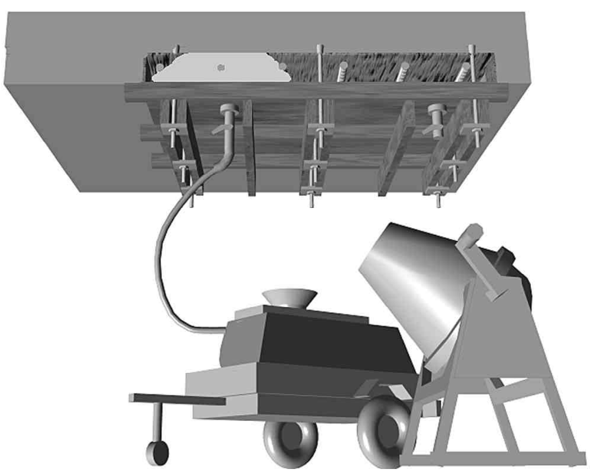
Cleaning of reinforcing steel and concrete with abrasive blast.



Erection of formwork with embedded anchors supporting formwork

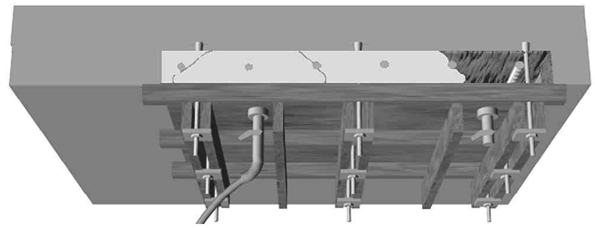


Formwork complete ready for pumpline hookup

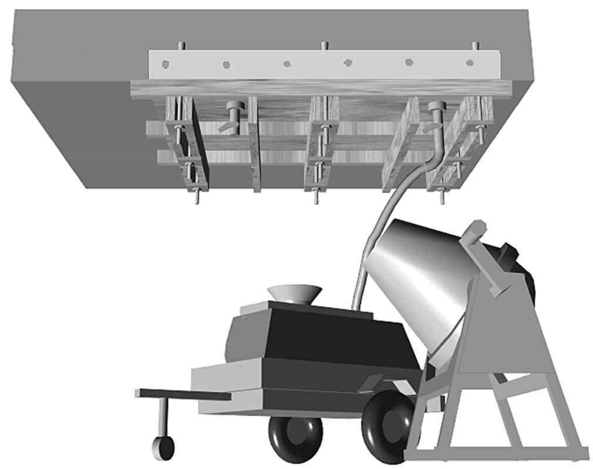


Pumpline hooked to form and pumping of repair material begins

Fig. 2.



Pumping underway. Note flow path radiating from pump line port.



Cavity is filled and pressurized. Pump line is shut down.

Fig. 3.

What are the safety considerations?

Job site safety practices include, but are not limited to, the following where applicable:

- Material Safety Data Sheets (MSDS) available;
- Protective clothing worn by workers handling or exposed to hazardous materials;
- Use of protective eyewear during pumping and preparation;
- Availability of eye wash facilities; and
- Use of respirators during preparation.

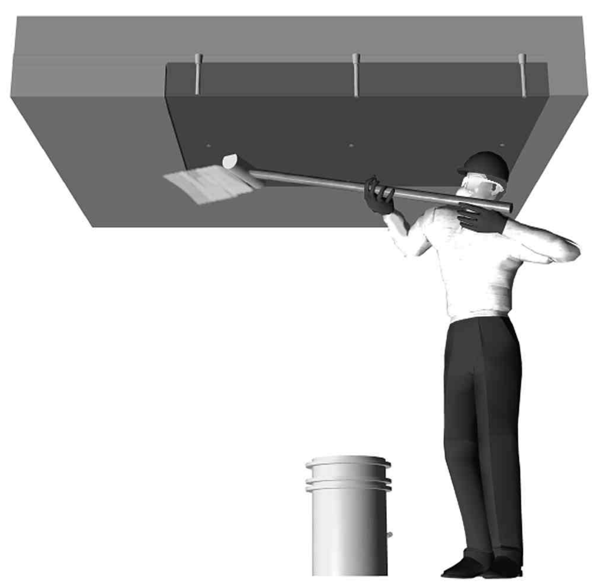


Fig. 4—Immediately after stripping of formwork, curing compound is either rolled or spray-applied.

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Preconstruction meeting and trial repair

Prior to proceeding with the repair, a preconstruction meeting is recommended. The meeting should include representatives from all participating parties (owner, engineer, contractor, materials manufacturer, etc.), and specifically address the parameters, means, methods, and materials necessary to achieve the repair objectives. Trial repairs using the proposed procedure and materials are highly recommended.

Repair procedure

Formwork construction—Formwork must accommodate the mass and pressure of the repair material. Design of the forms should follow standard practice for cast-in-place concrete construction except for the calculation of form pressure. Form pressure should be designed for a minimum of 14 psi (100 kPa). Maximum pressure exerted on formwork occurs after the formwork cavity is full and pressurized. Formwork is best attached directly to the concrete surface with expansion anchors or standard form ties. All anchors should be preloaded to prevent slippage during placement. In some applications, shoring or scaffolding can be used to support the formwork. Forms should be constructed to fit tightly against existing concrete surfaces. Preformed foam gaskets or cast-in-place foam works well to address difficult-to-match surfaces. Attachment of the pump hose to formwork is achieved with various techniques, including the use of plumbing fittings with flanges and ball valves or the use of a pump-line attachment with hand-held friction fit-insertion followed by wooden plugs.

Pumping procedure—The sequence of material placement into the formed cavity depends on the geometries involved. Vertical surfaces start at the lowest point, filling in a manner that prevents air entrapment. Arrangement of ports for pump line attachments is usually horizontal with spacing of 3 to 4 ft (900 to 1200 mm) in grid form. Pumping continues even

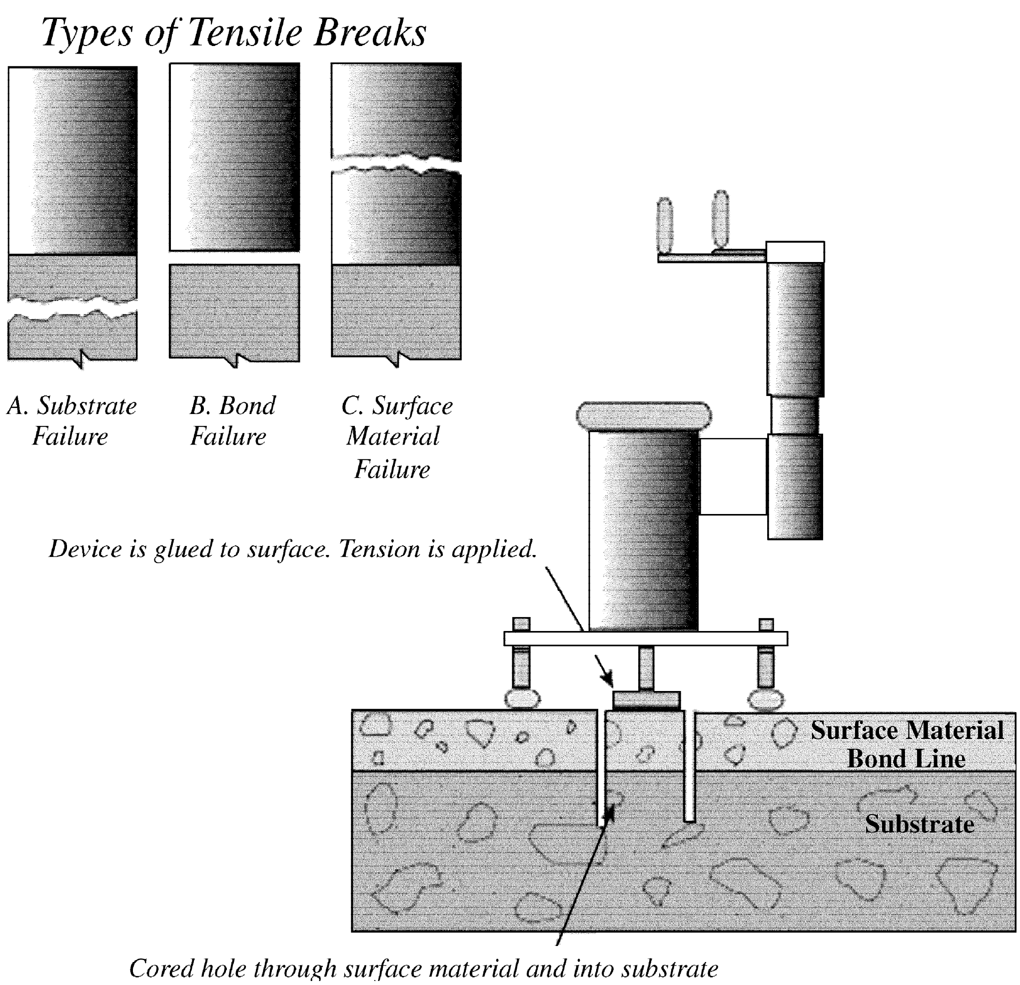
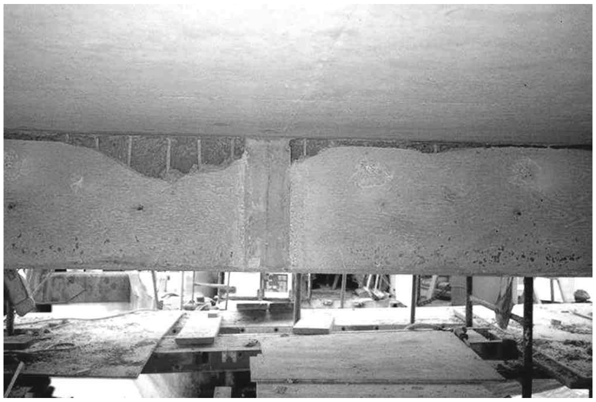


Fig. 5—Test procedure.



Effects of not filling the cavity.

after material flow occurs from adjacent ports to expel air. When the flow is without intrusion of air, the pump is temporarily shut off, the port closed off, and the pump line connected to the adjacent port that has seen flow. The sequence is continued until the cavity is filled. In some conditions, the cavity can be pumped from one port. In this situation, each adjacent port is capped off as flow occurs. It is necessary to monitor pump-line pressure to prevent excessive backpressure when pumping long distances. Once the cavity is filled, the full-line pressure is available to pressurize the formed cavity.

Care must be exercised in the final pressurization because the excessive pump-line pressure (hydraulic pumps can exert in excess of 800 psi [5 MPa]) may cause the form to fail. In most applications, pressure gages should be attached to the pump line near the exit port to monitor cavity pressure. If the formwork fails due to overpressurization, the failure will generally occur as a slight movement in a form panel seam or perimeter seal. The failure is not explosive because there is no significant stored energy. Overhead placements are accomplished by starting at an extremity of the surface and proceeding in a fashion similar to vertical placements. Material will flow radially from the injection port to adjacent ports. Repairs involving soffit and vertical faces of members can be combined into one placement. In this case, placement begins at the lowest elevation and follows the procedure detailed above for each orientation. Large areas of repair should be sectionalized utilizing bulkheads. Bulkheads can be constructed of repair material and left in place. Utilizing bulkheads and manageable placement volumes limits the risk of problems associated with large placements and allows pressurization to occur within shorter durations of material mixing.

How do I check the repairs?

After stripping of forms various tests can be performed to confirm the placement of repair material has achieved complete consolidation and intimate contact with the substrate to achieve bond. A uniaxial bond test can be performed by drilling through the repair into the substrate. A bonded plate attached to the core is pulled until rupture occurs. Bond values should exceed 100 psi (0.7 MPa), and in most cases exceed 150 psi (1 MPa). These tests are performed in accordance with ACI 503R Appendix (see Fig. 5).

The complete repair area should also be hammer-sounded or evaluated by other non-destructive methods to determine overall integrity. Any hollow sounds may represent poor bond or voids.

Sources for additional information

1. "Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforced Steel Corrosion," No. 03730, International Concrete Repair Institute.

2. "Guide for Selecting and Specifying Concrete Repair Materials," No. 03733, International Concrete Repair Institute.

3. ACI Committee 347, "Guide to Formwork for Concrete (ACI 347-01)," American Concrete Institute, Farmington Hills, Mich., 32 pp.

4. ACI Committee 546, "Concrete Repair Guide (546R-96)," American Concrete Institute, Farmington Hills, Mich., 41 pp.

5. ACI Committee 503, "Use of Epoxy Compounds with Concrete (503R-93 (Reapproved 1998))," American Concrete Institute, Farmington Hills, Mich., 1998, 28 pp.

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Surface Repair Using Form-and-Pour Techniques

Reported by ACI Committee E 706

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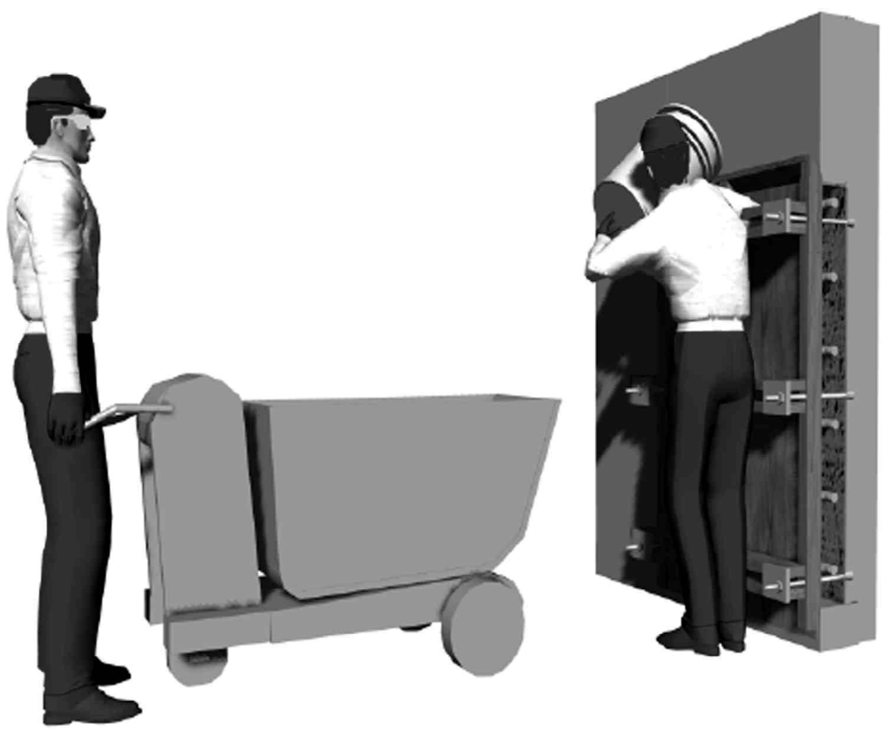
ACI RAP Bulletin 4

FIELD GUIDE TO
CONCRETE REPAIR
APPLICATION PROCEDURES

Surface Repair Using Form-and-Pour Techniques

by Peter Emmons



Introduction

The form-and-pour placement technique is a multistep process of preparation, formwork construction, and placement of repair materials. Repair materials are placed in the cavity between the formwork and the prepared substrate with buckets, pumps, chutes, or buggies. The form-and-pour technique allows the use of many different castable repair materials. Placeability is the primary consideration material selection. Depending on the consistency of the repair material, consolidation is accomplished by vibration, rodding, or when the material has extremely high slump (self consolidating), no additional steps may be required.

What is the purpose of this repair?

The primary purpose of this type of repair is to restore the structural integrity, or concrete cover requirements, or both, for the damaged element.

When do I use this technique?

This technique is commonly used on vertical surfaces such as walls, columns, and other combinations such as beam sides and bottoms. When used to repair slab soffits, the repair material is typically placed through holes or openings cut through the slab. Adhesive bonding agents or grouts are not commonly used with this technique. A trial installation is highly recommended for each project, to verify the preparation, material, and placement technique using quality-control procedures outlined at the end of this document.

The form-and-pour technique offers many advantages:

- Many different types of repair materials can be used;
- Repair material can be placed around reinforcing steel; and
- Formwork protects against early-age drying that promotes cracking.

The primary limitation of the form-and-pour technique is that formwork installation makes it more labor-intensive than alternative placement methods such as shotcrete or hand application (see Fig. 1).

How do I prepare the surface? (Fig. 2)

Regardless of the repair method, surface preparation is essentially the same. Concrete is removed until sound concrete is located. Exposed bars are undercut, and surfaces are cleaned with high-pressure water, or are abrasively blasted. With form-and-pour techniques, it is important to understand how the existing surfaces will permit the repair material to penetrate and flow. On partial-depth vertical repairs, the upper edges of vertical surfaces should be trimmed to eliminate potential pockets of entrapped air and promote complete filling from the location of the chute. Refer to page 3 for step-by-step preparation procedures.

Step 1—Sound the concrete to locate areas of delamination.

Step 2—Remove unsound concrete with a 15-lb chipping hammer. Hammers larger than a 15-lb class may cause damage to the substrate and reinforcement.

Step 3—Mark the perimeter of the repair area. Layout should be simple square or rectangular shapes.

Step 4—Sawcut the perimeter of the repair. Note: sawcut should not be deeper than the cover over reinforcement.

Step 5— Repair reinforcement as necessary. When reinforcing

Prepared concrete surfaces with formwork ready for erection.

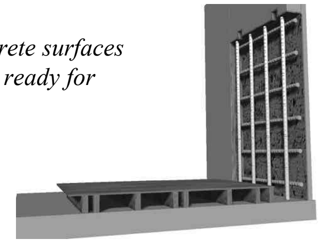


Fig. 1(a).

Section view through repair showing formwork and chute at top for placement of repair material.

Fig. 1(b).

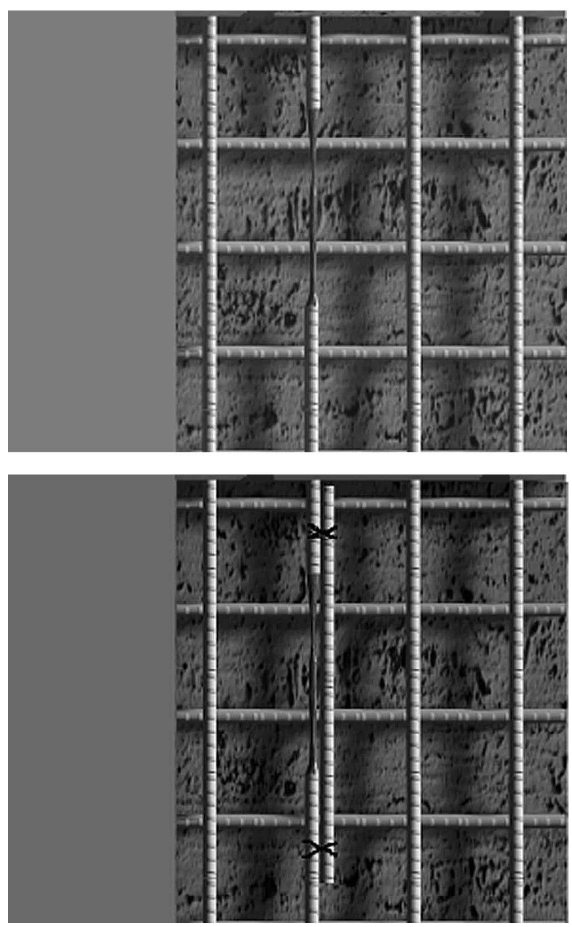


Fig. 2—Lapping of supplemental reinforcing.

steel is heavily corroded and the diameter is reduced, consult a structural engineer for repair procedures. For many applications, supplemental reinforcement can be lapped to adjacent damaged bars, as shown (see Fig. 2)

Step 6—Clean reinforcing steel and concrete with abrasive blasting.

How do I select the proper repair material?

Constructibility requirements for repair materials used with the form-and-pour technique are limited only by their ability to be transported to the formwork cavity. Maximum aggregate size should not exceed 25% of the space between the formwork and the substrate, or 50% of the distance between the reinforcing steel and the substrate—whichever is smaller. In general, the largest practical maximum size aggregate should be used to minimize drying-shrinkage and reduce the potential for cracking of the repair. Mixtures with high flowability (high slump) will make the placement easier; however, do not sacrifice a low water-cement ratio (< 0.40) for high slump. Use high-range water-reducing admixtures as necessary. Prepackaged repair materials, which are designed for high-flow placement, include shrinkage-compensating additives, and are appropriate for many applications. All mixture proportions should be optimized to minimize drying shrinkage. Shrinkage testing in accordance with ASTM C 157 measured over a 120-day period is recommended.

What equipment do I need?

Placement equipment may include either concrete buggies, buckets, or concrete pumps. Concrete pumps should be sized for the type of repair material being placed. If the repair is mixed on site, a portable mixer is required. Check with the manufacturer of the product to determine the recommended type of mixer.

What are the safety considerations?

Job site safety practices include, but are not limited to, the following where applicable:

- Material Safety Data Sheets (MSDS) available;

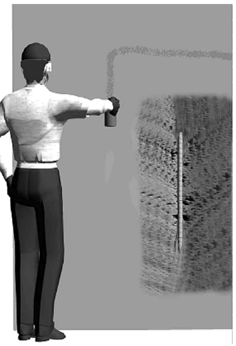
STEP 1

Sounding of concrete to locate areas of delamination.

STEP 2

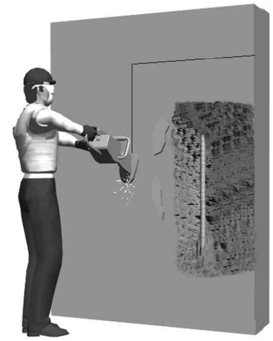
Unsound concrete removed with 15-lb chipping hammer. Hammers larger than a 15-lb class may cause damage to substrate and reinforcement.

STEP 3



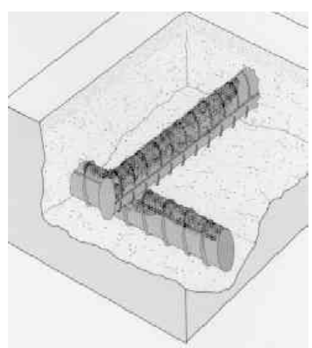
Mark perimeter of repair area. Layout should be simple geometric shapes.

STEP 4



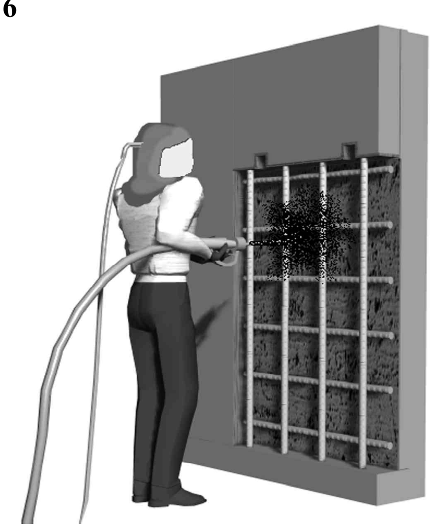
Sawcutting perimeter of repair. Note: sawcut should not be deeper than cover over reinforcement. Remove sound concrete within sawcut area.

STEP 5: *Reinforcement repair. When reinforcing steel is heavily corroded and the diameter is reduced, consult a structural engineer for repair procedures. For many applications supplemental reinforcement can be lapped to adjacent damaged bars, as shown.*



***Important Note**: If corroded reinforcing bars are encountered in the preparation process, then concrete surrounding the bars must be removed to fully expose the full circumference. Clearance under the bar should not be less than 3/4 in. (19 mm), or 1/4 in. (6 mm) greater than the largest aggregate size of the repair mixture, whichever is greater.*

STEP 6



Cleaning of reinforcing steel and concrete with abrasive blast.

- Protective clothing worn by workers handling or exposed to hazardous materials;
- Use of protective eyewear during pumping and placement of repair materials;
- Availability of eye wash facilities; and
- Use of respirators and ear protection during demolition.

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Preconstruction meeting

Prior to proceeding with the repair, a preconstruction meeting is recommended. The meeting should include representatives

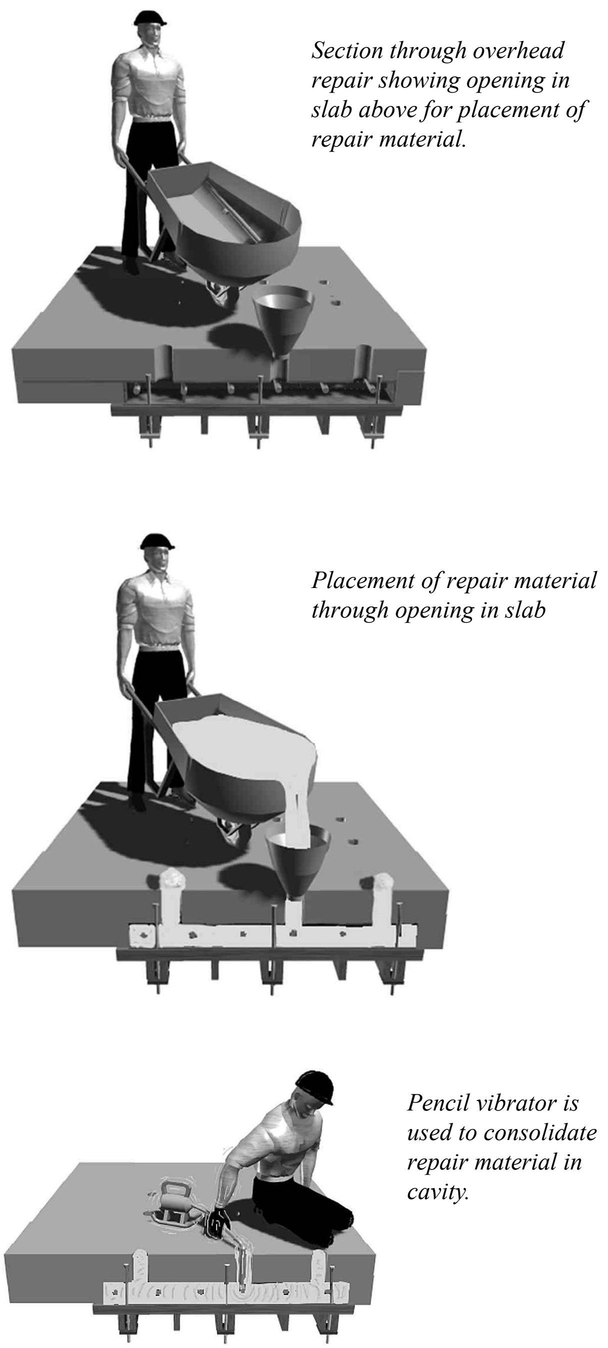


Fig. 3—Material placement: horizontal application.

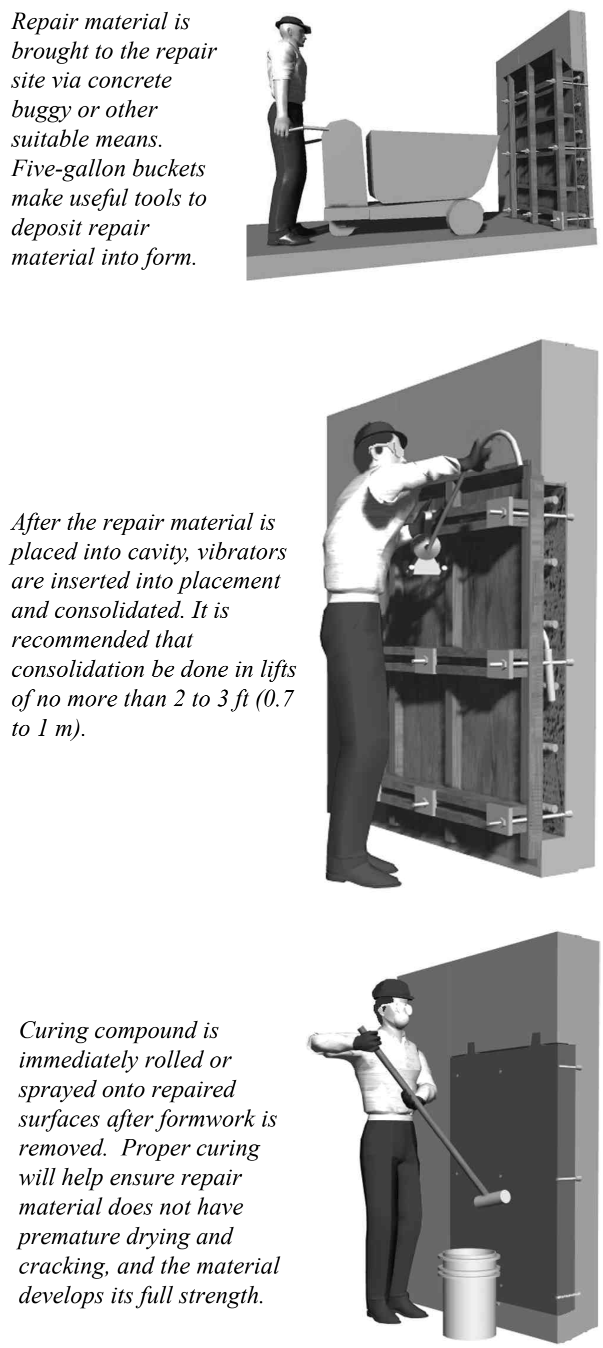


Fig. 4—Material placement: vertical application.

from all participating parties (owner, engineer, contractor, materials manufacturer, etc.), and specifically address the parameters, means, methods, and materials necessary to achieve the repair objectives.

Repair procedure

Formwork construction—Formwork must accommodate the mass and pressure of the repair material. Design of the forms should follow standard practice for cast-in-place concrete construction. Formwork is best attached directly to the concrete surface with expansion anchors or rock anchors designed for coil rod. In cases of repair of slab soffits (underside), scaffold frames or shoring posts can be used to support the formwork tight against the concrete surfaces. When expansion/rock anchors are used, ensure anchors are firmly set in place to prevent slippage under load. Preloading of rock anchors with coil rod can be accomplished with a center-hole jack applying loads to the coil rod with a stand-off. Forms should be constructed to fit tightly against existing surfaces. Preformed gaskets or cast-in-place foam work well on difficult-to-match surfaces. Placement openings or chutes are required to place the repair material behind vertical forms. Chutes should be constructed to permit development of a hydraulic head above the prepared upper edges of the concrete surface. This will provide for repair material supply into these upper horizontal zones after concrete is consolidated. For large, vertical surfaces exceeding 10 ft (3 m) in height, multiple lifts should be considered to reduce free-fall segregation and excessive formwork pressures. Formwork for overhead surfaces does not require openings for placement of repair materials. Generally, placement occurs through openings in the slab from above.

Material placement—Prior to placement of the repair material, moisture conditioning of the prepared surface should provide for saturated-surface dry conditions. It is important not to overwet the surface. Saturated surfaces will prevent proper bonding because the surface pores are clogged with water, unable to absorb the repair material. Mixed repair material is brought to the formed area via whatever transport technique is appropriate for the situation. This may include buckets, pumpline, buggies, or wheelbarrows. For vertical surfaces, material is placed into the chute or opening. External or internal vibration is a must for almost all mixture consistencies. Some self-leveling repair materials, also known as self-consolidating, can be placed without vibration. When the cavity is filled, extra care should be taken to ensure that the uppermost surfaces are filled adjacent to the chute or opening where placement occurs. Rodding or tamping can ensure proper filling. Formwork should be left in place for the prescribed curing period. After stripping of formwork, any spaces not filled should be trimmed, cleaned, and dry-packed. Placement of a membrane curing compound is recommended immediately after removal of formwork.

How do I check the repairs?

After stripping of forms, various tests can be performed to confirm that the repair material was thoroughly consolidated and that adequate bond to the substrate was achieved. A

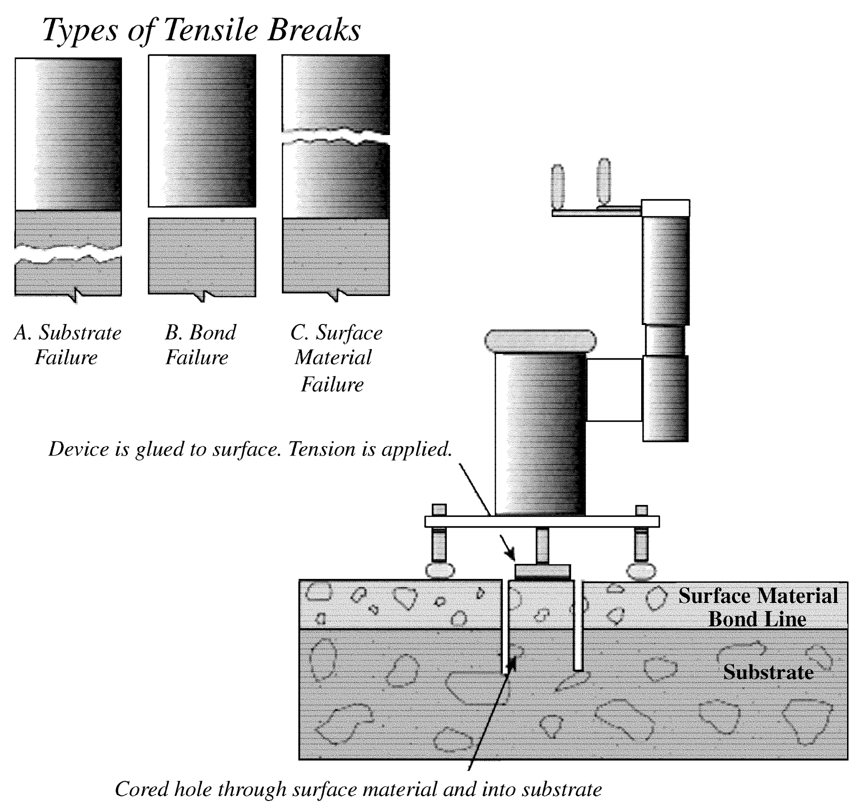


Fig. 5—Test procedure.

uniaxial bond test can be performed by drilling through the repair into the substrate. A bonded plate attached to the core is pulled until rupture occurs. The location of the failure should be reviewed. Bond values typically exceed 100 psi (0.7 MPa) and, in most cases, exceed 150 psi (1 MPa). These tests are performed in accordance with ACI 503R Appendix (see Fig. 5).

The complete repair area should also be hammer-sounded to locate voids and delaminations within the top 6 in. (150 mm). Any hollow sounds may indicate poor bond or voids.

Sources for additional information

1. "Guide for Surface Preparation for the Repair of Deteriorated Concrete Resulting from Reinforcing Steel Corrosion," No. 03730, International Concrete Repair Institute, 1995, 5 pp.

2. "Guide for Selecting and Specifying Concrete Repair Materials," No. 03733, International Concrete Repair Institute, 1996, 34 pp.

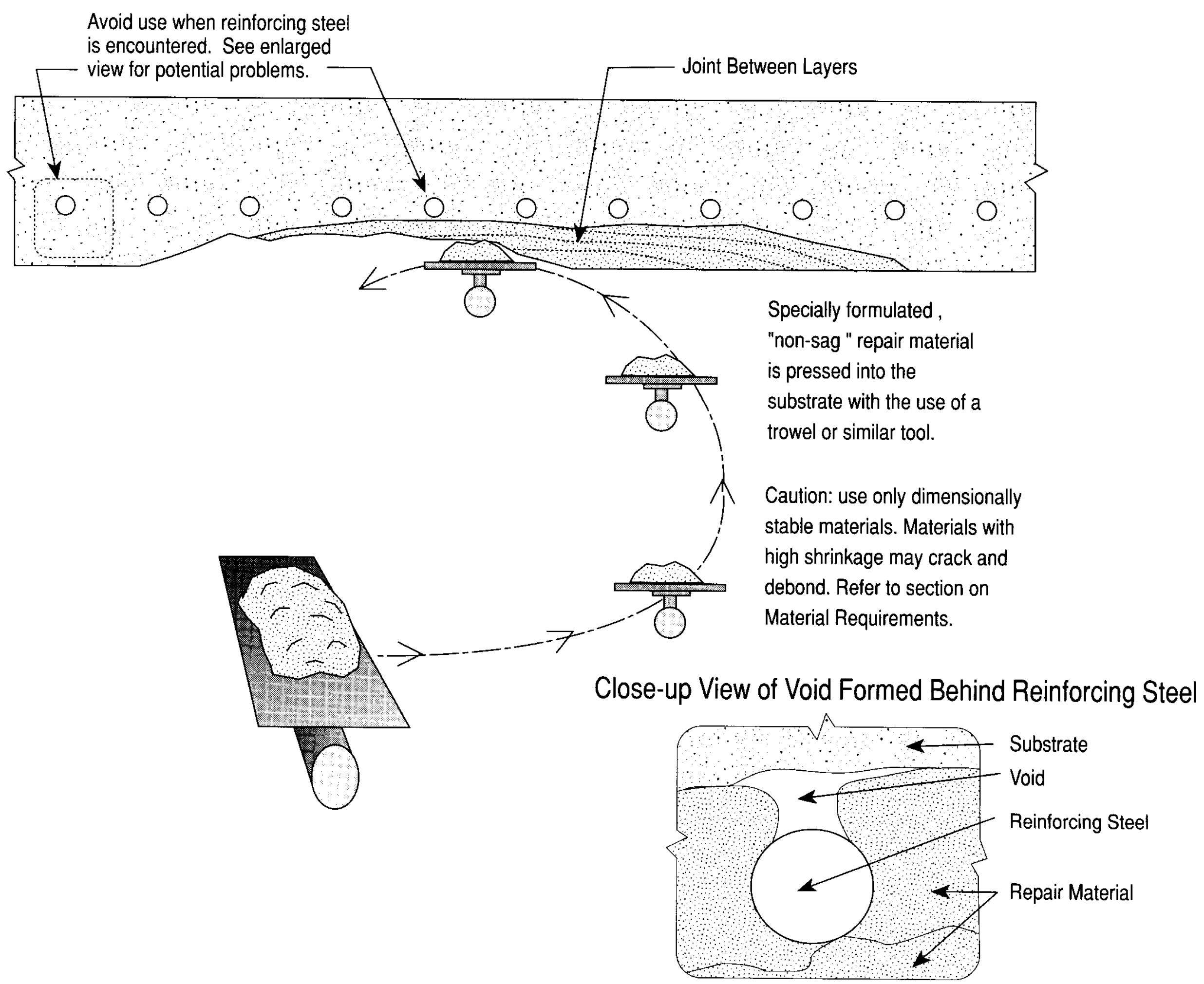
3. ACI Committee 347, "Guide to Formwork for Concrete (ACI 347-01)," American Concrete Institute, Farmington Hills, Mich., 2001, 32 pp.

4. ACI Committee 546, "Concrete Repair Guide (ACI 546R-96 (Reapproved 2001))," American Concrete Institute, Farmington Hills, Mich., 1996, 41 pp.

5. ACI Committee 503, "Use of Epoxy Compounds with Concrete (503R-93 (Reapproved 1998))," American Concrete Institute, Farmington Hills, Mich., 1998, 28 pp.

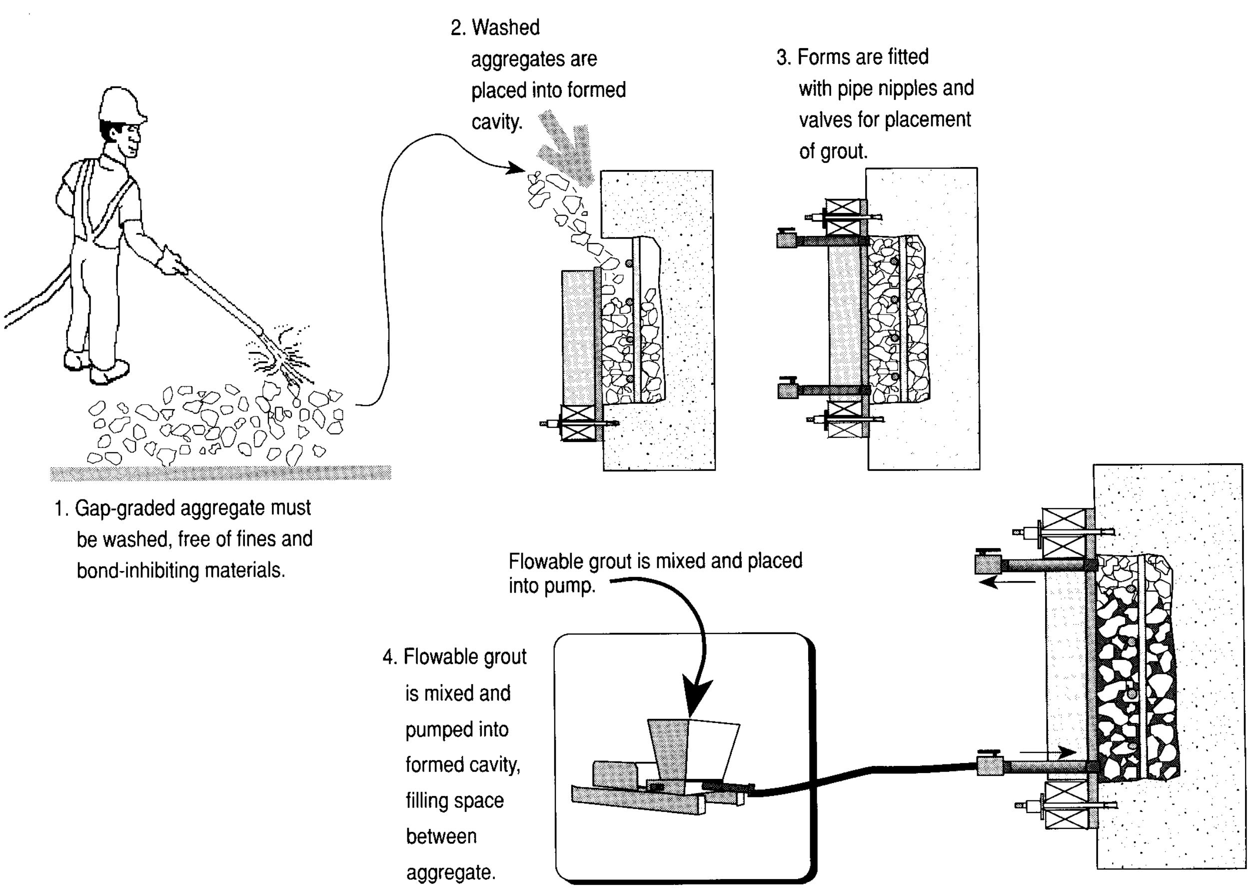
Hand-Applied

Hand-applied techniques are used to place non-sag repair materials on vertical and overhead locations. Most hand-applied materials are special blends of cement, finely graded aggregates, non-sag fillers, shrinkage compensating systems, and water. The mixed material is applied to the prepared surface with either a trowel or by hand. The applied pressure drives the repair material into the pore structure of the exposed concrete. The repair material is designed to "hang" in place until subsequent layers are added. Each layer is roughened to promote bond with the next layer. The best use of this technique is for topical cosmetic repairs not involving reinforcing steel. When reinforcing steel is encountered, it is very difficult to consolidate and provide for complete encapsulation of the reinforcing steel. Problems associated with this technique involve poor bond between layers and voids around embedded reinforcing steel.



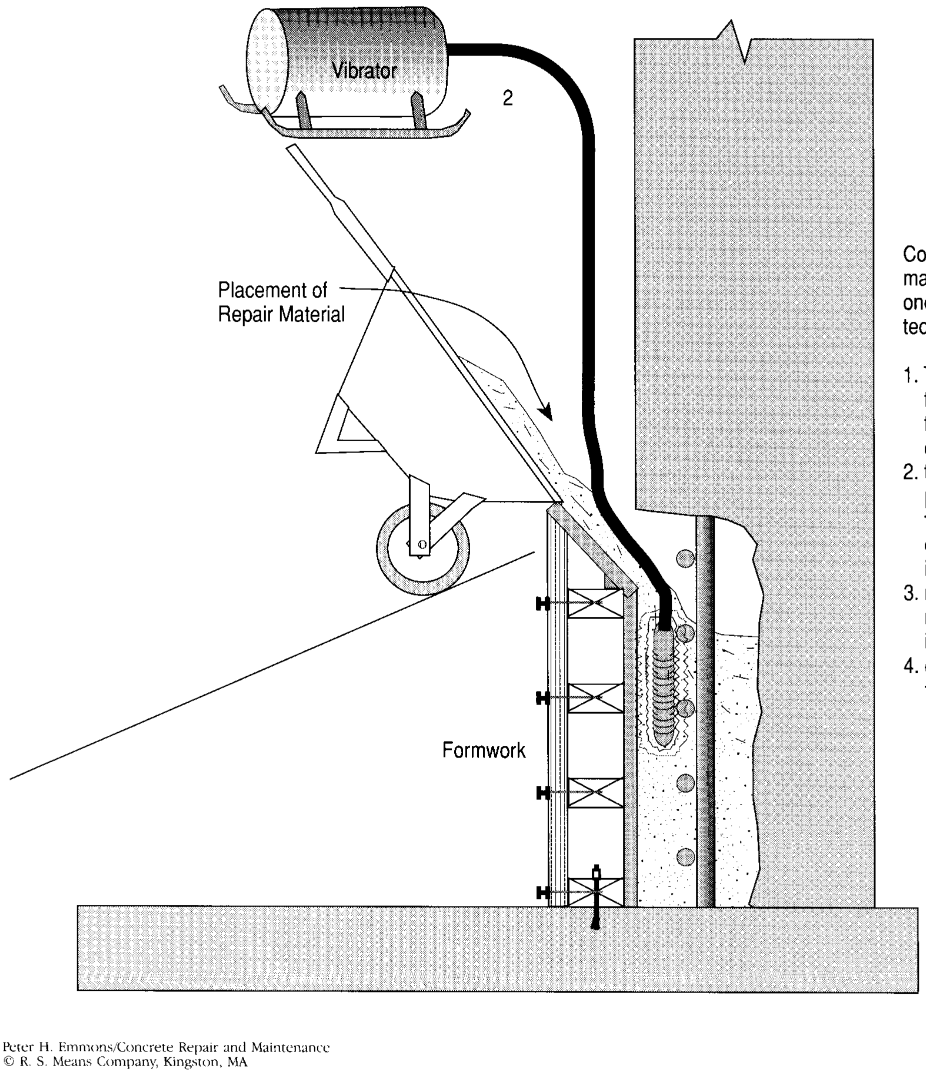
Grouted Preplaced Aggregate

Grouted preplaced aggregate is a two-step process. The first step involves aggregate placement into the cavity during the erection of formwork. The aggregate is gap-graded and washed of all fines. The void ratio of the cavity, after the aggregate is placed, ranges from 40% to 50%. The second step involves pumping a highly flowable grout through the formwork and into the preplaced aggregate. Grout flow fills the lower voids and progressively fills the cavity, eventually flowing to higher elevation ports. After grout flows from adjacent ports, the grout hose is disconnected from the port being pumped, and reconnected to the port showing new flow. The process continues until the cavity is full and pressurized. The grout flow makes contact with the prepared substrate as the cavity is filled, providing intimate contact and bonding. A unique advantage of this method is the low drying shrinkage of the repair material due to the point-to-point contact between the coarse aggregates. The aggregate contact restricts the volume change of the cement grout as drying shrinkage occurs. Various grouts can be used for the grouting process. Most popular are Portland cement-based grouts and, for special applications, epoxy resins.



Form and Cast-in-Place

One of the most common methods of surface repair of vertical and, in some cases, overhead locations is the placement of formwork and casting of repair material into the prepared cavity. Formwork facilitates the use of many different repair materials, selected on the basis of in-place performance vs. constructibility. The repair material must be of low shrinkage and provide the necessary flowability. Placement of repair materials follows normal placement practice. Rodding or internal vibration is necessary to remove air and provide intimate contact with the existing concrete substrate. Formed surfaces make the placement of bonding agents difficult and, in most applications, unnecessary. Forms are made with necessary shoots to provide access of the repair material into the formed cavity. In some applications, complete filling of the cavity may be difficult. In those cases, a final step of dry packing the remaining cavity works well.

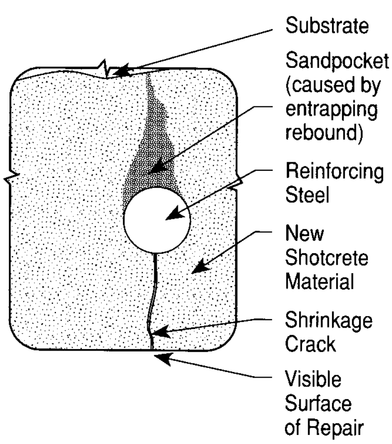


Consolidation of the repair material is accomplished with one of the following techniques:

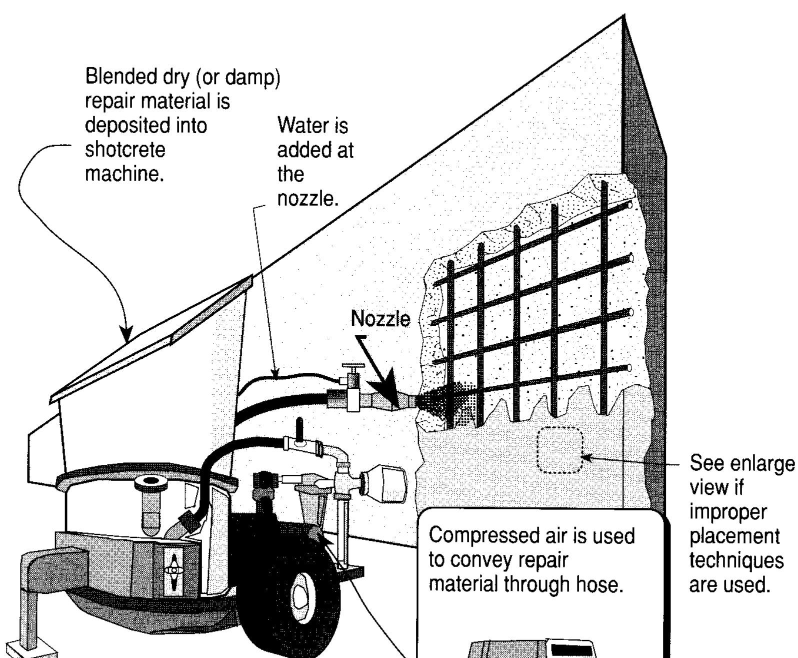
1. The repair material is formulated to be extremely flowable and self-consolidating, or...
2. the repair material is placed into top of form and free falls into the prepared cavity where conventional internal vibrators are used, or...
3. rodding of the repair material from an access point in the formwork, or...
4. external vibration of formwork.

Dry Mix Shotcrete

Enlarged View of Sandpock Formed Behind Reinforcing Steel



Dry mix shotcrete is a method that involves the premixing of binder and aggregates, which are then fed into a special mechanical feeder metering the premixed materials into a hose. The material is conveyed through the hose with compressed air to a nozzle which is outfitted with a water ring where additional water is mixed with the binder and aggregates. The mix is jetted from the nozzle at high velocity onto the prepared concrete surfaces. The process varies, depending upon the necessary thickness and orientation. Where the repair is thick, the process may involve the placement of multiple layers. Excessive thickness of individual layers may result in sloughing off. The use of special admixtures has helped improve the workability and performance of shotcrete. Silica fume is a good property enhancer. It improves the concrete's adhesive and cohesive properties, along with its ability to provide for larger placement thicknesses. The resulting hardened properties include increased flexural and compressive strengths and increased durability to freeze-thaw and chemical attack. The use of chemical accelerators should be avoided where not absolutely necessary. Accelerators have been found to cause increased drying shrinkage[1].



Shotcrete Machine

Typical Problems Associated with Shotcrete Repairs

- Presence of voids due to encapsulated rebound; common when multiple layers are used or when heavy reinforcing is encountered.
- Shrinkage cracking caused by high cement content, improper curing, or excessive water content.

[1]Morgan, D.R., Developments in Shotcrete for Repairs and Rehabilitation, Concrete Construction, No. 9, September 1991.

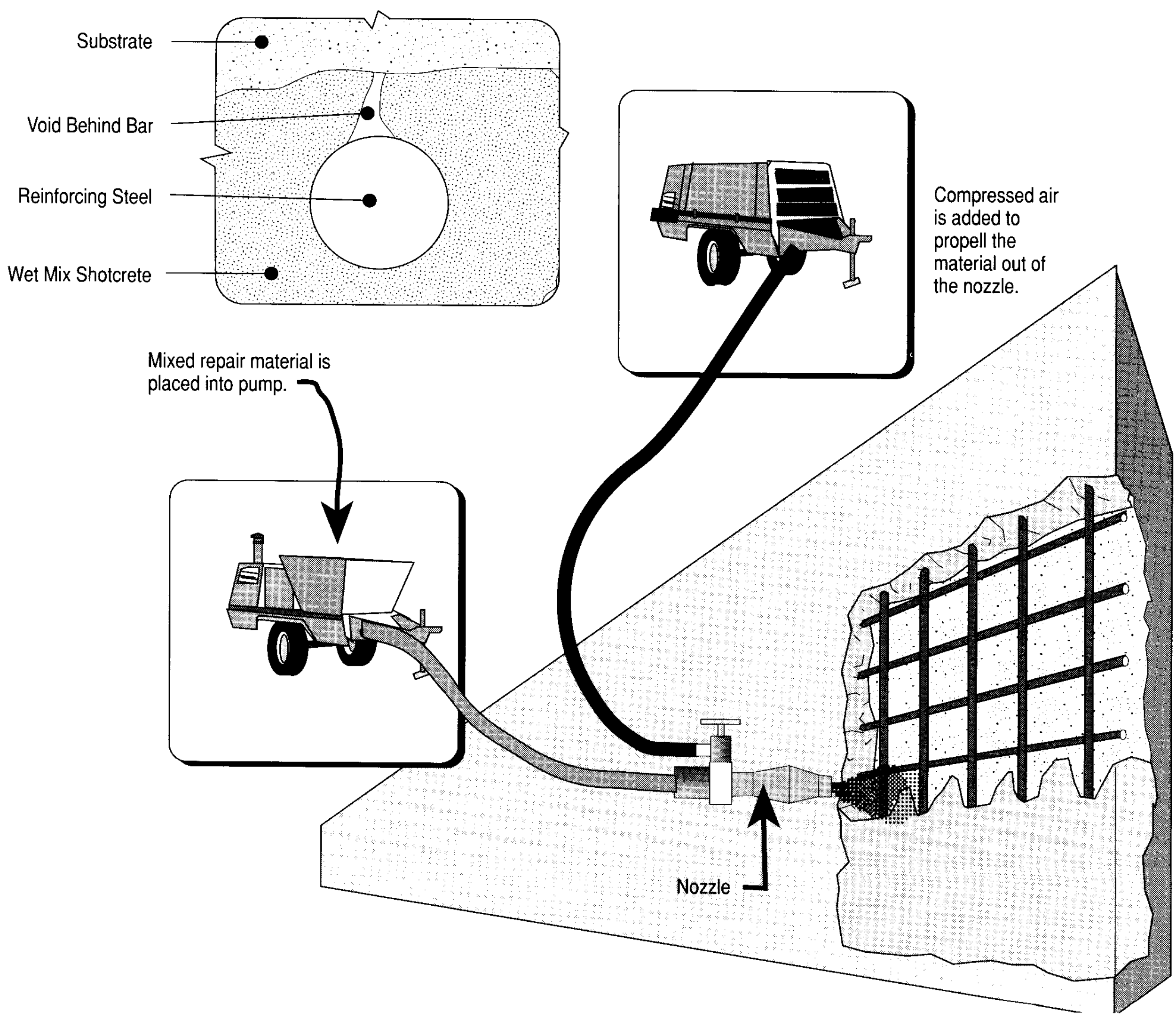
Additives for Dry Mix Shotcrete

Additives	Benefit	Comments
Silica Fume	• Increased thickness • Increased density • Increased freeze-thaw resistance • Increased chemical resistance • Reduced rebound • Increased adhesion • Increased flexural and compressive strength	
Accelerators	• Increase/buildup of layers • Reduced initial set time • Increase early strength gain	• Increased drying shrinkage • Reduced shotcrete strength with age • Not necessary if silica fume is used
Steel Fiber	• Elimination of shadows and voids which are created with conventional reinforcement • Improved impact resistance.	
Polypropylene Fibers	• Reduced plastic shrinkage cracking	
Latex	• Improved flexural,tensile bond strengths • Increased reisistance to freeze-thaw and chemical attack.	• Latex hardened film may occur between layers, causing delamination.

Wet Mix Shotcrete

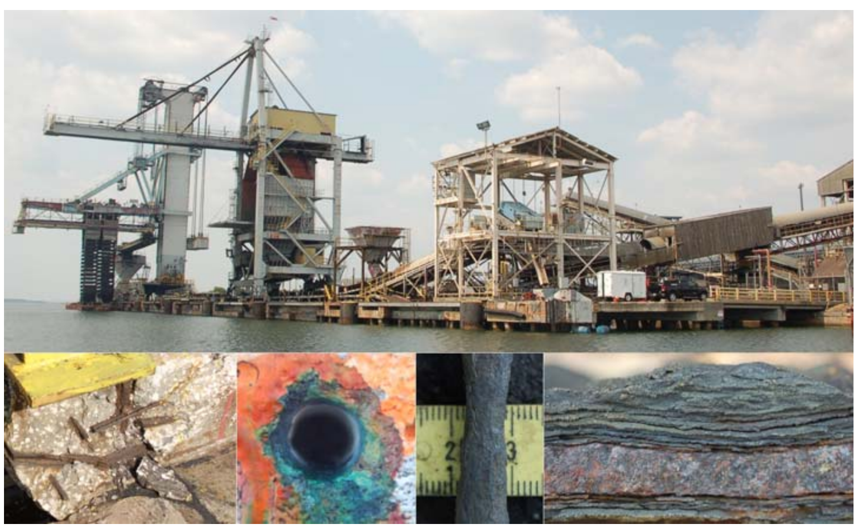
Wet mix shotcrete is a method that involves premixing of all ingredients (except accelerators) including binder, aggregates, admixtures, and mixing water. The premixed repair materials are deposited into a pump or pressure vessel which transports the materials to an exit nozzle, where compressed air is introduced. The repair material is propelled onto the substrate with compressed air. Admixtures can be used to enhance the shotcrete material. Silica fume and fibers are commonly used to enhance durability. Air entrainment is required for freeze-thaw resistance.

Potential Problem if Improperly Placed



Section 9

Glossary of Terms



SECTION 9

Glossary of Terms

Covered in this section

- Inspection Terms
- Corrosion Terms
- Concrete Terms

Acid: Containing an excess of hydrogen ions over hydroxyl ions.

Alkaline: Containing an excess of hydroxyl ions over hydrogen ions.

Anode Zone: An anode zone is defined as the area of the impressed current cathodic protection system which is powered by an independent DC power supply unit.

Alternative Current (AC): The movement of electric charge which periodically reverses direction.

Anion: A negatively charged ion of an electrolyte, which migrates toward the anode under the influence of a potential gradient.

Anode: The electrode of an electrochemical cell at which oxidation occurs. (Electrons flow away from the anode in the external circuit. It is usually the electrode where corrosion occurs and metal ions enter solution.

Anodic Area: The part of the metal surface that acts as an anode.

Attenuation: Electrical losses in a conductor caused by current flow in the conductor.

Carbonation: The chemical reaction between carbon dioxide and the calcium hydroxide present in Portland cement.

Carbonation of Concrete: Carbon dioxide present in the atmosphere combines with moisture in the concrete to form carbonic acid. This reacts with the calcium hydroxide and other alkaline hydroxides in the pore water resulting in a reduction in the alkalinity of the concrete. The rate at which this neutralisation occurs is influenced by factors such as moisture levels and concrete quality.

Cathode: The electrode of an electrochemical cell at which reduction is the principal reaction. (Electrons flow toward the cathode in the external circuit.).

Cathodic Area: The part of the metal surface that acts as a cathode.

Cathodic Protection: A technique to protect metal from further corrosion by making the protected metal a cathode of an electrochemical cell.

Cation: A positively charged ion of an electrolyte, which migrates toward the cathode under the influence of a potential gradient.

Chloride: The negative ion in salt (sodium chloride), found in sea salt, deicing salt and calcium chloride admixture for concrete. Chloride ions promote corrosion of steel in concrete but are not used up by the process so they can concentrate and accelerate corrosion.

Chloride Diffusion: Chloride ions enter concrete by the concentration gradient from an external resource, such as sea water, deicing salts, etc. It obeys the Fick's 2nd Law.

Conductor: A substance (mainly a metal or carbon) in which electric current flows by the movement of electrons.

Copper/Copper Sulfate Reference Electrode ($Cu/CuSO_4$): A reference electrode consisting of copper in a saturated copper sulfate solution.

Corrosion: The deterioration of a material, usually a metal, that results from a chemical or electrochemical reaction with its environment

Corrosion Potential (E_{corr}): The potential of a corroding surface in an electrolyte relative to a reference electrode under open-circuit conditions (also known as *rest potential, open-circuit potential, or freely corroding potential*).

Corrosion Rate: The weight loss of a corrosion coupon after exposure to a corrosive environment, expressed as mils (thousandths of an inch) per year penetration.

Current Density: The electric current flowing to or from a unit area of an electrode surface.

Direct Current (DC): Unidirectional flow of electrical charge.

Depolarization: The removal of factors resisting the current in an electrochemical cell.

Electrical Continuity: A closed circuit (unbroken electrical path) between metal components under consideration.

Electrical Isolation: The condition of being electrically separated from other metallic structures or the environment.

Electrochemical Cell: Electrochemical cell consisting of an anode and a cathode immersed in an electrolyte. The anode and cathode may be separate metals or dissimilar area on the same metal.

Electrode Potential: The potential of an electrode in an electrolyte as measured against a reference electrode. (The electrode potential does not include any resistance losses in potential in either the electrolyte or the external circuit. It represents the reversible work to move a unit of charge from the electrode surface through the electrolyte to the reference electrode.)

Electrolyte: A chemical substance containing ions that migrate in an electric field.

Electromotive Force Series (EMF): A list of elements arranged according to their standard electrode potentials. A sign being positive for elements whose potentials are cathodic to hydrogen and negative for those anodic to hydrogen.

Energizing: The process of initially applying power to turn on an electrical system.

Equilibrium Potential: The electrode potential with reference to a standard equilibrium.

External DC Power Source: DC Current is provided by an external transformer rectifier or battery.

Extraneous: Existing on or coming from the outside or not forming an essential or vital part.

Foreign Structure: Any metallic structure that is not intended as a part of a system under cathodic protection or electro osmosis.

Galvanic Anode: A metal provides sacrificial protection to another metal that is nobler in the galvanic series when coupled in an electrolyte due to its relative position in the galvanic series. These anodes are the current source in galvanic/sacrificial cathodic protection.

Galvanic Series: A list of metals and alloy arranged according to their relative potentials in a given environment.

General Corrosion: Corrosion in a uniform manner, usually quite predictable**.**

Half Cell: A half-cell is half of an electrolytic or voltaic cell, where either oxidation or reduction occurs. The half-cell reaction at the anode is oxidation, while the half-cell reaction at the cathode is reduction.

Immediate Voltage Shift: The difference between the potential value when the power source is on and the instant off value. (This is also referred to as *IR Drop*.)

Impressed Current: The connection of an external DC power source between the anode and the cathode.

Instant-Off Potential: The polarized half-cell potential of an electrode taken immediately after the cathodic protection current is stopped, which closely approximates the potential IR drop (i.e., the polarized potential) when the current was on.

Insulating Coating System: All components comprising the protective coating provide effective electrical insulation of the coated structure.

Ion: An electrically charged atom or group of atoms.

IR Drop: The voltage across a resistance in accordance with Ohm's Law.

Linear Polarization Resistance: A technique to measure corrosion rate. It requires to polarize the steel with an electric current and monitors its effect on the half cell potential. The change in the half cell potential is simply related to the corrosion current by the equation $I_{corr}=B/ R_p$ when B is a constant and R_p is the polarization resistance. It is decided by the change in potential divided by applied current.

Mixed Potential: A potential resulting from two or more electrochemical reactions occurring simultaneously on one metal surface.

Native Potential: See Corrosion Potential.

Open-Circuit Potential: The potential of an electrode measured with respect to a reference electrode or another electrode in the absence of current.

Oxidation: Loss of electrons by a constituent of a chemical reaction.

Passive Film: The passive film is a thin, dense layer or iron oxides and hydroxides with some mineral content. It is initially formed when bar steel is exposed to oxygen and water but then protects the steel from further corrosion due to its high density which doesn't allow humidity and oxygen to reach the steel.

Passivation: The process by which steel in concrete is protected from corrosion by the formation of a passive film due to the highly alkaline environment in concrete.

Pitting Corrosion: Localized corrosion of a metal surface that is confined to a small area and takes the form of cavities called pits.

Polarization: The change from the open-circuit potential as a result of current across the electrode/electrolyte interface.

Polarization Decay: The decrease in electrode potential with time resulting from the interruption of applied current.

Potential Survey: Obtaining potentials with respect to a reference electrode at multiple locations on the surface of the concrete structure.

Protection Current: The current made to flow into a metallic structure, with respect to a specified reference electrode in an electrolytic environment, to effect cathodic protection of the structure.

Reaction: A process of chemical or electrochemical change, particularly taking place at or near an electrode in an electrochemical cell.

Rectifier: An electrical device for converting alternating current (AC) to direct current (DC).

Reduction: Gain of electrons by a constituent of a chemical reaction.

Reference Electrode: An electrode whose open-circuit potential is constant under similar conditions of measurement, which is used for measuring the relative potentials of other electrodes.

Rest Potential: See Corrosion Potential.

Rust: Corrosion product consisting of various iron oxides and hydrated iron oxides. (This term properly applies only to iron and ferrous alloys.)

Sacrificial Protection: Reduction or prevention of corrosion of a metal in an electrolyte by galvanically coupled it to a more anodic metal.

Silver/Silver Chloride Reference Electrode [Ag/AgCl]: A reference electrode consisting of silver, coated with silver chloride, in an electrolyte containing chloride ions.

Step-and-Touch Potentials: The electrical potential gradients that may exist between two points on the electrolyte surface equal to one pace (one meter) or between a grounded metallic object and a point on the electrolyte surface separated by the distance equal to a human's normal reach (one meter).

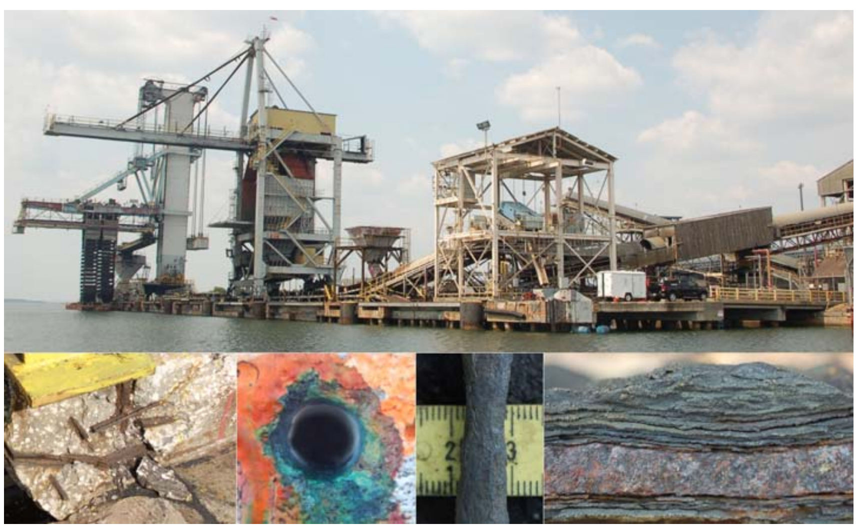
Stray Current: Current through paths other than the intended circuit.

Stray Current Corrosion: Corrosion resulting from direct current flowing through paths other than the intended circuit.

Structure-to-Electrode Potential: The voltage difference between a buried or embedded metallic steel and electrolyte that is measured with reference to an electrode in contact with the electrolyte.

Section 10

Recommendations



SECTION 10

Recommendations

Covered in this section

- Decision Tree
- Phased Approach

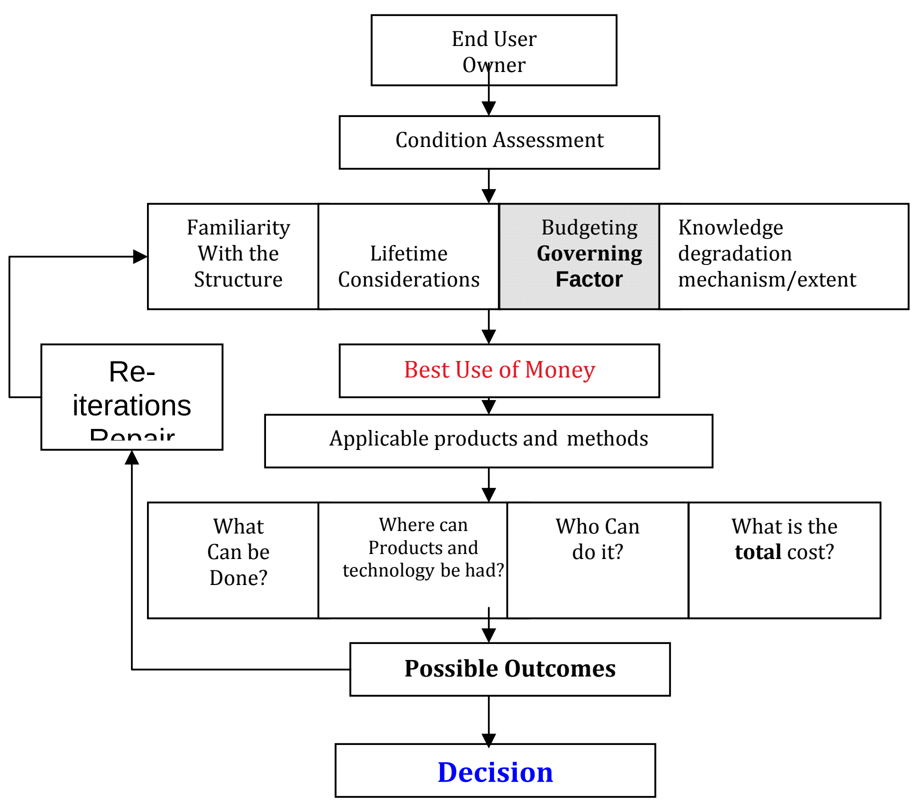
Decision Tree

The flow diagram in the figure below shows the re-iterative process of finding the best maintenance strategy within the budgeted resources. These steps are necessary to provide information and documentation on Reliability, Availability, Maintainability and Safety of repair and retrofitting products and techniques. For technical purposes, these are limited to:

1. Surface protection materials and techniques

2. Mechanical repair materials and techniques

3. Electrochemical repair materials and techniques



The corrosion site inspection provides the first link in the chain when trying to diagnose the problem and provides us with the ability to answer the questions as detailed within the decision tree as follows:

Q1 Condition Assessment

This report provides the basis of the condition assessment and the ability to answer the series of questions required to make a repair recommendation.

Q2 Familiarity with the structure

The condition assessment enables us to become familiar with the structure and understand all of the different elements and conditions.

Q3 Lifetime Considerations

Lifetime considerations are critical in extending the life of the structure and is typically built around understand at what rate the structure will deteriorate in the future and how long is needed to react to current conditions.

Q4 Budgeting (Governing Factor)

To enable budgets to be formed an understanding of the condition combined with the governing factors allows us to phase the work based on available funding.

Q5 Knowledge Degradation Mechanism/Extent

The condition assessment has enabled us to quantify the visible defects and provide the answers to the mechanism of those defects and the extent.

Q6 Best Use of Money

When all of the above questions can be satisfactorily answer it becomes a more simple process to use the money available on the right parts of the structure.

Q7 Applicable Products and Methods

The use of the right product in the appropriate manner is essential when trying to achieve an extension of life. Our investigation provides us with recommending means and method with the correct product in the right application.

Q8 What can be done

As detailed within our phased approach a series of work is required in accordance with our findings in the report.

Q9 Where can Products and Technology be had

Incorporating technology in to the repair process more guarantee is achieved with the target goal of lifetime requirements. All of the chosen methods of repair utilize the appropriate product to fit the technology used.

Q10 Who Can do it

Selecting the right contractor Structural to carry out complex technology is centered on experience and the quality of the personnel carrying out the work. In addition to this it is always more beneficial to have a design build concept to extract best value.

Q11 What is the Total Cost

Budget estimates have been prepared to enable the possible outcome and ultimately a decision in how to repair the dock going forward.

Q12 Possible Outcomes

When a number of options are available it is sometimes necessary to reiterate the information and provide alternative outcomes. This is typically the result of a budgeting factor. As with TECO coal dock the possible outcomes for repair are detailed within the phased approach of this section.

Q13 Decision

The ultimate decision based on information and knowledge available and the quality of the results provided.

Phased Approach

Stage 1 Engage Structural engineers Robert Silman Associates who specialize in durability engineering on corrosion and structural implications.

Contact: Mr Derek Trelstad PE

88 University Place
New York
NY 10003
Tel: 212 620 7970
Email: trelstad@silman.com

This stage allows a structural integrity concept to be used to ensure that repairs can be carried out in the appropriate sequence and the durability of the structure to be predicted.

Stage 1a As part of stage 1 further investigation to all of the dolphins is required due to the limit inspection carried out and the conditions found. This would involve the use of Dive Tech to complete this task.

Stage 1b In addition to stage 1a further investigation to the condition of the prestressed slab is required to ensure the structural integrity. As the wearing course on the deck should provide adequate protection to the prestressing wires we would recommend this at one location which is currently heavily delaminated on the deck.

Stage 2	Develop contract drawings for the repair of the structural deficiency in the dolphins. This would involve the structural engineers to design a retrofit system for underwater strengthening of the "H" piles..
Stage 2a	Develop contract drawings for the design of the bulk anode system for the steel "H" sections of the dolphins. .
Stage 2b	Issue Contract documents to install strengthening work to dolphins an bulk anode system.
Stage 3	Develop contract drawings for the design of the lifejacket system for the transverse beams. This would include on site measurements of the beams to ensure correct sizing of the jackets.
Stage 3a	Issue Contract documents to install Lifejacket system to transverse beams including all associated concrete repairs.
Stage 3b	Issue Contract documents to install expansion joints on main deck and include areas of deck defects with exposed reinforcing steel.
Stage 4	Develop contract drawings for the design of the impressed current cathodic protection system for the walkways.
Stage 4a	Issue Contract documents to install impressed current cathodic protection system to access walkways.

The above sequence of works can be staged as proposed or issued as one contract.