INSPECTION AND REPAIR OF UNDERWATER CONCRETE STRUCTURES

by

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by

Domenic D'Argenzio

ABSTRACT

The service life of any concrete marine structure is influenced by the physical condition of both the above water and below water portions of structure. This requires implementing an adequate inspection, maintenance, and repair program for the entire structure. To develop an effective maintenance and repair program for the submerged portion of the structure, the causes and extent of concrete distress or deterioration must be clearly understood. This requires a selective underwater condition survey, using a range of in-situ and laboratory testing and inspection techniques, to obtain the necessary information to assess the condition of the submerged portion of the structure. The cause and extent of deterioration, site logistics, and the clients needs will dictate the methods of inspection and repair. Recent developments in concrete admixtures has made it possible to place higher quality concrete suitable for underwater repairs. This thesis provides a summary of the most common forms of concrete distress found in a marine environment, along with a state-of-the-art review of existing and recently developed underwater inspection and repair techniques.

In addition, four case studies are presented to illustrate the application of the above knowledge. The first case study describes the special aspects of underwater repairs to a concrete storm surge barrier damaged during construction. The second case study summarizes the procedures used for repairing a cracked concrete gravity dam by polyurethane resin injection methods. The third case study presents the various procedures used for repairing concrete railway bridge piers in a marine environment which were damaged by severe alkali-aggregate reaction. The final case study describes underwater repair procedures and concrete investigation techniques used in repairing the piers of a highway bridge.

INSPECTION ET RÉPARATION
DES
STRUCTURES EN BÉTON DANS L'EAU

par

Domenic D'Argenzio

SOMMAIRE

La durée de vie de toute structure maritime en béton est influencée par les conditions physiques, les parties immergées, ainsi que les parties submergées. L'augmentation de la durée de vie nécessite un système adéquat d'inspection et de réparation. Pour développer un programme de réparation de la partie submergée de la structure, les causes et l'éntendue des dommages et des détériorations doivent être bien définies. Ceci nécessite une surveillance sélective des conditions sous-marines, en utilisant un système technique d'inspection au laboratoire, et in situ, dans le but d'obtenir les informations nécessaires pour évaluer les différentes conditions structurales. Les causes et le degré de détérioration, les conditions du chantier, et les besoins du client définissent les moyens d'inspection et de réparation.

Ce mémoire présente un résumé sur les formes de détérioration du béton connues dans un environnement marin, une description des différentes techniques d'inspection et de réparation récemment développées ou déjà existantes, ainsi que des additifs utilisés qui ont permis d'améliorer la qualité du béton utilisé dans des environnements sous-marins.

En plus, quatre cas d'études sont présentés pour illustrer l'application des connaissances résumées dans ce mémoire. Le premier cas décrit l'aspect spécial de la réparation dans l'eau pour le béton du barrage d'assaut, endommagé durant construction. Le second résume les procédures utilisées pour la réparation des barrages-poids par injection de résine polyuréthane. Le troisième cas présente les differentes procédures utilisées pour la réparation des piliers des ponts ferroviaires dans un environnement marin, endommagés par des réactions alkali-agrégats. Le dernier cas décrit la réparation dans l'eau et les techniques de sondage du béton utilisées pour réparation des piliers de pont.

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INTRODUCTION

1.1 STATE OF URBAN INFRASTRUCTURE

The quality of a nation's infrastructure is essential to its continued competitiveness and economic growth. A good economy as well as healthy and liveable cities, towns, and villages depend on reliable and adequate transportation networks (i.e., airports, roadways, waterways), clean and efficient water distribution systems, and safe disposal of domestic and hazardous wastes. Their deterioration or failure to perform adequately will create an intolerable hardship to everyday life and endanger the productivity of a nation's economy.

Many concrete structures are now approaching or have reached or exceeded their design service life and are being used in excess of their design capacity. In reviewing government studies and surveys conducted in Canada and the United States, there is convincing evidence that the quality of the infrastructure in the western world is barely adequate to meet current requirements, however, it is inadequate to meet the projected demands primarily due to increasing budgetary constraints. For instance, a recent report by the Road Transportation Association of Canada (RTAC)[1] revealed that 38 percent of the national highway system was found to be substandard and 22 percent (790) of the bridges needed major strengthening and rehabilitation, requiring a total rehabilitation investment of $13-14 billion (1993 dollars) in the next five years.[2] A similar study conducted in the United States on the state of the nation's infrastructure revealed that, based on an academic scale, their performance (in 1987) would receive a "C - barely adequate to support the required demands".[3] The cost to upgrade public facilities were reported to be in the range of hundreds of billions of dollars. Similar results have been reported in Europe, the U.K., and other countries.

Part of the problem has been lack of public awareness, mainly due to the fact that public works facilities are often taken for granted, since many of them are out of sight. A major contributing factor to the infrastructure problem has been the decline in government assistance programs throughout Canada and other countries, mainly due to large existing deficits. The fear of increasing their deficit further, governments are unwilling to accept responsibility for rebuilding and upgrading urban infrastructure facilities.[4]

However, studies by the Federation of Canadian Municipalities (FCM)[5] and others have demonstrated that infrastructure renewal can directly improve the economic viability of commerce and industry, thereby generating increased national revenues which can reduce deficits just as effectively as reducing expenditures. For every billion dollars invested in Canada's construction industry, 20,000 jobs can be created.[2] Similarly in the United States, for every public dollar spent annually to build and maintain the infrastructure, the private sector spends $15 (US) to move people and goods. For every aviation dollar, private firms and individuals spend nine dollars[3] Taking into account unemployment insurance and welfare savings, and increased government revenues, these benefits are compounded. Although public awareness to the crisis and government funding has increased, the problem still remains a large one. The material in this chapter has been adapted from different available references, especially 7 and 13.

1.2 RESOLVING THE INFRASTRUCTURE CRISIS

An obvious solution to the problem may be to make available the funds necessary for rehabilitation. However, a dependable, high quality infrastructure is not attained by money alone: building and maintaining public works requires the skill, and commitment of time and energy of people throughout the public and private sector. For example, Curtis[4] reports that rehabilitation costs can be reduced in Canada by:

- Resistance to public pressure for elaborate and expensive facility treatments.
- Elimination of unnecessary delays in correcting problems.
- Improving technical efficiency. Generally these are associated with standardized treatment and automated mass production techniques.
- Greater reliance on payment by users.

A report on the state of public works in the United States[3] recommends that a strategy to upgrade the infrastructure must include other mechanisms in addition to increased investment. These include:

- Classification of the respective roles of all levels of government (federal, state, and local) in the construction and management of infrastructure to increase accountability.
- More flexible administration of federal and state mandates to allow cost-effective methods of compliance.
- Steps to upgrade the quality and quantity of basic public works management information

in order to measure and improve the performance and efficiency of existing facilities.

- Financing of a larger share of the cost of public works by those who benefit from service.
- Additional support for research and development to improve technologies and for training of public works professionals.

All of the above leads to six main categories of needs to improve response to the infrastructure crisis:

- Apply the best management techniques to infrastructure management;
- Eliminate corruption wherever it exists in the infrastructure field;
- Apply the best skills of the private sector;
- Reduce built-in industry structure problems such as fragmentation and harmful duplication and competition;
- Improve the process of education for the infrastructure managers; and
- Consider trade-offs, and where a level of infrastructure service cannot be maintained, it should be reduced.

Clearly, the required task dictated by these recommendations is indeed a challenging one. In fact, it will require a shared vision and effort by both the public and the private sectors. With cooperation, the infrastructure problem can be resolved over a reasonable period of time without unreasonable budget increases. The sooner the problem is addressed, the easier it will be to resolve. It should be kept in mind that major public works projects have a long lead time, especially if they are located in a metropolitan area or involve controversial issues. Continued delay will only allow deterioration to advance to the point where costs will escalate.

1.3 INNOVATIVE REINVESTMENT OPTIONS

Many industrialized nations of the world have come to realize the importance of investing in their public infrastructure. Due to increased competition for the tax dollar, finding innovative ways for financing current and future public works needs has become equally important. While several different options have been proposed, they all depend on general tax revenues and user fees. A brief [2] submitted to the federal government of Canada in February 1993 provided the following options as possible financing mechanisms:

- Build-operate-transfer systems

- Lease-to-purchase options
- Dedicated taxes
- User fees and other privatization schemes, and
- The issuance of tax exempt bonds

Consequently, a national poll commissioned by The Road Information Program (TRIP) of Canada in April 1993 determined that 58 percent of the respondents questioned supported the idea of a road user fee to rehabilitate the National Highway System.

Similar recommendations were developed by the National Council on Public Works Improvement in the United States.[3] These included:

- Users and other beneficiaries should pay a greater share of the cost of infrastructure service;
- The federal government should be an equal partner in financing public works;
- States should develop comprehensive finance strategies; and
- Local governments should give financial priority to funding the maintenance of existing facilities.

In planning for public works investments, both the public and private sector should be given clear stated performance objectives, consider alternative ways of achieving them, and have easy access to information about costs of operation and maintenance.[3] Implementing these steps will not be an easy task, but the increase in costs due to delay will hinder a nation's ability to cope with the infrastructure crisis.

1.4 REHABILITATION STRATEGIES

Rehabilitating concrete structures is a specialized field requiring skill and expertise. Due to the complexity of the restoration process, the design engineer must be able to perform several roles: that of an investigator, designer, materials performance specialist, and construction inspector.[6] To simplify the rehabilitation process, engineers have attempted to devise a systematic (or decision-making) approach to arrive at an appropriate solution. One possible approach is that developed by Tracy and Fling,[6] which emphasizes the implementation of three distinct phases to the rehabilitation process (Figure 1.1): the concrete condition survey which identifies the cause, rate, and extent of deterioration; the structural aspects investigation which places structural, functional, and operational constraints on the rehabilitation process, and the repair program which gives various solutions to the identified problems.

A similar but more rigorous approach was developed by Chung[7] for repairing concrete structures damaged by steel reinforcement corrosion, although the approach can generally be adapted and

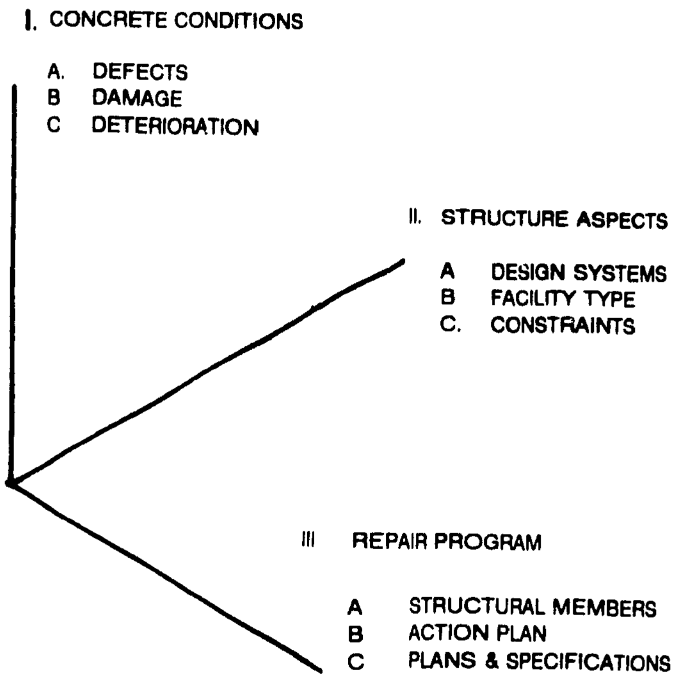


FIGURE 1.1 - REHABILITATION MATRIX[6]

applied to structures in general. The inter-related factors involved in the repair strategy are shown in Figure 1.2. The basic steps involved in the procedure are summarized below.

1.4.1 THE CONDITION SURVEY

The first step in the restoration process is to conduct a condition survey of the structure to collect sufficient data to determine the cause, extent and rate of deterioration. The survey should not be limited to the damaged portions only, but should include the structure as a whole. Test results from sound areas of the structure are essential in providing necessary baseline data for comparison. It is good practice to obtain a sufficient number of test results to perform the required statistical analyses.

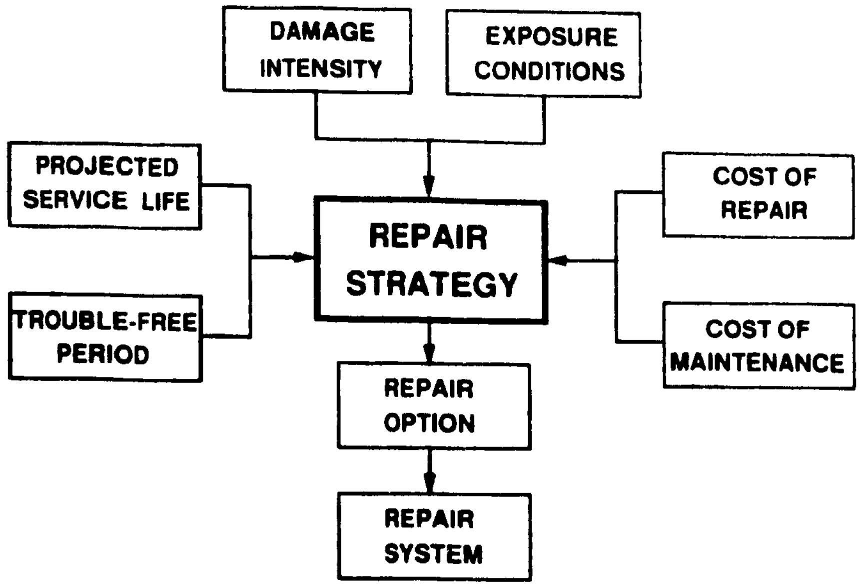


FIGURE 1.2 - INTER-RELATED FACTORS OF REPAIR STRATEGY[7]

From the condition survey an assessment of the visible and hidden damage in the individual structural members is made, with each member or portion of the structure being classified in accordance with the intensity of damage (i.e., depth of carbonation, chloride content, degree of spalling and cracking, and degree of rusting).[7] The classification shown in Table 1.1 may be used as a guide. Although classification of damage intensity is mostly arbitrary, it serves as a useful guide for selecting the appropriate method of repair.

1.4.2 PROJECTED SERVICE LIFE VS. TROUBLE-FREE PERIOD

The decision to execute repairs and the complexity of the repair system to be used depends on the extent of deterioration and the length of time the structure is still required to function after the repairs are made. This time span is referred to as the projected service life (PSL). According to Chung[7], the PSL of a structure can be categorized as follows

- Short projected service life: < 5 years
- Medium projected service life: 5-15 years
- Long projected service life: > 15 years

TABLE 1 1 - CLASSIFICATION OF DAMAGE INTENSITY[7]

Damage intensity	Visible signs of distress
Light	spalling not apparent; hair-line cracks only; little rust stain
Moderate	spalling at isolated spots; fine cracks (<0.2 mm) with rust stain
Severe	extensive spalling and cracking; corroded steel visible
Very severe	extensive spalling and cracking; substantial steel pitting

If the structure has completed its full design life when deterioration reaches the unacceptable level, then repairs will not be necessary, unless the structure is required to continue its intended function. Often, the structure will be in active service when deterioration becomes unacceptable. Sometimes deterioration has progressed to the point where the structure or member is either technically or economically beyond repair. In such a case, the only alternative is to rebuild the structure in its entirety, or in part.

If the structure is to have a short PSL, extensive repairs may not be economically justifiable. In most cases, no repairs are needed, unless safety is a concern. With the PSL falling in the medium or long range, the degree of deterioration which requires immediate repair may vary according to opinion. Some owners or managers tend to postpone any action until the operation or safety of the structure is impaired, while others prefer to perform minor repairs or maintenance more often. The deciding factors include accessibility of the repair areas, provision of facilities for repair/maintenance and availability of funds.

If the intensity of damage is light and the concrete and reinforcing steel are in good condition, immediate repairs are not necessary. However, implementing preventive maintenance procedures, such as applying protective coatings, frequently helps to prevent light damage from becoming critical, thereby extending the service life of the structure.

Any repair work or protective measure chosen for the structure must perform adequately for some period of time before another repair job becomes necessary. Chung[7] defines this period as the trouble-free period (TFP) of the structure, and is characterized as follows:

- Short trouble-free period: < 5 years
- Medium trouble-free period: 5-15 years
- Long trouble-free period: > 15 years

The TFP of a repaired structure will vary with the complexity and thoroughness of the repair work performed. In general, a TFP as long as the PSL is preferred, however, availability of funds is usually limited, thereby affording a less durable repair. The relationship between TFP and PSL is represented schematically in Figure 1.3. A flow chart which may be used for selecting an appropriate TFP is shown in Figure 1.4.

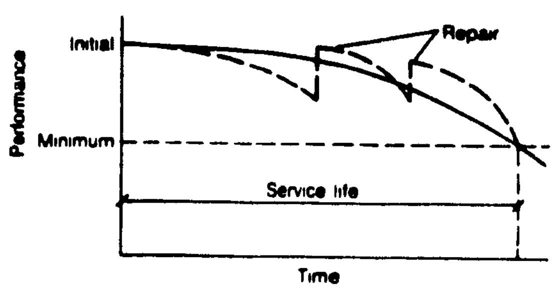


FIGURE 1.3 - RELATIONSHIP BETWEEN CONCRETE PERFORMANCE AND SERVICE LIFE[23]

1.4.3 REPAIR ALTERNATIVES

Once the probable cause or causes of deterioration have been determined, an appropriate repair option can be selected. In selecting a repair option, the owner of a deteriorated structure must first make a decision, based on economical reasons, whether:[8]

- To do nothing and allow the structure to deteriorate and eventually demolish it;
- To implement short-term repairs knowing that further repair work will be needed before the end of the useful life of the structure; and
- To undertake a major rehabilitation or replacement project to provide a structure which will provide a trouble-free period equal to its projected service life with only routine maintenance.

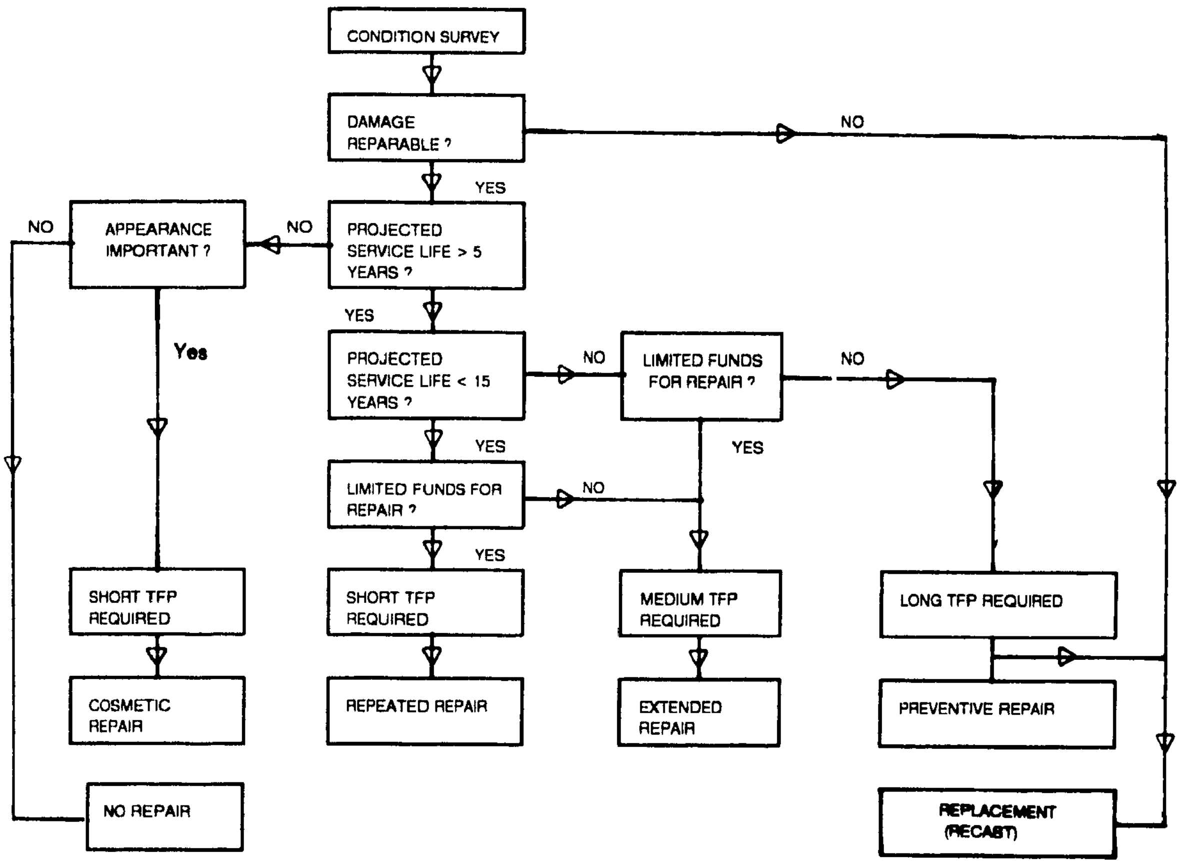


FIGURE 1.4 - FLOW CHART FOR DECISION ON TROUBLE-FREE PERIOD AND REPAIR OPTION[7]

Depending on the rehabilitation scheme chosen the objective of the repair is to restore the structure to a point where it can provide the required trouble-free period. The possible options of repair include cosmetic, repeated, extended, preventive and replacement.[7] The decision to make extended repairs or replacement is usually difficult to make and depends on several interdependent elements including, the extent of damage, the temporary shoring required during the repair, and the extent of interference with the operation of the structure. The choice in the end will be an economical one.

1.4.4 REPAIR TECHNIQUES

There are several repair systems available for performing repair work, including crack injection, patching, shotcrete, protective coatings, cathodic protection, chloride removal and replacement. Some systems, such as cathodic protection are designed to make the concrete inconducive to reinforcement corrosion, while others will prevent the ingress of solutions that promote corrosion (e.g., protective coatings). However, systems that replace the damaged concrete (i.e., patching, shotcrete) or fill the cracks within the concrete (i.e., crack injection) are more commonly used

Although it may seem desirable to use the same repair method throughout the entire structure, a combination of one or more of the above repair methods is often required and is usually more economical. In any case, if the cause of concrete deterioration is not removed, the repair method chosen will only conceal, and often, aggravate the problem.

1.5 LIFE-CYCLE COSTING

Choosing an appropriate repair option and repair system is not a simple matter, although several options and systems may be equally effective in restoring the structure. In some cases, different options or systems may either be required or may be more economical for different parts of the structure. In any case, each possible solution should be costed for the projected service life of the structure. The solution which is chosen is typically the one with the lowest cost.

The estimated cost of any repair option or system should include the expenses incurred for access, equipment, labor and materials, and future maintenance. The total cost is the sum of the capital cost for repair and the projected cost for operation/maintenance of the repair system, including adjustment for interest rates and inflation. Any indirect cost associated with loss of revenues due to interruption of daily operations during the repair and maintenance periods should also be included. Often, it is necessary to prepare a detailed estimate for comparing the relative benefits of repeated repairs and extended repair. In many cases, due to budgetary constraints, a less desirable solution is chosen instead of the optimal one.[7]

1.6 RESEARCH AND DEVELOPMENT NEEDS

In the past decade or so, there has been a worldwide effort into developing new construction materials and techniques for rehabilitating deteriorated concrete structures. The main thrust of development, however, has been in the field of synthetic construction materials. Technologies for applying these materials are for the most part improvements on existing construction techniques, although some new methods have been put forth. Nevertheless, upgrading the existing infrastructure and building more durable structures in the future does not rest on developing materials alone. The available research and development capabilities must be applied to the innovative questions associated with alternative infrastructure technologies.

Numerous studies conducted in the mid 1980s assessed research needs in various categories concerning infrastructure management. The studies focussed on both management and technological issues. For instance, a common conclusion was that the process of public works management and the effectiveness of public works managers needed to be vastly improved. This would require an analysis of institutional problems of planning and management of facilities, including the process used for decision making.

The identification of the effects of new technology on urban infrastructure are important factors which need to be considered to improve the performance and reduce the cost of existing systems and facilities. This would include adjusting infrastructure management for future patterns of living. Equally important is to develop standards and criteria for the design and performance of urban public facilities, against which national and local needs for investment can be measured. Also, constraints caused by the existing codes and standards should be minimized.

On the technological front, there is a need for technologies far beyond those in use today for quality assurance in construction. The construction industry is becoming worldwide, demanding access to the world's best products and services. A prerequiste to acceptance of products or services in international trade is a demonstration that they conform to international safety and performance standards. This may be achieved through highly developed "infratechno-logies".[9] These are tools used by engineers through the entirety of the life cycle of the structure or facility for developing and applying information, standards, codes and quality assurance. Infratechnologies required to demonstrate such conformance will consist of:

- Performance standards and codes

- Procedures to assess the conformity of innovative products and services
- Computer-based knowledge systems
- Automatic information exchange systems
- Open systems for products and services

Predicting the remaining service life of existing deteriorated concrete structures has gained interest in recent years. Present methods and tools to predict future performance and service are limited. There is a great need for refinement of available life prediction methods and service life design criteria must be further defined. In a recent review of some case histories, Hookham[10] cited the following important research needs:

- Further research is needed to characterize degradation processes in terms of their rate of attack, threshold level, and treatment in life prediction models.
- Developing appropriate accelerated aging techniques and tests is needed to improve mathematical degradation models instead of empirical data
- Defining periodic nondestructive testing and inspection methods and appropriate acceptance criteria are necessary to provide the data required to predict the remaining service life.
- The combined effects of several deterioration processes acting simultaneously needs to be investigated and included in life prediction modelling.

It should be realized that there are thousands of research needs in all areas of infrastructure management, and the only source for satisfying this demand is dedicated research efforts by the engineering community. However, the role of the government and the public in providing the necessary support for accomplishing such an enormous task cannot be underemphasized.

1.7 MARINE STRUCTURE TYPES

Marine structures have always been critically important to the operation and economic growth of all nations. They have been in existence for centuries and have been constructed of stone, timber, concrete, and steel to withstand the harshness of the marine environment. They are not only designed to carry their service loads, but loads from ship and wave impact as well. Although the average service life of a marine structure is approximately 25 years, marine structures 50, 75 and over 100 years old are still in service providing the necessary means for movement of goods, vital to a nation's growth.[11] Therefore, it becomes imperative to keep these facilities operating at a serviceable level and to maintain their capacities.

Marine structures include a variety of structures and are normally grouped either according to their function or their general layout and overall geometrical configuration.[12] The term is normally applied to berthing and mooring facilities, container terminals and oil jetties, breakwaters, embankments, slope protection structures, tidal barriers, dams, navigation locks, and outlet tunnels. It is not the intent of this thesis to provide an exhaustive review of all the types of marine structures in use, but to summarize the main features of those most commonly encountered in the marine environment. These include:[13]

- Berthing facilities for mooring and providing support to ships and craft.
- Drydocks used for construction of ships and to expose the underside of ships for inspection, maintenance, repair, or modification.
- Coastal protection structures designed to protect shorelines or harbors.

1.7.1 BERTHING FACILITIES

The basic berthing facilities that provide berthing support for ships and craft are piers (normal to the shore) and wharves (parallel to the shore). Piers and wharves provide a transfer point for cargo and/or passengers between water carriers and land transport. The three major structural types for piers and wharves are open, solid, and floating.[14]

Open-type piers and wharves are pile supported platform structures which permit water to flow beneath. Solid-type piers consists of a retaining structure such as an anchored sheet pile wall or quaywall, behind which earth fill is placed to create a working surface. A floating-type pier is a pontoon structure that is anchored to the shore by trestles or ramps. The top of the pontoon can be utilized as the working deck. These structures are not affected by tidal fluctuations but obstruct water flow to some extent. Open and solid type structures can be combined to provide a more advantageous layout. These structures are discussed in more detail in the following sections.

1.7.1.1 PIERS

Piers are structures which extend outward from the shore into the waterbody. Piers may be used for berthing on one or both sides of their length. The length of a pier is usually equal to or greater than the length of the longest ship to be accommodated. The width of a pier is usually established by functional, geotechnical, and structural considerations.[14]

Open piers are pile-supported platform structures which allow water to flow underneath. Open piers are usually single-deck structures, although recently a double-deck pier was constructed by the U.S. Navy.[13] A schematic of a single and double-deck open pier is shown in Figures 1.5 and 1.6, respectively.

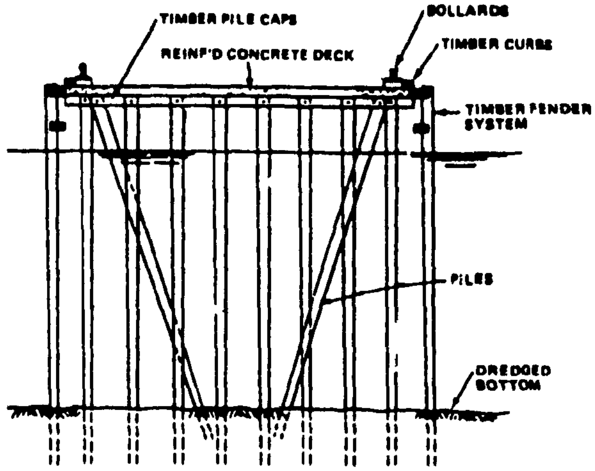


FIGURE 1.5 - SINGLE-DECK OPEN PIER[13]

Closed piers, or solid fill piers, are constructed so that water is prevented from flowing underneath. The solid fill pier is surrounded along its perimeter by a bulkhead or wall which retains the fill. A schematic of a typical solid fill pier is shown in Figure 1.7. A special type of solid fill pier is a mole pier. Mole piers are earth-filled structures that extend outward from the shore. The sides and offshore end of the pier are retained and protected by masonry or concrete sheet pile walls.[13]

Floating piers are connected to the shore with access ramps. To prevent lateral movement and allow vertical movement of the pier with the tidal fluctuations, guide piles in the center of the pier, or a chain anchorage system is utilized.

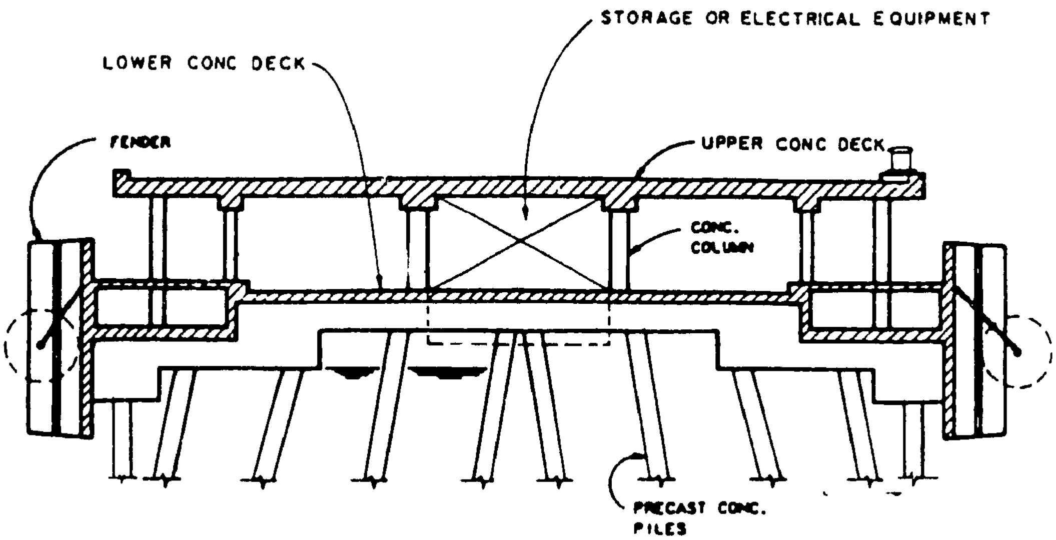


FIGURE 1.6 - TWO-STORY CONCRETE PIER[14]

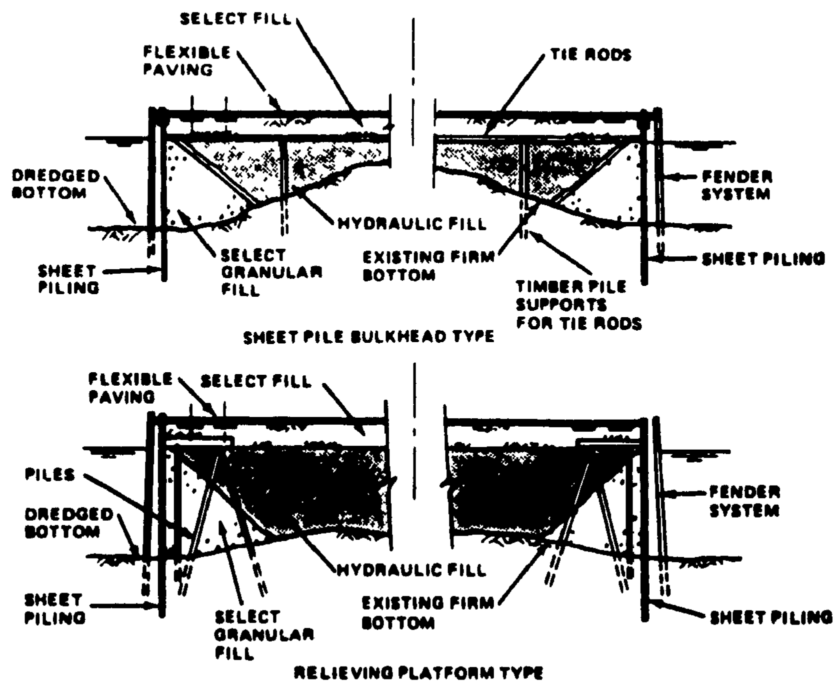
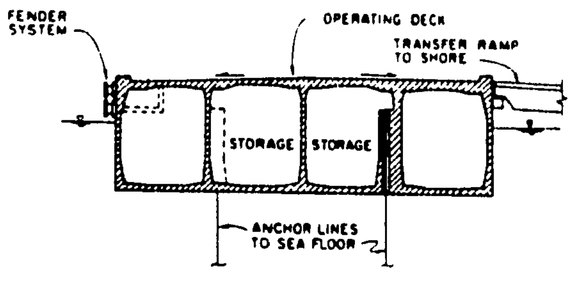
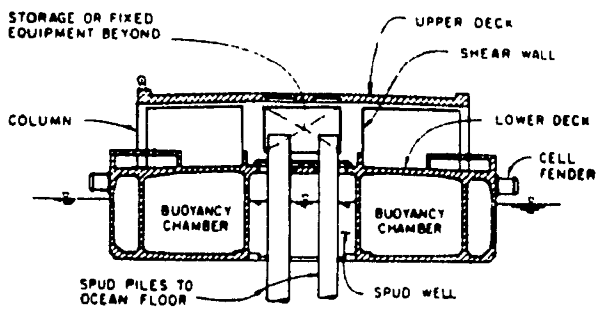


FIGURE 1.7 - SOLID FILL PIER TYPES[13]

The floating pier may also be a single or double-deck structure. A floating pier concept which was developed by the U.S. Navy is shown in Figure 1.8.[13] A more detailed discussion of the design and configuration of piers is provided in the U.S. Navy Design Manual NAVFAC DM 25.01.[15]



(A) FLOATING PONTOON TYPE (WHARF)



(B) FLOATING TWO STORY TYPE (PIER)

FIGURE 1.8 - FLOATING TYPE PIERS[14]

1.7.1.2 WHARVES

Wharves are structures which are constructed approximately parallel to the shore. A marginal wharf is attached to the shore along its full length and a retaining structure is used to retain earth or stone placed behind the wharf. The retaining structure is usually referred to as a bulkhead or quaywall. With this structure, the ships can berth along the outshore face only. The typical wharf types are similar to the basic pier types and include open and closed (or solid fill) layouts.[13]

Wharf length and width is based on the same considerations as those for piers. Examples of open and solid fill wharves are shown in Figures 1.9 and 1.10, respectively.

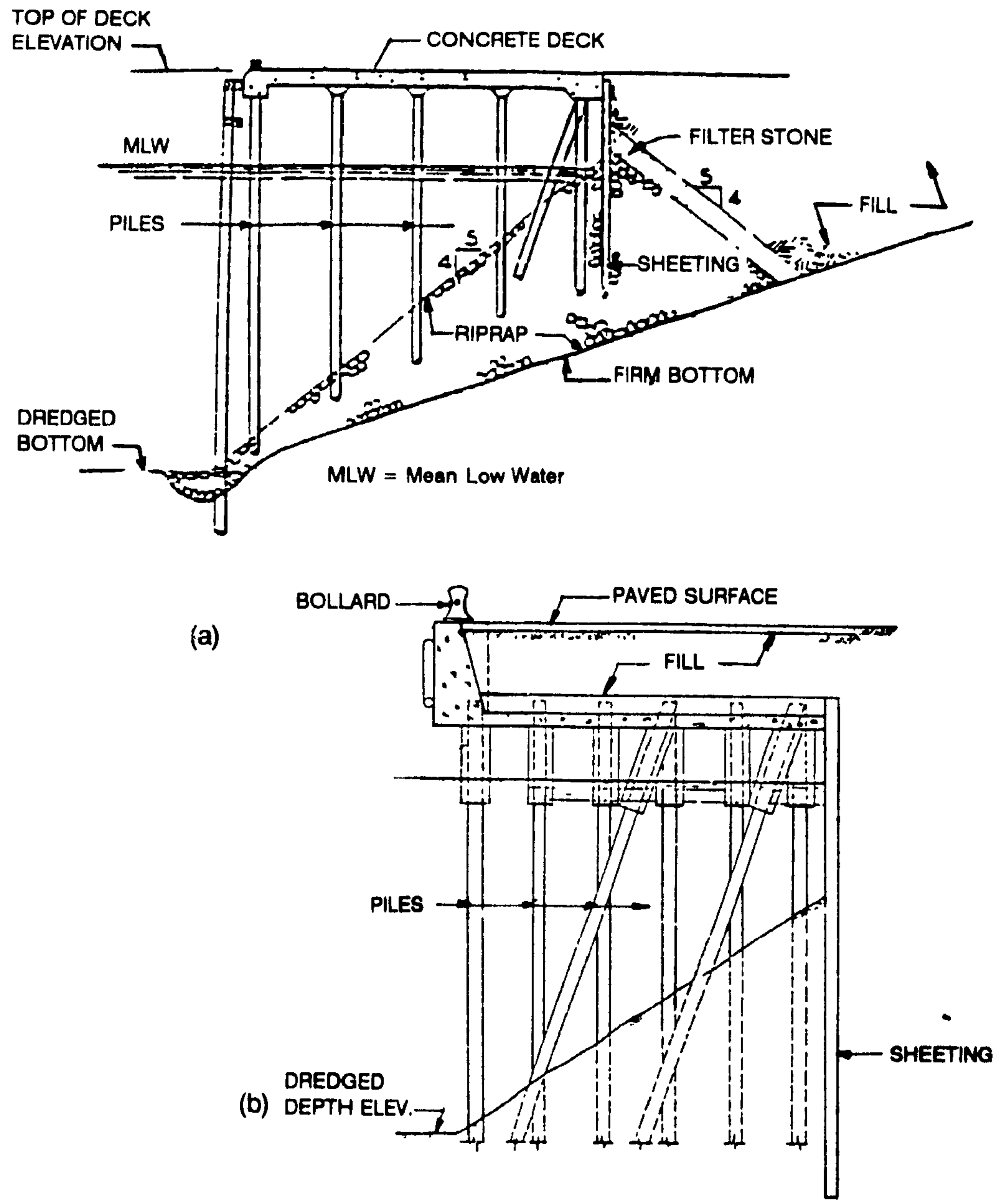


FIGURE 1.9 - OPEN WHARVES: (a) HIGH-LEVEL FIXED WHARF (b) RELIEVING PLATFORM TYPE WHARF[24]

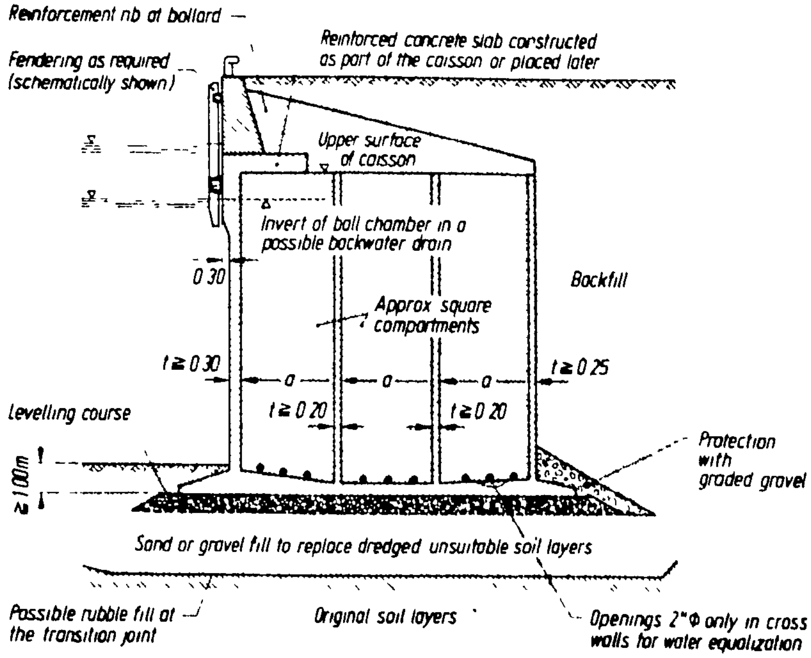


FIGURE 1.10 - SOLID FILL CAISSON WHARF[25]

When the depth of water adjacent to the shore is too shallow for deep draft ships, the wharf, which consists of a platform on piles, is located some distance away from the shore in deeper water and is attached to the shore by pile-supported trestles. The trestles are usually oriented perpendicular to the wharf (Figure 1.11). If the trestle is located at the center of the wharf, the structure is referred to as a T-type wharf; if the trestle is located at an end, it is called an L-type wharf; and if trestles are located at both ends, it is referred to as a U-type wharf.[14]

1.7.2 DRYDOCKING FACILITIES

Drydocking facilities are used for construction of ships and to expose the underside of ships for inspection, maintenance, repair, or modification. There are various types of drydocks that exist, including graving drydocks, floating drydocks, marine railways, and vertical syncrolifts.[13] These are briefly described below. A more detailed discussion of drydocking facilities can be found in References 16 through 18.

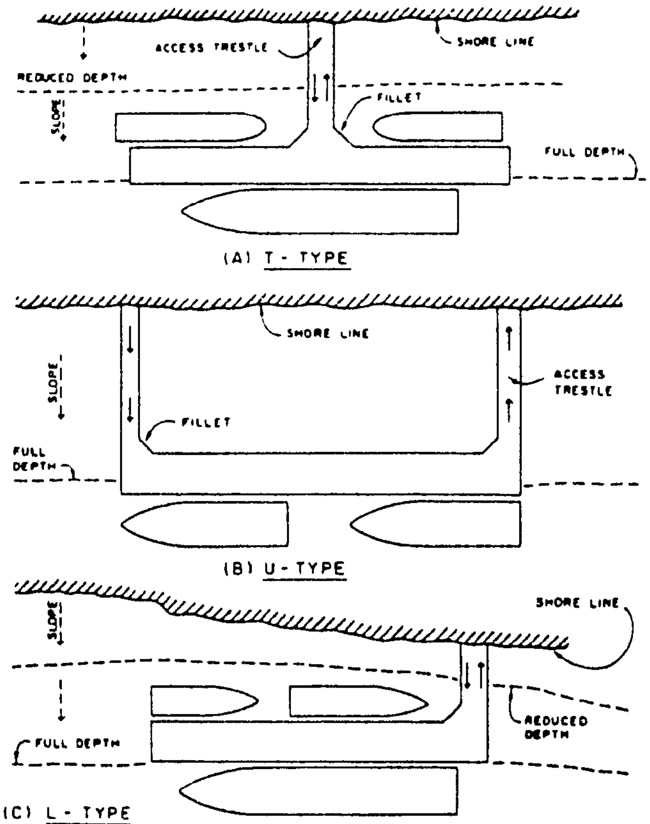


FIGURE 1.11 - MARGINAL WHARF TYPES[14]

1.7.2.1 GRAVING DRYDOCKS

Graving drydocks are fixed basins adjacent to the shore and are constructed of stone, masonry, concrete, or sheet pile cells. They can be closed off from the outshore side by a movable water-tight barrier (entrance caisson or flap gate). After the barrier is closed, the water is pumped out of the basin to allow the ship to settle on blocking set on the dock floor. A schematic of a typical graving dock is shown in Figure 1.12.

1.7.2.2 FLOATING DRYDOCKS

Floating drydocks are U-shaped structures that are used to raise ships or vessels out of the waterway. The structure is flooded, permitting the vessel to enter, and then it is pumped dry.

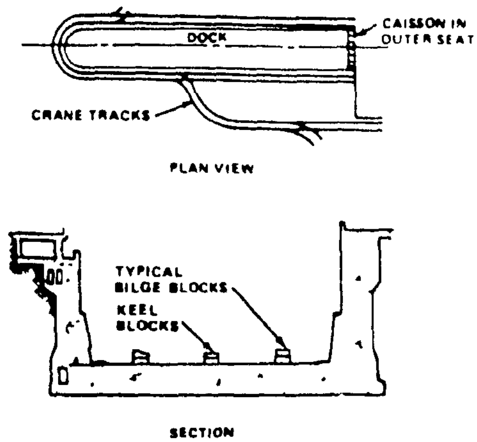


FIGURE 1.12 - TYPICAL GRAVING DRYDOCK[13]

1.7.2.3 MARINE RAILWAYS

Marine railways provide an access for a vessel to enter the waterway from land and vice versa Marine railways consist of a ramp which extends into the water, a mobile ship cradle on wheels or rollers, groundway ship cradle tracks, hoisting machinery, and chains or cables for hauling the ship cradle. A typical marine railway is shown in Figure 1.13.

1.7.2.4 VERTICAL SYNCROLIFTS

Vertical syncrolifts consist of platforms which are lowered into the water to receive ships. The ship is then lifted out of the water on the platform by electrically powered hoisting equipment. Figure 1.14 shows a typical vertical syncrolift drydocking system.

1.7.3 COASTAL PROTECTION STRUCTURES

The primary function of these structures is to protect harbors from the erosive effects of wave action. Structures which commonly fall in this category include seawalls, bulkheads, groins, jetties, and breakwaters. A brief description of each follows; more detailed information on the design and configuration of these structures is available in References 19, 20, and 21.

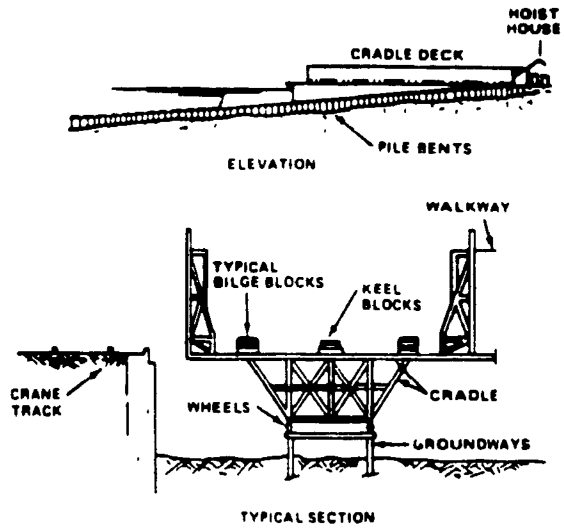


FIGURE 1.13 - TYPICAL MARINE RAILWAY[13]

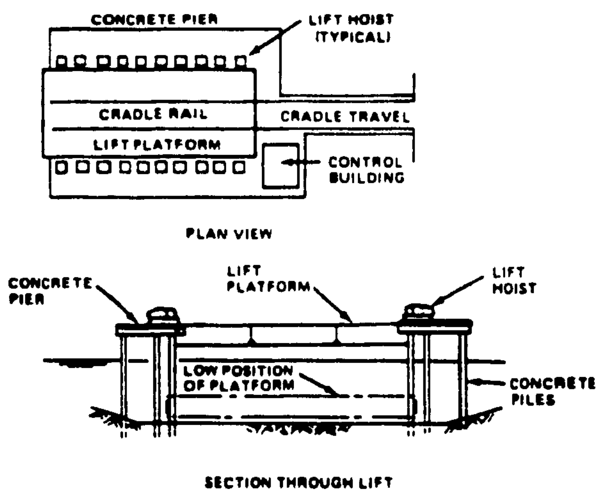


FIGURE 1.14 - TYPICAL VERTICAL SYNCROLIFT[13]

1.7.3.1 SEAWALLS

Seawalls are massive coastal structures built along the shoreline to protect coastal areas from scour caused by severe wave action and flooding during storms.[19] Seawalls are constructed of a variety of materials including reinforced concrete, rubble mounds, or granite masonry. Figure 1.15 shows three basic types of seawall configurations.

A curved-face seawall (Figure 1.15a) uses a sheet pile cut-off wall to prevent loss of foundation material by wave scour and leaching from overtopping water or storm drainage beneath the wall. The toe of the curved-face seawall consists of large stones to prevent or reduce scour

The stepped-face seawall (Figure 1 15b) is designed for stability against moderate waves. This seawall type uses reinforced concrete sheet piles with tongue-and-groove joints The space that is created between the piles may be filled with grout to form a sandtight cut-off wall. Alternatively, a geotextile fabric can be placed behind the sheeting to provide a sandtight barrier, while permitting the water to seep through the cloth to prevent the buildup of hydrostatic pressure.

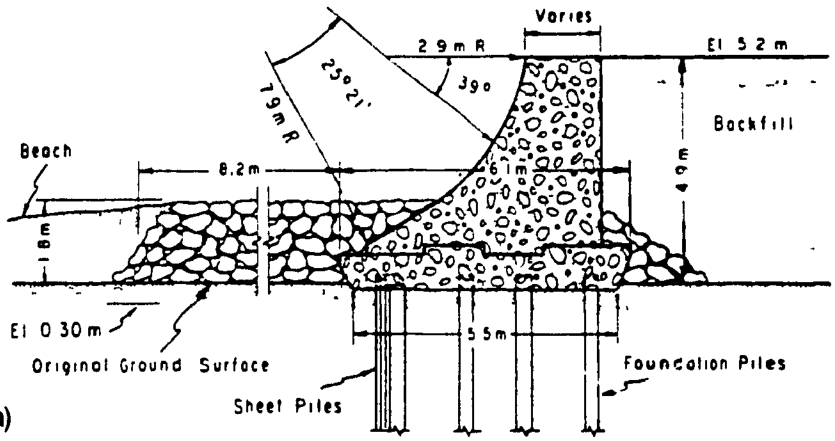
Rubble-mound seawalls can withstand severe wave action (Figure 1.15c). Although scour may occur, the quarrystone comprising the seawall can readjust and settle without causing structural failure.

1.7.3.2 BULKHEADS

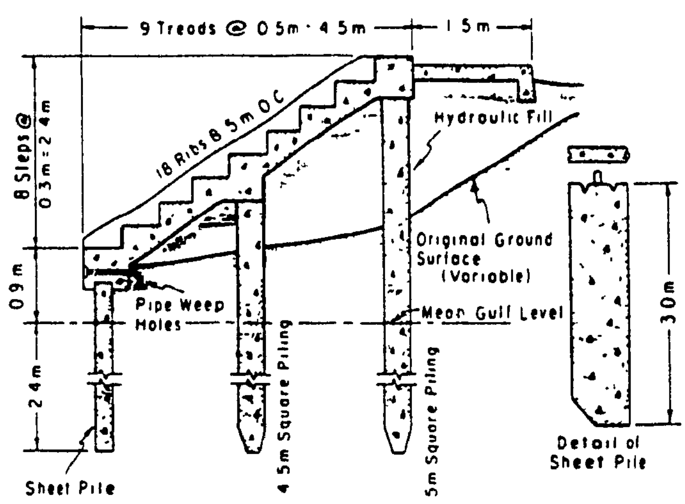
Bulkheads are flexible soil retaining structures which attain their stability from the structural members and the shear strength of the soil.[22] The primary function of bulkheads is to retain fill, and although not usually exposed to severe wave action, they are still required to resist erosion Bulkheads are generally either anchored vertical sheet pile walls or gravity structures.[19] The two basic structural types are shown in Figures 1.16 and 1.17. Cellular steel sheet pile bulkheads are sometimes constructed where rock is close to the surface and sufficient penetration cannot be provided for the anchored bulkhead type.

1.7.3.3 GROINS

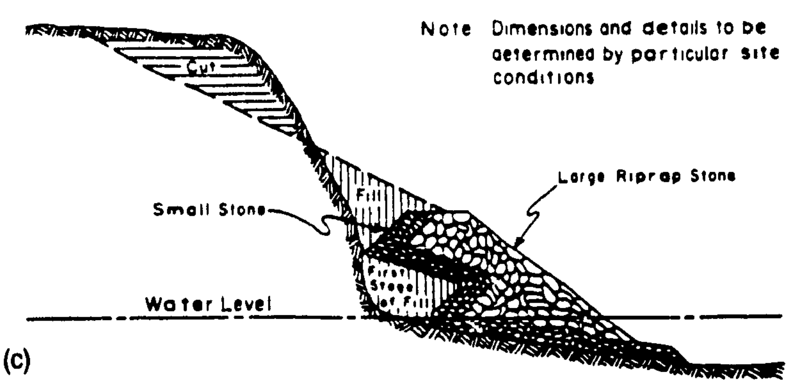
Groins are structures designed to reduce the effects of erosion to the shoreline by altering offshore current and wave patterns. Groins are normally constructed perpendicular to the shoreline and can be made permeable or impermeable.[19] Materials used for constructing groins include stone, concrete, timber, and steel.[13] Figure 1.18 shows an example of a concrete sheet pile groin.



(a)



(b)



(c)

FIGURE 1.15 - TYPICAL SEAWALL STRUCTURES (a) CURVED-FACE SEAWALL (b) STEPPED-FACE SEAWALL (c) RUBBLE-MOUND SEAWALL[19]

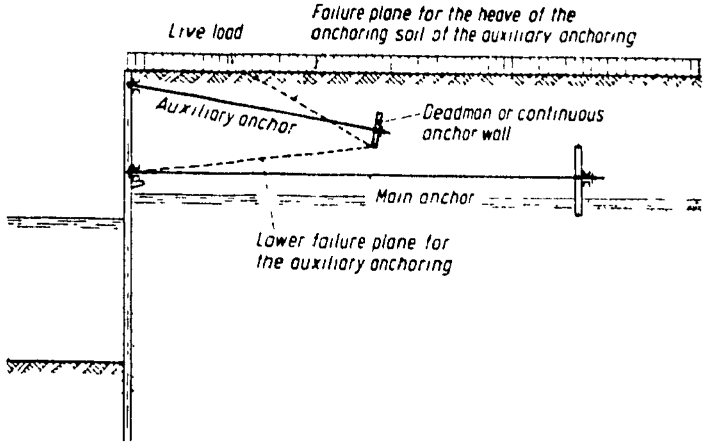


FIGURE 1.16 - ANCHORED VERTICAL SHEET PILE BULKHEAD[25]

1.7.3.4 JETTIES

Jetties are structures which extend from the shore into deeper water to prevent the formation of sandbars and control water currents. These structures are ordinarily located at the entrance to a harbor or a river estuary. Jetties are usually constructed of rubble mounds to a height above high tide.[13] A typical rubble-mound jetty configuration is shown in Figure 1.19.

1.7.3.5 BREAKWATERS

Breakwaters are large rubble-mound structures constructed outside of a harbor or coastline to protect inner shorelines from severe wave action. These barriers help to create a safe environment for mooring, operating, loading, or unloading of ships within the harbor. There are three general types of breakwaters, and may be either connected to or detached from the shore.[13] Figure 1.20 shows a cross-section of a typical rubble-mound breakwater.

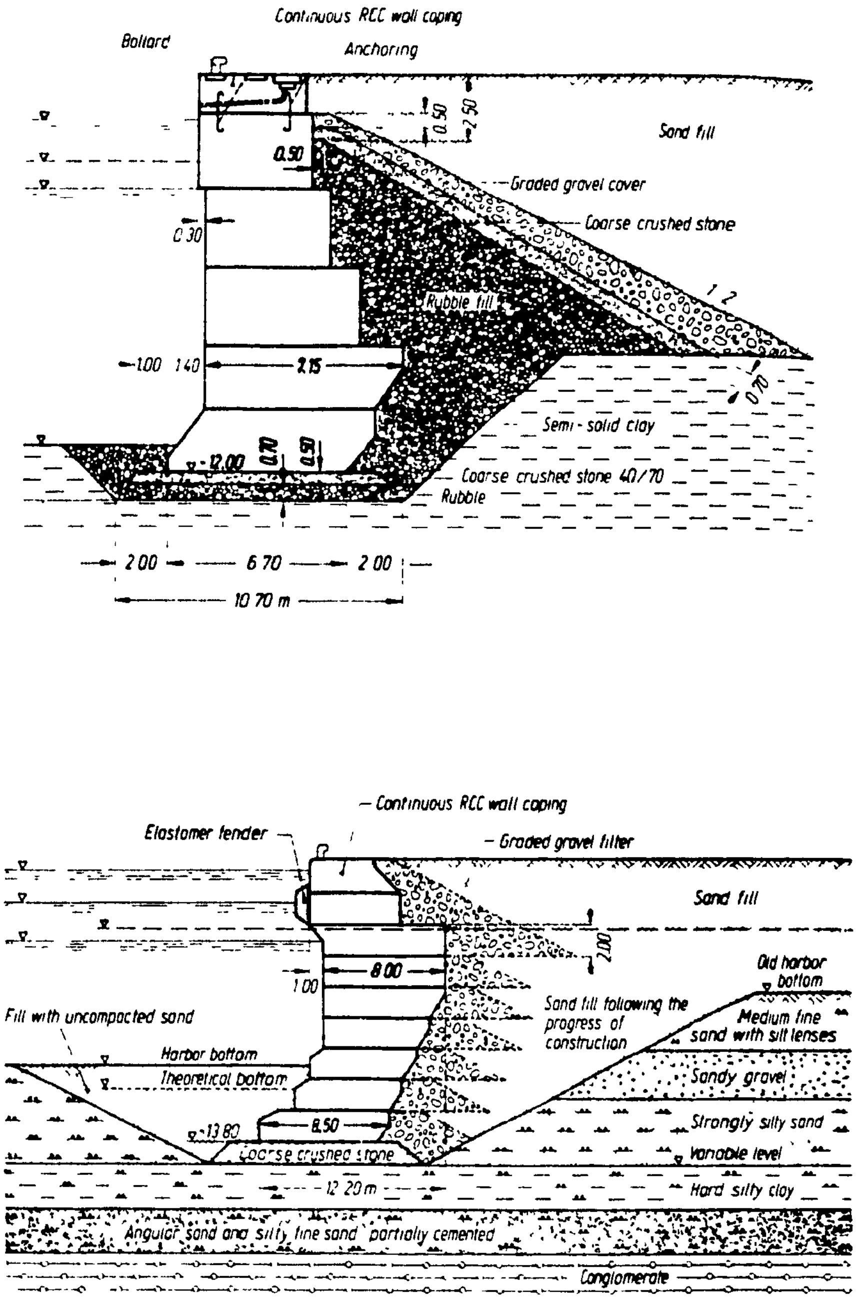


FIGURE 1.17 - GRAVITY BULKHEAD IN BLOCKWORK CONSTRUCTION[25]

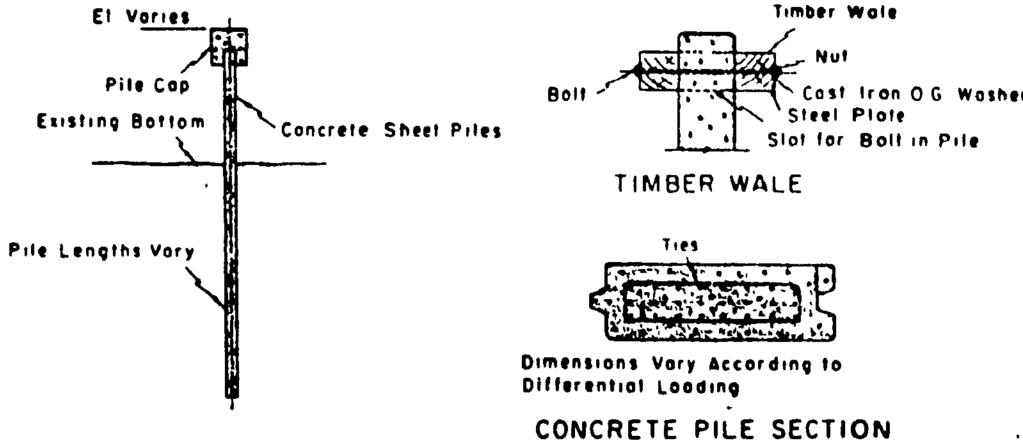


FIGURE 1.18 - PRESTRESSED CONCRETE SHEET PILE GROIN[19]

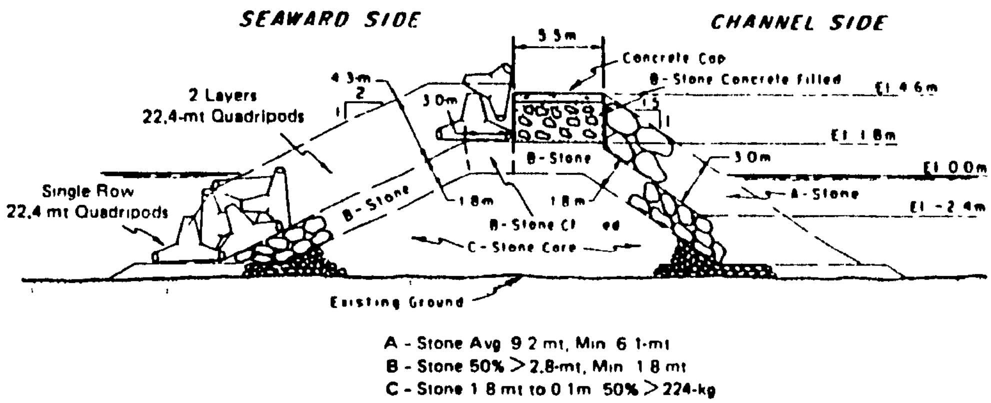


FIGURE 1.19 - QUADRIPOD AND RUBBLE-MOUND JETTY[19]

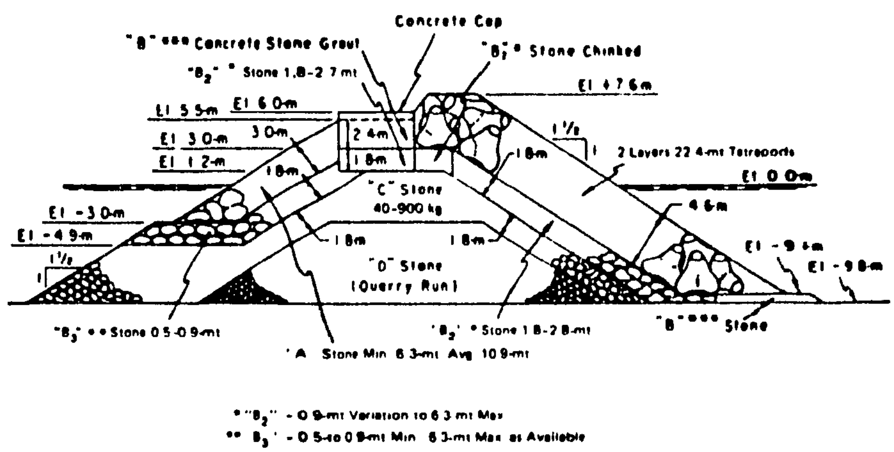


FIGURE 1.20 - TETRAPOD AND RUBBLE-MOUND BREAKWATER[19]

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CHAPTER 2

DETERIORATION OF CONCRETE IN THE MARINE ENVIRONMENT

2.1 INTRODUCTION

A vast number of concrete structures are in direct contact with seawater or are exposed to seawater spray carried by winds.[1] The marine environment is one of the harshest environments known to man. Concrete exposed to the marine environment may deteriorate due to the combined effects of various physical and chemical phenomena. Some of the most common forms of deterioration include, chloride-induced corrosion of the reinforcing steel, freeze-thaw attack, alkali-aggregate reaction, sulfate attack, and physical erosion due to wave action and floating debris.

According to a study of case histories of concrete failures in seawater (Appendix A), investigators have determined that the degree of deterioration (physical or chemical) is dependent on where the structural member is located with respect to tidal activity. Therefore, Mehta[2] grouped marine concrete into three exposure zones: submerged, splash, and atmospheric (Figure 2.1). The atmospheric zone, which is always exposed to the atmosphere, is susceptible to cracking by several causes including, freeze-thaw action, wetting and drying, thermal cycles, and corrosion of embedded steel reinforcement. Also, concrete in the tidal or splash zone, which is located between high and low tide, may experience cracking by impact of floating debris and by deleterious chemical reactions between the seawater and cement paste constituents. The submerged zone, which is continuously covered with seawater, is susceptible to chemical attack only.

Clearly, the most severe deterioration will occur in the tidal zone because the structure is exposed to nearly all the physical and chemical attacks. Concrete deterioration caused by any one of these phenomena will increase the permeability of concrete which will cause further deterioration by other types of attack. This chapter outlines the various physical and chemical phenomena which cause deterioration in the marine environment, and measures to control such deterioration are also presented. A brief review of concrete deterioration by bacteriological attack and hard impact are also provided. The material in this chapter has been adapted from different available references, especially 1, 4, 6 and 45, and is presented here for completeness.

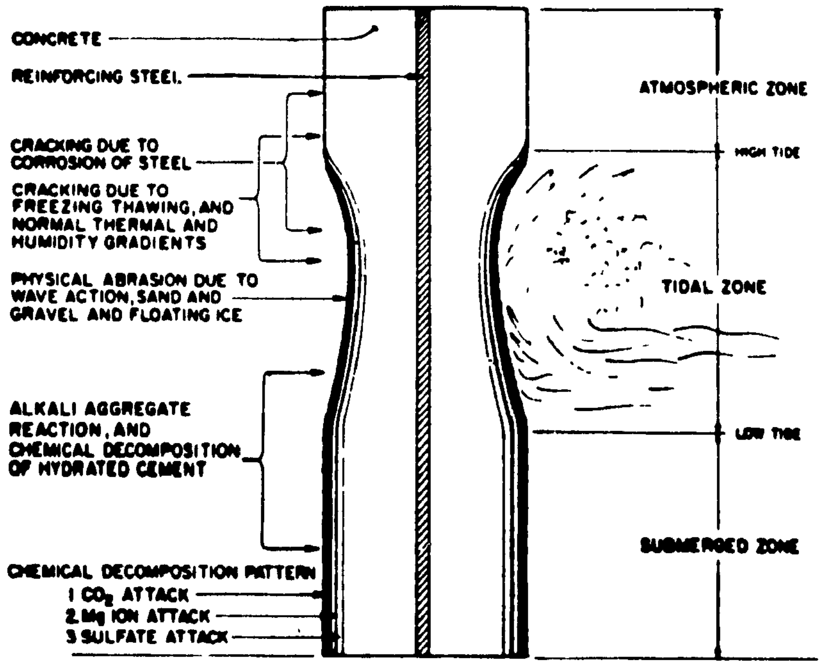


FIGURE 2.1 - EXPOSURE ZONES IN A MARINE ENVIRONMENT[1]

2.2 CONCRETE DETERIORATION DUE TO PHYSICAL PROCESSES

According to Mehta and Gerwick,[3] there are two classifications of physical causes of concrete deterioration: surface wear and cracking (Figure 2.2). The various phenomena in each classification are discussed in the following sections.

2.2.1 CRACKING

The major causes of concrete deterioration are attributed to cracking and subsequent corrosion of embedded reinforcing steel (Section 2.3).[3] The causes and types of cracking are many and can occur in both plastic and hardened concrete. An excellent review of the causes, mechanisms, and control of all types of cracking in concrete is provided by the ACI Committee 224.[4,5] The basic mechanisms by which cracking strains may be generated in concrete are:[6]

- Internal movements caused by drying shrinkage, plastic settlement or shrinkage, and expansion or contraction.

- Expansion of embedded metals, such as reinforcement corrosion.
- External loading conditions, such as deformations caused by differential foundation settlement.

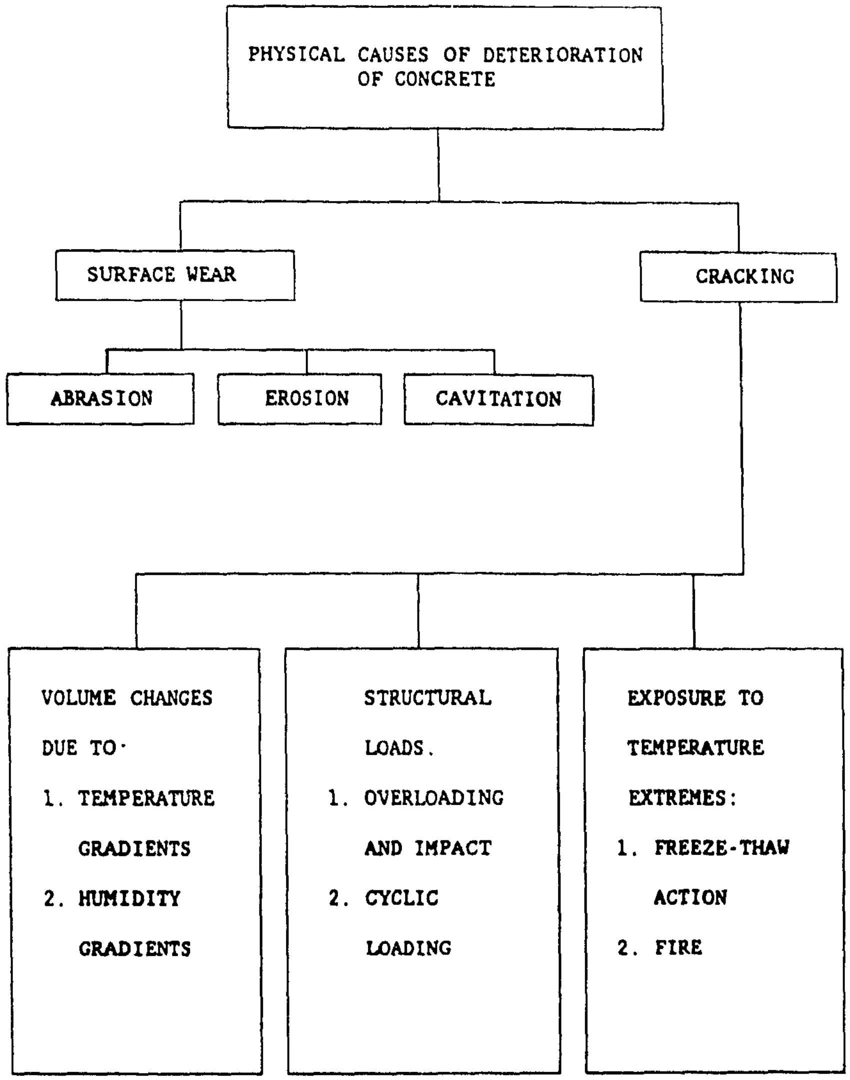


FIGURE 2.2 - PHYSICAL CAUSES OF CONCRETE DETERIORATION[7]

A summary of the various possible causes of cracking is provided in Figure 2.3. A general guide of the age at which these cracks occur in concrete is given in Figure 2.4. A summary of common defects occurring during construction is provided in Appendix B. The various types of cracks which occur most often in practice are summarized below, and are adapted from a review of References 4, 5, 6, and 7.

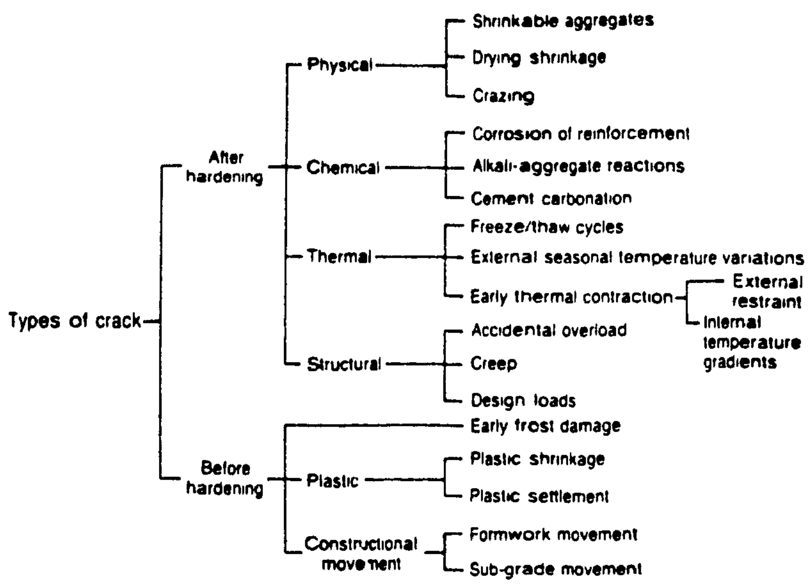


FIGURE 2.3 - TYPES OF CRACKS IN CONCRETE[6]

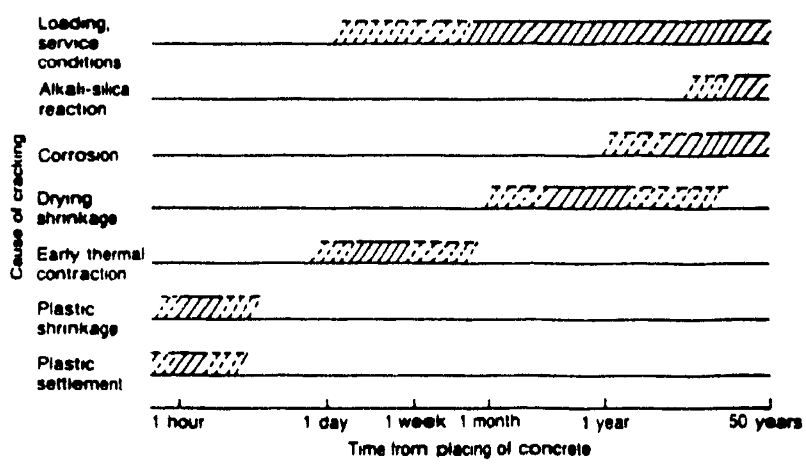


FIGURE 2.4 - TIME OF APPEARANCE OF CRACKS FROM PLACING OF CONCRETE[6]

2.2.1.1 PLASTIC SHRINKAGE CRACKING

Plastic shrinkage cracking usually occurs on the surface of freshly poured concrete when it is subjected to rapid loss of moisture. Cracking usually occurs within the first two to four hours after placement if the loss of moisture exceeds the supply by bleed water

The subsequent shrinkage at the surface and the restraint provided by concrete below the drying surface layer induce tensile strains which cause cracking. These cracks are usually short and run in several directions (Figure 2.5). The width of a typical crack is about 2 to 3 mm (0.08 to 0 12 in.) at the surface and decreases as the depth from the surface increases. The length of the cracks can vary from a few centimeters to over 1 m (3 ft.) in length and are spaced from a few centimeters to as much as 3 m (10 ft.) apart. In some cases, plastic shrinkage cracks can extend the full depth of the member.

Measures to prevent plastic shrinkage cracking include the use of "fog nozzles" to humidify the concrete surface and covering the concrete surface with plastic sheeting. The concrete can also be protected by erecting wind breakers to diminish the wind velocity, and sunscreens to lower the surface temperature. ACI Committees 224R,[5] 302.1R[8] and 305R[9] provide other recommendations to prevent rapid moisture loss.

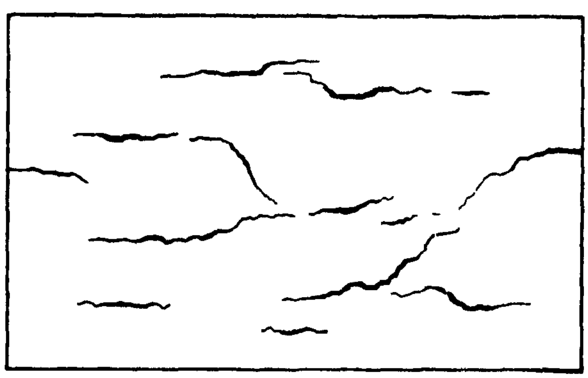


FIGURE 2.5 - TYPICAL PLASTIC SHRINKAGE CRACKING[4]

2.2.1.2 SETTLEMENT CRACKING

After the concrete is placed and compacted, it will continue to consolidate due to the movement of mixing water toward the surface. If settlement of concrete is restrained by reinforcing steel or formwork, cracking will occur near the element where it is being restrained. In the case of reinforcing steel, longitudinal settlement cracks will form along the top of the rebar (Figure 2.6). Increasing the rebar size and slump, and decreasing the concrete cover will increase settlement cracking. If the reinforcing bars are closely spaced, horizontal settlement cracking may occur (Figure 2.7). These cracks over the top layer of the reinforcement will cause the concrete cover to spall. Steps to prevent settlement cracking include proper form design (ACI 347R)[10], adequate compaction, the use of the lowest possible slump, and increasing the concrete cover.

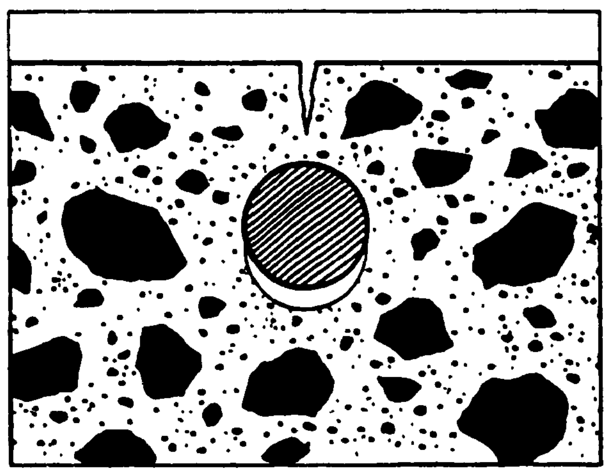


FIGURE 2.6 - SETTLEMENT CRACKING[4]

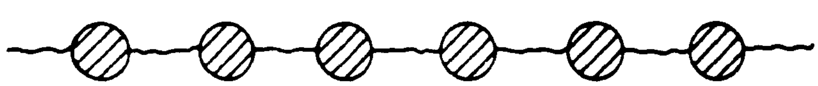


FIGURE 2.7 - HORIZONTAL SETTLEMENT CRACKING BETWEEN CLOSELY SPACED REINFORCING BARS[6]

2.2.1.3 DRYING SHRINKAGE

Restrained drying shrinkage is caused by loss of moisture from the cement paste constituents. The degree of drying shrinkage primarily depends on the amount and type of aggregate and the water-cement ratio of the mix. As the amount of aggregate is increased the amount of shrinkage will decrease. Surface crazing on walls and slabs is an example of drying shrinkage. This often occurs when the concrete near the surface contains a higher water content than the interior concrete, resulting in a series of shallow, closely spaced, fine cracks.

Drying shrinkage can be minimized by incorporating the maximum practical amount of aggregate and the lowest possible water content in the mix, or using shrinkage-compensating cement. However, this requires careful control and proper consolidation. The use of properly spaced contraction joints is an effective means of controlling shrinkage cracking. ACI Committee 224R[5] provides more details and other construction practices which help to control drying shrinkage in concrete.

2.2.1.4 THERMAL CRACKING

Hydration of cement paste and changes in ambient conditions may cause thermal gradients within a concrete structure, which will in turn create differential volume changes. These differential volume changes will create tensile strains that may exceed the tensile strain capacity of concrete, causing it to crack. Mass concrete structures, such as piers, wharfs, and dams are prone to thermal cracking. The larger the structure, the greater the risk for thermal gradients. The cracks are usually found on the surface of the concrete, mostly in the form of map cracking, and are normally a few millimeters or centimeters deep.

Reducing the temperature of the internal concrete, delaying the start of cooling, controlling the rate at which the concrete cools, and increasing the tensile strain capacity of the concrete, all help to reduce thermal cracking. These and other methods to reduce cracking in mass concrete are discussed in ACI 207.1R,[11], ACI 207.2R,[12], and ACI 224R.[5]

2.2.1.5 STRUCTURAL CRACKING

Structural cracks can occur as a result of externally applied loading either during construction or during the service life of the structure. Construction loads are often significantly more than service loads. Since these conditions usually occur when the concrete is most vulnerable to damage, the cracks which develop are usually permanent. Overstressing the concrete locally may also cause the concrete to crack. For instance, concentrated wheel loads may cause cracking along the direction of the reinforcing bar as a result of high bond stresses. Concentrated loads at anchorages of prestressing tendons can cause cracking along the direction in which the load is applied. Figure 2.8 summarizes the various forms of load-induced cracking.

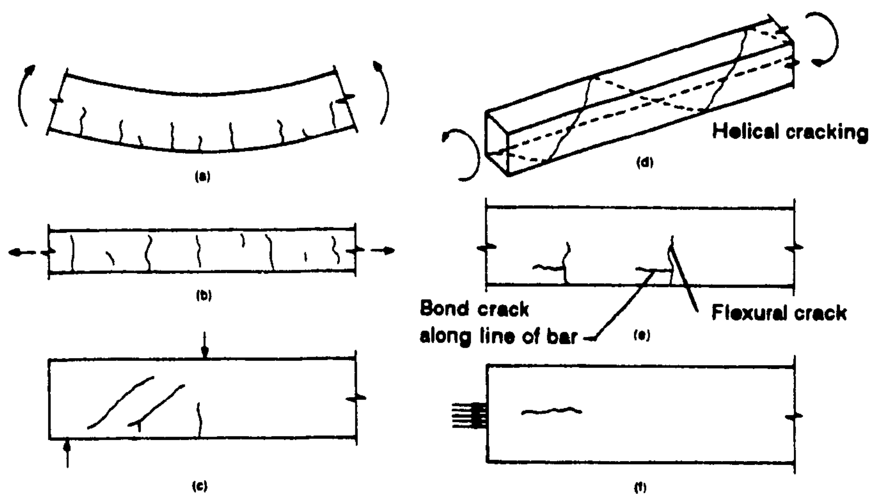


FIGURE 2.8 - LOAD-INDUCED CRACKS; (a) PURE FLEXURE; (b) PURE TENSION; (c) SHEAR; (d) TORSION; (e) BOND; (f) CONCENTRATED LOAD[6]

2.2.1.6 CRACKING DUE TO CHEMICAL REACTIONS

There are several deleterious chemical reactions which may cause cracking in concrete. These reactions may be caused by reactive aggregates in the concrete or substances that come into contact with the hardened cement paste. Certain aggregates containing active silica react with the alkalies found in the hydrated cement paste to form an expansive silica gel. The resulting local stresses which occur as a result of this expansion causes the concrete to crack and may often lead to complete deterioration. Water which contains sulfates also reacts with the portland cement paste constituents to form an expansive product resulting in high local stresses which cause the concrete to crack and deteriorate. Deterioration may also occur from the repeated application of deicing salts to the concrete surface. The effects of these and other chemical reactions relating to the durability of concrete are discussed in greater detail in Section 2.4.

2.2.1.7 CRACKING DUE TO REINFORCEMENT CORROSION

Reinforcing steel in concrete is usually protected by a passive oxide coating which forms in the highly alkaline pore solution in hydrated concrete. However, reinforcing steel may corrode if the alkalinity (pH) of the concrete is reduced by carbonation or by destruction of the passive film by aggressive ions such as chlorides. The resulting steel corrosion produces expansive products which occupy a much greater volume than the original steel. This increase in volume causes high radial bursting stresses around reinforcing bars which lead to cracking of the surrounding concrete. The principles of reinforcement corrosion are discussed in more detail in Section 2.3

2.2.1.8 CRACKING DUE TO CRYSTALLIZATION OF SALTS IN PORES

Crystallization of sulfate salts in concrete pores can lead to significant damage. This occurs when one side of a concrete member is exposed to a salt solution and the other sides are exposed to the atmosphere. Many porous materials are susceptible to cracking from crystallization pressures.[1]

2.2.1.9 CRACKS CAUSED BY DESIGN AND DETAILING ERRORS

Errors in design and detailing may lead to unacceptable cracking of concrete. The effects of cracking range from "poor appearance to lack of serviceability to catastrophic failure".[4] Common errors in design and detailing that may lead to cracking include poorly detailed corners, sudden changes in cross-sectional area, improper selection and/or detailing of reinforcement, member restraints, insufficient number of contraction joints, and improper design of foundations, leading to differential settlement. The degree to which improper design and detailing will cause cracking depends on the particular structure and loading conditions involved.

2.2.1.10 WEATHERING

The various weathering processes which cause concrete to crack include freezing and thawing, wetting and drying, and heating and cooling. All of these processes create volume changes in the concrete which lead to excessive cracking. The best defence against deterioration due to natural weathering is to provide a concrete with the lowest practical water-cement ratio, durable aggregate, adequate air entrainment, and proper curing. A more detailed discussion of these forms of deterioration is provided in the following sections.

2.2.2 FREEZING AND THAWING

Deterioration of concrete due to freeze-thaw action is one of the major durability problems with structures in cold climates. The cause, rate of deterioration, and extent of damage depend on the characteristics of the concrete pore matrix and specific environmental conditions. The frost damage to concrete usually manifests in cracking and spalling, and scaling. Cracking and spalling are the most common forms of damage and are caused by continuous expansion of the concrete from repeated freeze-thaw cycles. Scaling usually occurs as a result of freeze-thaw action in the presence of deicing salts.[1] The various mechanisms by which frost damage occurs in the cement paste are described below. Since hardened cement paste and aggregate particles in the concrete behave differently when subjected to freeze-thaw action, these are described separately.

2.2.2.1 FROST ACTION ON HARDENED CEMENT PASTE

Powers[13,14] theorized that frost damage in cement paste is caused by hydraulic pressures generated during the freezing of water in the capillaries or pores of the concrete. When water begins to freeze there is a corresponding increase in volume of nine percent.[1,6] The resulting hydraulic pressure depends on the distance to an "escape boundary", the permeability of the concrete, and the rate of ice formation. In the case of completely filled water pores, the concrete will crack.

Powers also suggested that osmotic pressure, caused by differences in salt concentrations in the pore fluid, can be another source of destruction in cement paste.[1] Since solutions freeze at lower temperatures than water, the higher the salt concentration in the pore fluid, the lower the freezing point (Figure 2.9). When the temperature of the concrete drops below the freezing point, ice crystals will form in the large capillaries, resulting in "an increase in the alkali content in the unfrozen portion of the solution in these capillaries".[15] An osmotic potential is created which causes water in the adjacent unfrozen pores to move towards the solution in the frozen pores (Figure 2.10). This decreases the alkali content of the solution adjacent to the ice and causes the ice crystals to grow (ice-accretion). When the pore is completely filled with ice and solution, any further crystal growth produces expansive pressures which eventually lead to destruction of the cement paste. According to the theory proposed by Litvan,[16] the process explained by Powers causes a portion of the paste in the unfrozen regions to dry up, and the frozen regions to expand. In addition, damage occurs when the moisture within the pores is not adequately redistributed to accommodate the conditions, either due to a high degree of water saturation, rapid cooling, or low permeability.[1] In such cases, when the pore water freezes it forms a semi-amorphous solid which produces high internal pressures.[15]

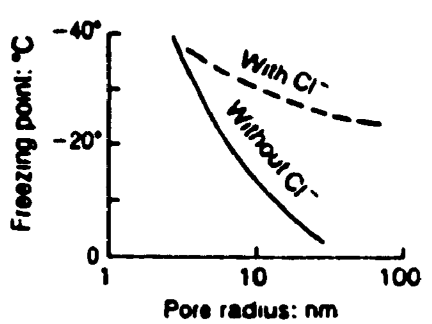


FIGURE 2.9 - EFFECT OF CHLORIDES ON THE FREEZING POINT OF PORE WATER[6]

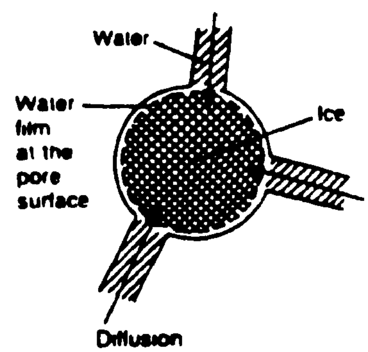


FIGURE 2.10 - DIFFUSION DURING COOLING[6]

2.2.2.2 FROST ACTION IN AGGREGATE PARTICLES

Powers[17] found that aggregates bleed internal water during freezing, and the hydraulic pressure theory developed to explain the damage to cement paste by frost action is also believed to be applicable to porous aggregates, such as sandstones, shales, and certain cherts. The behavior of an aggregate particle when exposed to freeze-thaw action depends on the pore size distribution and permeability.[1] With regard to resistance to frost action, Verbeck and Landgren[18] proposed three classes of aggregate: low permeability, intermediate permeability, and high permeability. The first category includes aggregates of low permeability and high strength. In these aggregates, when water in the pores freezes, the elastic strain in the aggregate particle is absorbed without causing any damage.

Aggregates of intermediate permeability have a significant amount of small pores (500 nm or smaller). Verbeck and Landgren[18] showed that for any natural rock, there is a critical particle size below which internal water can be frozen without damage. There is no unique critical size for aggregates because this depends on several factors including the degree of saturation, freezing rate and permeability of the aggregate. However, some aggregates, such as granite, basalt, diabase, quartzite, and marble do not produce stress when freezing occurs regardless of the particle size.[15] If aggregates larger than the critical size are used in concrete, the primary failure mode is accompanied by pop-outs, as shown in Figure 2.11.[1]

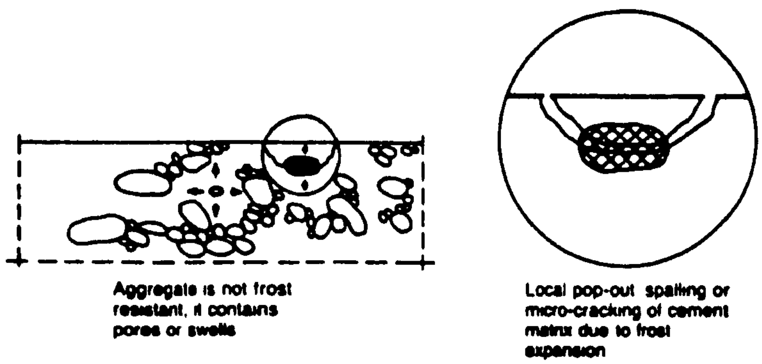


FIGURE 2.11 - POP-OUT DUE TO NON-FROST-RESISTANT AGGREGATE[8]

Aggregates having a significant number of large pores are considered to be highly permeable. Although the high permeability of the aggregate allows water to penetrate and exit easily, frost damage can still occur. When pressurized pore water is forced out from an aggregate particle, the interface between the aggregate surface and cement paste may be damaged. In this case, frost action does not cause damage to the aggregate particles.[1]

2.2.2.3 CONTROL OF FREEZE-THAW DAMAGE

The characteristics of the cement paste and aggregate both have an effect on the frost resistance of concrete. In each case, the resulting behavior is dependent on the interaction of several factors, such as the location of escape boundaries, pore structure, the degree of saturation, the rate of cooling, and the tensile strength of the concrete. Providing air entrainment in concrete, using frost-resistant aggregate, and the use of proper mix proportioning and curing increase the resistance of concrete to frost damage.[1] These are summarized below. These and other measures to protect concrete against frost damage are described in more detail in ACI 201.2R.[15]

(a) Air Entrainment. The frost resistance of concrete can be substantially improved by providing an adequate air-void system within the concrete to reduce its permeability (Figure 2.12). The addition of small amounts of air-entraining agents to the fresh concrete mixture (e.g., 0.05 percent by weight of cement), small bubbles ranging from 0.05 to 1.0 mm (0.002 to 0.004 in.) in diameter are created for protection of concrete against frost damage.[1]

Depending on the aggregate size and exposure conditions, the dosage can be varied to produce the desired air content. The recommended air content varies with aggregate size because concrete mixes that contain large aggregates require less cement paste than rich concretes with smaller aggregates. Therefore, the latter would need more air entrainment to provide the same protection against frost damage.[19] The recommended air contents for frost-resistant concrete, according to ACI 318-92,[20] are shown in Table 2.1. The ACI 318 permits a one percent decrease in total air content for concretes having a specified 28 day compressive strength in excess of 34 MPa (5000 psi).

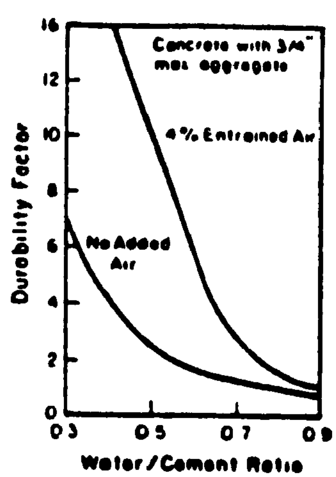


FIGURE 2.12 - EFFECT OF AIR-ENTRAINMENT ON CONCRETE DURABILITY[1]

TABLE 2.1 - RECOMMENDED AIR CONTENT FOR FROST-RESISTANT CONCRETE[20]

Nominal maximum aggregate size*		Air content, percent	
		Severe exposure	Moderate exposure
9 mm	⅜ in.	7-½	6
12 mm	½ in.	7	5-½
19 mm	¾ in.	6	5
25 mm	1 in.	6	4-½
38 mm	1-½ in.	5-½	4-½
50 mm	2 in.**	5	4
75 mm	3 in.**	4-½	3-½

*See ASTM C 33 for tolerances on oversize for various nominal maximum size designations.
**These air contents apply to the total mix, as for the preceding aggregate sizes. When testing these concretes, however, aggregate larger than 38 mm (1-½ in.) is removed by handpicking or sieving and air content is determined on the minus 38 mm (1-½ in.) fraction of mix, (Tolerance on air content as delivered applies to this value.) The air content of the total mix is computed from the value determined on the minus 38 mm (1-½ in.) fraction

During the placement of concrete, the air content of the concrete should be measured frequently. According to ACI Committee 201,[15] the following test methods may be used: volumetric method (ASTM C 173), pressure method (ASTM C 231), or the unit weight test (ASTM C 138). An air meter may also be used to estimate the air content. For lightweight concrete, the volumetric method is recommended. The air content in hardened concrete can be determined using microscope techniques in the laboratory (ASTM C 457).

(b) Low Water-Cement Ratio. Verbeck and Klieger[21] confirmed the hypothesis that at a given freezing temperature the amount of available water which can be frozen will be more with higher water-cement ratios (Figure 2.13). When the water-cement ratio is decreased and the cement content is increased, the frost resistance of the concrete will increase.

Accordingly, ACI Committees 201.2R[15] and ACI 318-92[20] have set guidelines for producing frost-resistant concrete for a variety of conditions: for frost-resistant normal weight concrete, the water-cement ratio should not exceed 0.45 for thin sections (bridge decks, railings, curbs, sills, ledges, and ornamental works) and sections with less than 25 mm (1 in.) of concrete cover over the reinforcement, and any concrete exposed to deicing salts; and 0.50 for all other structures. Also, for lightweight concrete, a minimum 28 day compressive strength of 28 MPa (4000 psi) is recommended.

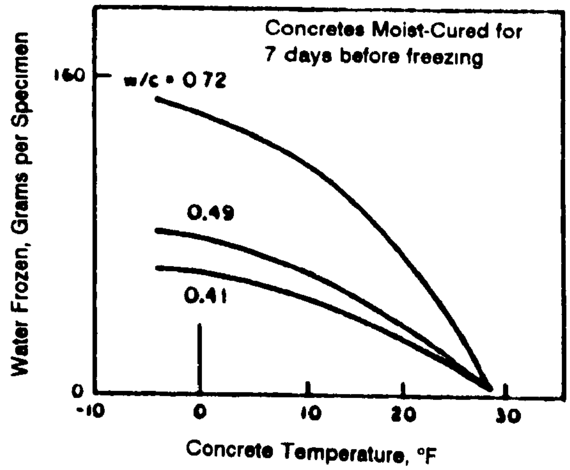


FIGURE 2.13 - EFFECT OF WATER-CEMENT RATIO ON AMOUNT OF FREEZABLE WATER IN CONCRETE[1]

(c) Frost-Resistant Aggregates. Sound coarse aggregates will produce frost-resistant concrete. Aggregates which are crushed to a nominal size of 12 to 19 mm (½ to ¾ in.) will usually produce satisfactory results, since the crushing process tends to break the aggregate along its weaker planes. Aggregates which are highly porous, such as some cherts, sandstones, limestones, and shales are more susceptible to frost damage than aggregates like granite, basalt, quartzite, or marble.[19]

Natural aggregates should meet ASTM C 33 requirements and lightweight aggregates should meet the requirements of ASTM C 330. The best way to evaluate aggregate performance is by field experience, but if this is not feasible, the engineer must rely on laboratory testing, such as petrographic examination, rapid freezing and thawing tests (ASTM C 666), and dilation tests (ASTM C 671).

(d) Adequate Curing. Proper consolidation and curing are also important factors influencing the frost resistance of concrete. The ACI Committee 201.2R[15] report recommends that air-entrained concrete should resist the effects of freezing and thawing (one or two cycles) as soon as a compressive strength of 3.45 MPa (500 psi) is attained. Concrete should have a compressive strength of about 28 MPa (4000 psi) before it is exposed to freezing temperatures. For moderate

exposure conditions, a specified compressive strength of 21 MPa (3000 psi) should be attained. For water-cement ratios of 0.50 or less, at least seven days of moist curing at normal temperature is recommended before exposing the concrete to freezing conditions.[1]

2.2.3 CONCRETE SCALING

The combined effects of frost action and the presence of deicing salts on concrete produce a more severe attack than frost alone. Applying deicing agents, such as ammonium chloride, calcium chloride, and sodium chloride to a concrete surface covered with ice will cause the concrete surface to experience temperature shock when the ice melts. The temperature gradient which is created between the surface and the interior of the concrete will generate internal stresses that may cause the outer layer of the concrete to crack.[6]

As previously described in Section 2.2.2.1, the change in the freezing behavior of the pore water is due to the ingress of salts (deicing agents) from the outside of the concrete. The content of deicing solution will decrease with increasing distance from the surface of the concrete, creating a freezing temperature gradient within the concrete.[22] As a result of both the change in temperature and salt concentration gradients, some layers of concrete will freeze at different times, causing scaling (Figure 2.14). Researchers have noted that the most damage to the concrete surface by scaling occurs when salt concentrations reach about four to five percent.[1] The use of chlorides as deicing agents also increases the risk of reinforcement corrosion (Section 2.3).

2.2.4 DETERIORATION BY SURFACE WEAR

The resistance of concrete to surface wear is defined as the "ability of a surface to resist being worn away by rubbing or friction".[24] Concrete does not have a high resistance to repeated abrasion cycles, especially if the concrete is very porous or has a low strength, and contains an aggregate of low wear resistance.[1] Surface wear can occur due to abrasion, erosion and cavitation. From a review of References 1, 6, 15, and 25, the three phenomena are summarized below.

2.2.4.1 ABRASION

The term 'abrasion' is usually used to describe wear on pavements and industrial floors by vehicular traffic.[15] Although this is not a significant problem in the marine environment, such wear may occur on surfaces of concrete piers or wharves. According to Prior,[26] abrasion of pavements can be classified into two types:

- "Wear on concrete floors due to foot traffic and light trucking, skidding, scraping or sliding of objects on the surface (attrition)", and
- "Wear on concrete road surfaces due to heavy trucks and automobiles with studded tires or chains (attrition, scraping and percussion)".

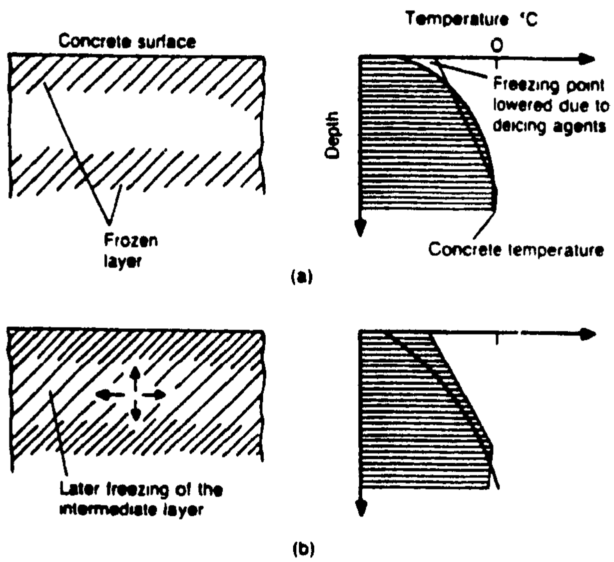


FIGURE 2.14 - SCALING DUE TO VARIATIONS IN THE TIMING OF FREEZING OF LAYERS:
(a) INTERMEDIATE LAYER IS INITIALLY UNFROZEN;
(b) INTERMEDIATE LAYER FREEZES LATER, CAUSING SCALING[6]

Tire chains and studded snow tires can also cause significant wear damage to good quality concrete surfaces. Tire chains cause wear by "flailing and scuffing action" as the metal studs contact the concrete surface. The damage caused by studded snow tires is due to dynamic impact of the small tungsten carbide tip of the studs. A study conducted by Smith and Schonfeld[27] in Ontario, Canada, determined that ruts from 6 to 12 mm (¼ to ½ in.) deep may form in a single season where traffic is heavy. In general, wear due to abrasion does not affect the concrete structurally, but in some cases it may cause serviceability or dusting problems.

2.2.4.2 RECOMMENDATIONS FOR CONTROLLING ABRASION

The abrasion resistance of concrete is dependent on compressive strength, aggregate properties, use of toppings, and finishing and curing methods. Test and field experience have shown that compressive strength is the most important factor influencing the abrasion resistance of concrete. Accordingly, ACI Committee 201[15] recommends that the compressive strength of concrete should be more than 30 MPa (4000 psi). This can be achieved by using a low water-cement ratio, proper grading of fine and coarse aggregate (limit the maximum nominal size to 25 mm), and minimum air content as dictated by exposure conditions. Using hard, tough, coarse aggregates will provide additional abrasion resistance.

2.2.4.3 EROSION

Erosion damage occurs as a result of abrasion caused by silt, sand, gravel and rocks which are carried by flowing water over a concrete surface (attrition and scraping).[29] Erosion is recognized by the smooth, worn appearance of the concrete surface. This damage is common to hydraulic structures at bridge piers, and structures protecting embankments or coasts. Due to the presence of high water velocities, spillway aprons, stilling basins, sluiceways, and outlet tunnel linings are especially susceptible to erosion damage.

The rate at which erosion occurs depends on several factors including the porosity or strength of concrete, and on the amount, size, shape, density, hardness, and velocity of the particles being transported. Depending on flow conditions, erosion damage can range between a few centimeters to several meters. The relationship between fluid-bottom velocity and the size of particles which a specific velocity can transport is shown in Figure 2.15. If the quantity and size of the particles are small, erosion will not be significant at water velocities of up to 2 m/s (6 ft/s).

2.2.4.4 RECOMMENDATIONS FOR CONTROLLING EROSION

Numerous materials and techniques have been utilized for constructing and repairing structures damaged by severe erosion. Investigations by Liu[30] have shown that "abrasion-resistant concrete should include the maximum amount of the hardest available coarse aggregate and the lowest practical water-cement ratio" (Figure 2.16). For instance, concrete containing chert aggregate will provide approximately twice the abrasion-erosion resistance of concrete containing limestone.

When a structure will be exposed to severe erosion or abrasion conditions, ACI 201.2R[15] recommends that, in addition to using hard aggregates, the concrete should have a minimum 28 day compressive strength of 42 MPa (6000 psi) and moist-cured for at least seven days before exposing the concrete to the aggressive environment. If hard aggregate is not available, the use of silica fume and high-range water-reducing admixtures will produce a very strong concrete.[29]

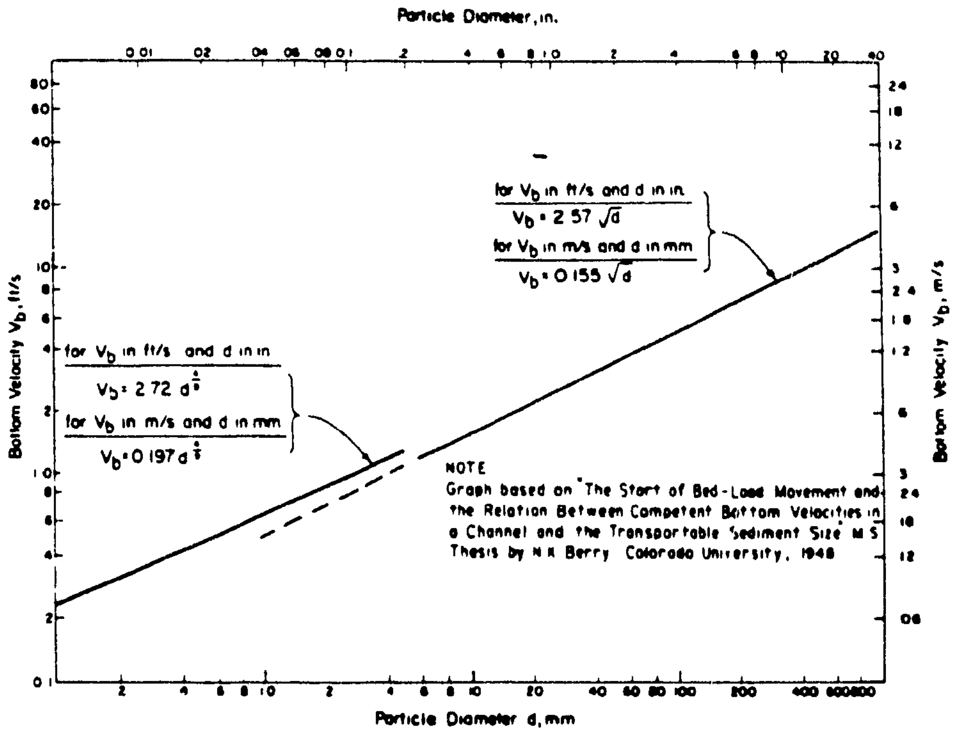


FIGURE 2.15 - BOTTOM VELOCITY VS. TRANSPORTED PARTICLE SIZE[29]

Vacuum-treated concrete, polymer concrete, polymer-impregnated concrete, and polymer-portland cement concrete will provide a higher resistance to abrasion-erosion damage than conventional concrete. According to ACI Committee 223,[31] concrete produced with shrinkage-compensating cement, will provide an abrasion resistance from 30 to 40 percent higher than portland cement concrete with similar mixture proportions. The various materials and techniques used for repairing erosion damage to hydraulic concrete structures are discussed in Chapter 7.

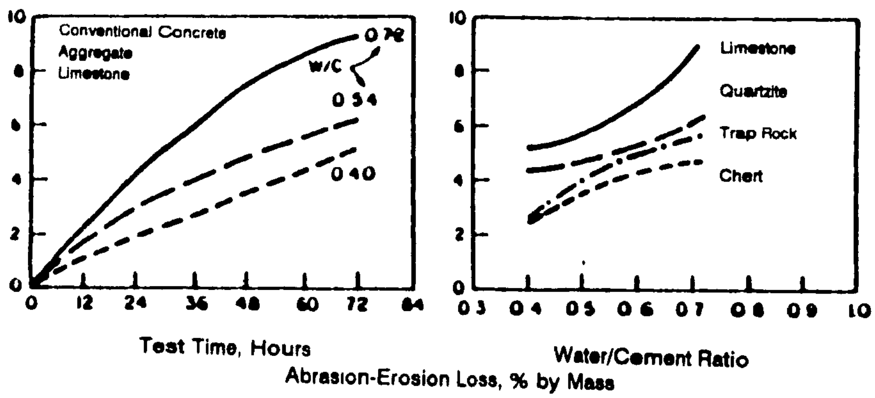


FIGURE 2.16 - INFLUENCE OF WATER-CEMENT RATIO AND AGGREGATE TYPE ON ABRASION-EROSION DAMAGE IN CONCRETE[1]

2.2.4.5 CAVITATION

Erosion damage to concrete hydraulic structures can also be caused by cavitation, resulting from the sudden collapse of vapor bubbles in water that is flowing at velocities in excess of 12 m/s (40 ft/s), or 7.6 m/s (25 ft/s) in closed conduits.[29] In flowing water, vapor bubbles form when the local absolute pressure in water drops to the ambient vapor pressure of water corresponding to the ambient temperature. Figure 2.17 shows examples of concrete surface irregularities which can cause vapor bubbles to form.

Cavitation damage is produced when the vapor bubbles collapse or implode. The collapses that occur near the concrete surface produce very high instantaneous pressures that impact on the concrete surfaces. Repeated impact of these high-energy pressures will cause severe local pitting. The damage caused by cavitation is different from that caused by erosion because cavitation pits cut around the coarse aggregate particles. This continuous action eventually undermines the aggregates causing them to come loose.

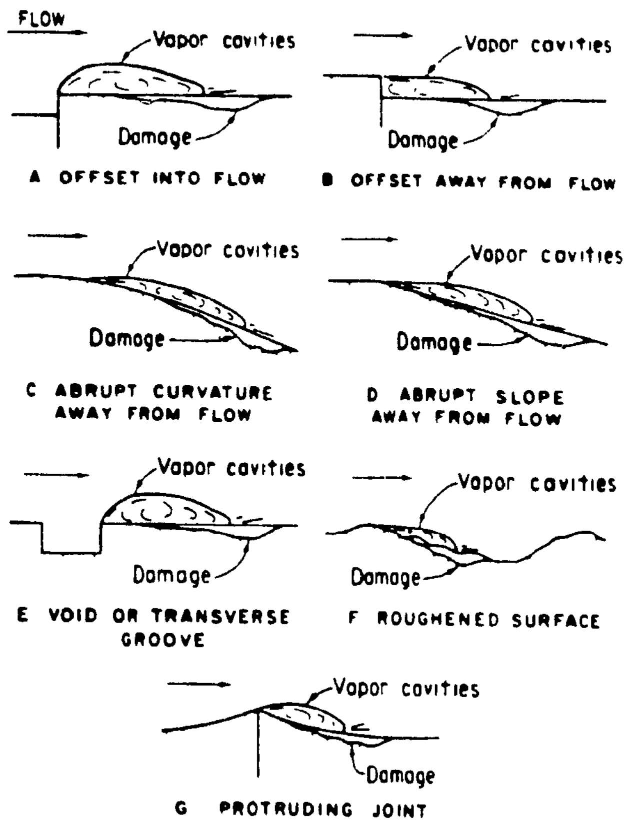


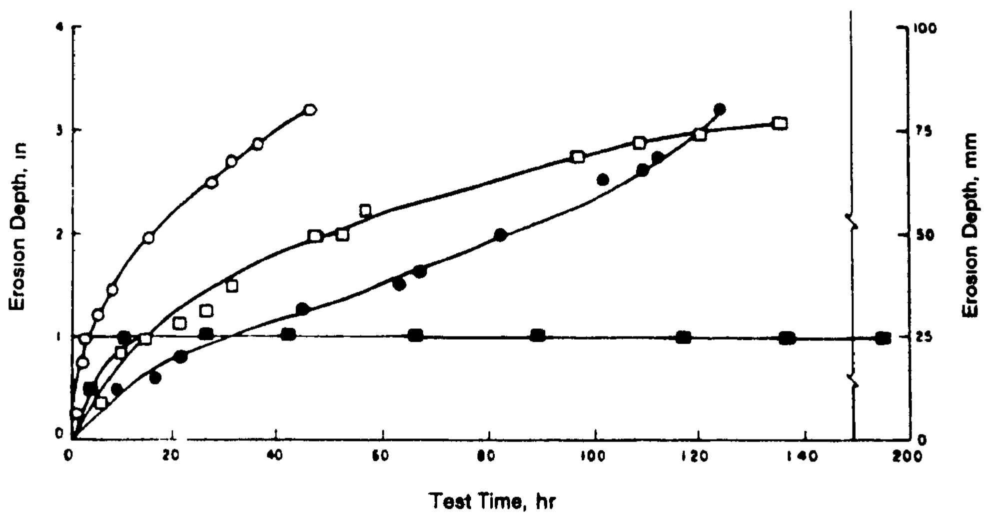
FIGURE 2.17 - CAVITATION OCCURRENCE AT SURFACE IRREGULARITIES[29]

2.2.4.6 RECOMMENDATIONS FOR CONTROLLING CAVITATION

According to ACI Committee 210R[29], the cavitation resistance of concrete can be increased by using high-strength concrete with a low water-cement ratio, provided it is not subjected to abrasion-erosion damage. The use of hard, dense aggregate with a nominal maximum size of 38 mm (1-½ in.) is recommended for producing a good bond between the aggregate and the cement paste. This is essential for achieving increased resistance to cavitation damage.

Cavitation damage has been successfully repaired using steel fiber- reinforced concrete.[32] This material provides good impact resistance to cavitation damage and appears to reduce cracking and disintegration of the concrete. The use of polymers has also shown to improve the cavitation resistance of both conventional and fiber-reinforced concrete.[33,34] Various materials and coating systems have been tested at the U.S. Army Detroit Dam (Oregon) High Head Erosion test flume.[35] Figure 2.18 shows the performance of several of these materials subjected to water flows with velocities of 37 m/s (120 ft./s).

○ TEST SLAB NO. 1 - CONVENTIONAL CONCRETE - Cement 600 lb/yd³ (356 kg/m³), MSA 1½" (38 mm)
□ TEST SLAB NO 2 - STEEL FIBER CONCRETE - Cement 690 lb/yd³ (409 kg/m³), MSA 3/4" (19 mm)
● TEST SLAB NO 3 - POLYMERIZED CONVENTIONAL - Cement 600 lb/yd³ (356 kg/m³), MSA 1½ (38 mm)
■ TEST SLAB NO 4 - POLYMERIZED STEEL FIBER - Cement 690 lb/yd³ (409 kg/m³), MSA 3/4" (19 mm)



MSA - Maximum size aggregate

FIGURE 2.18 - EROSION DEPTH VS. TIME FOR VARIOUS TYPES OF CONCRETE MIXTURES[29]

Although using the proper materials will increase the cavitation resistance of concrete, the best defence is to minimize or eliminate the factors causing cavitation, such as misalignments or abrupt changes of slope. In some cases, this may be unavoidable and the designer can minimize the effect of cavitation by the use of aeration devices designed to supply air to the flowing water.[29] Research has shown that irregularities on the concrete surface will not cause cavitation damage if the air/water ratio in the water adjacent to the concrete surface is about eight percent. Such devices are shown in Figures 2.19 and 2.20.

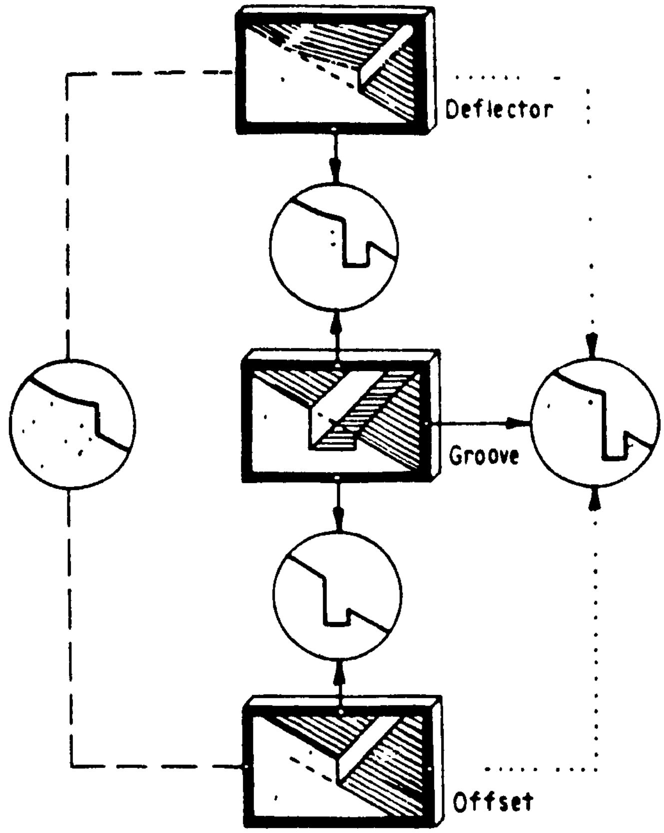


FIGURE 2.19 - TYPES OF AERATORS[29]

2.2.4.7 MATERIAL EVALUATION

Another approach to controlling surface wear of concrete is by testing and evaluating the materials prior to using them in hydraulic structures. A variety of standardized test methods are available for determining abrasion-erosion resistance of concrete surfaces in terms of weight loss after a specified time.[1] ASTM C 779 describes three methods for testing the relative abrasion resistance of horizontal concrete surfaces in terms of weight loss after a specified time. These tests include an abrasive type apparatus, such as steel dressing wheels and rolling steel balls under pressure. ASTM C 418 describes the sandblast test, which determines the abrasion resistance of concrete by subjecting it to the abrasive action of air-blown silica sand. The modified Los Angeles rattler tests (ASTM C 131 and C 535) have also been used to determine abrasion-erosion resistance of concrete surfaces.

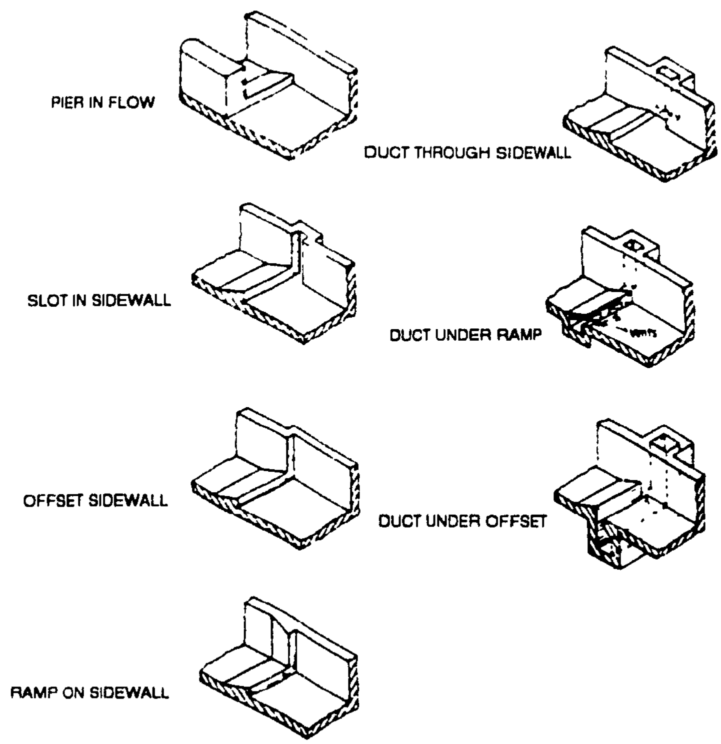


FIGURE 2.20 - AIR SUPPLY TO AERATORS[29]

These tests are designed to simulate erosion caused by heavy foot or wheeled traffic on concrete surfaces and are not appropriate for modelling erosion in hydraulic structures. Accordingly, the U.S. Army Corps of Engineers has developed a test method for simulating and measuring abrasion-erosion in hydraulic structures. This test (CRD-C 63-80), "Test Method for Abrasion-Erosion Resistance of Concrete (Underwater Method)," subjects concrete specimens to abrasion-erosion under the action of steel grinding balls and circulating water with an approximate velocity of 2 m/s (6 ft./s). The damage is measured by determining the amount of lost material as a percentage of the original mass. The development of the test procedure and data from a variety of tests of various concrete mixtures have been reported by Liu.[30]

2.3 REINFORCEMENT CORROSION IN CONCRETE

A recent review by Mehta[36] of proceedings of the cement Chemistry Congresses and other symposia held by ACI, ASTM, and RILEM in the last 50 years observed that "corrosion of reinforcing steel is considered to be the most serious problem responsible for lack of durability" in modern (post 1960) structures. He states that bridge-deck and parking garage deterioration due to reinforcement corrosion has become a major concern in the United States and Canada. A recent report by Gerwick[37] discloses that many worldwide concrete-lined tunnels are leaking as a result of corrosion of reinforcing steel. According to Mehta[1], a survey of collapsed buildings during the 1974-78 period in England showed that reinforcement corrosion of prestressing steel was the cause of failure in at least eight structures (12-40 years old). In marine structures, the most significant damage from corrosion of steel occurs within the splash zone, where the structure is exposed to alternating cycles of wetting and drying[38].

The damage caused by reinforcement corrosion consists of expansion, cracking and eventual spalling of concrete cover. Corrosion is often readily identified by rust stains which are bled through cracks on the concrete surface. Unfortunately, such signs usually indicate that the damage is already well advanced. In some cases, reinforcement corrosion can result in loss of bond between concrete and the steel, resulting in structural failure.[1,36]

2.3.1 MECHANISM OF CORROSION IN STEEL

Reinforcement corrosion of steel in concrete is an electrochemical process which involves the transformation of metallic iron (Fe) to rust [$Fe(OH)_3$]. The phenomenon can be represented by an anode process and a cathode process[3] as shown by the reaction below. This is illustrated also schematically in (Figure 2.21):

$$\underset{(metallic\ Iron)}{2Fe} \rightarrow 4e^- + 2Fe^{2+} \qquad (anode)$$

$$O_2 + 2H_2O + 4e^- \rightarrow 4(OH)^- \qquad (cathode)$$

$$2Fe^{2+} + 4(OH)^- + \frac{1}{2}O_2 + H_2O \rightarrow 2Fe(OH)_3 \qquad (rust)$$

When metallic iron is converted to rust, it produces an increase in volume which may be as much as six to seven times that of the original steel, causing expansion and cracking of the concrete.

Embedded steel in concrete is usually protected from corrosion by a passive film of iron oxide formed on the steel surface due to the high alkalinity (pH of 13.5) of the pore solution in the hydrated cement paste. This passive film must be disrupted before the anodic reaction can begin. Similarly, for the cathodic reaction to occur, oxygen and water must be continuously available. Mehta[38] and Hertlein[39] state that for the transformation of iron to rust to occur, all of the following essential conditions must be satisfied:

- For the anode process to occur, metallic (Fe) iron must be available at the surface of the reinforcing steel,
- There must be voltage potential differences along the steel surface or the surrounding concrete,
- Oxygen and moisture must be available for the concrete to have electrical contact with the steel, and
- The electrical resistivity of concrete must be low enough to allow electrons to flow in the steel from anodic to cathodic areas.

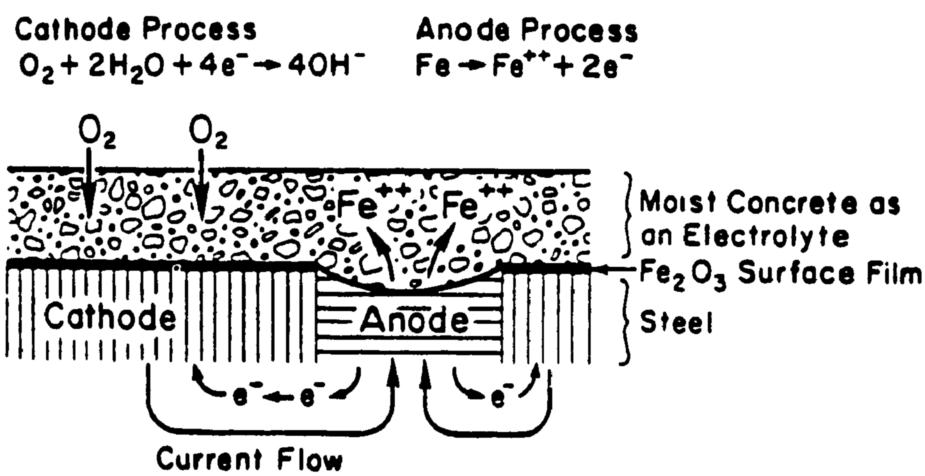


FIGURE 2.21 - TYPICAL CORROSION OF STEEL REINFORCEMENT IN CONCRETE[7]

To explain the deterioration of reinforced concrete structures in the marine environment, Mehta and Gerwick[3] developed a cracking-corrosion interaction model (Figure 2.22), according to which an increase in the permeability of concrete caused by enlargement and interconnection of microcracks is necessary for reinforcement corrosion to occur. As the rate of corrosion increases, the formation of rust will increase microcracking further, thus increasing the risk of reinforcement corrosion. Ultimately, this process leads to severe deterioration of both the steel and the concrete.

2.3.2 DEPASSIVATION

Depassivation of steel in concrete is caused by the removal of calcium hydroxide around the rebar, and the breakdown of the iron oxide film present on the surface of the rebar.[36] Once the passive film is destroyed, the corrosion activity will depend on the electrical resistivity of the concrete and the amount of oxygen available at the cathode.[3] The following sections provide a brief summary of some important factors contributing to steel depassivation. The principles of depassivation are also valid for prestressing steel.[6]

2.3.2.1 CARBONATION OF CONCRETE

Carbon dioxide (CO_2) present in the air or in some waters penetrates the concrete and reacts with the pore fluid to form carbonic acid (Section 2.4.2.1). This reacts with the alkaline calcium hydroxide [$Ca(OH)_2$] in the hydrated cement paste to form calcium carbonate ($CaCO_3$), which reduces the pH of the concrete to around 9.4.[39] This may be represented by the following reaction:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

Research has shown that reducing the pH of the pore solution in the cement paste to below 11.5 destroys the passive film on the steel, and initiates the corrosion process.[1,3] The rate of carbonation (increase of carbonation depth with time) depends on the rate of CO_2 penetration into concrete, and appears to follow a square-root time law (Figure 2.23).[6] Penetration of CO_2 can only occur in air-filled pores. Concrete which is completely saturated with water will not carbonate, unless it is subjected to repeated wetting and drying cycles. This is why in marine structures deterioration of concrete from corrosion of steel is more of a problem in the splash zone.

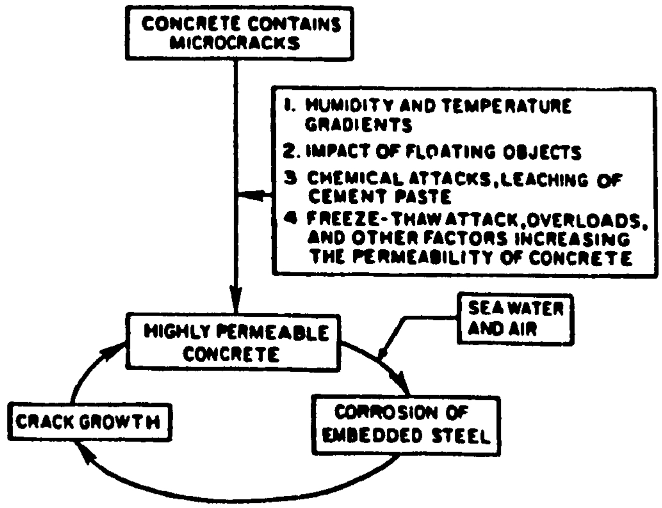


FIGURE 2.22 - DIAGRAMMATIC REPRESENTATION OF THE CRACKING-CORROSION INTERACTION IN REINFORCED CONCRETE[38]

2.3.2.2 PENETRATION OF CHLORIDES INTO CONCRETE

The presence of free chloride (Cl) ions in concrete destroys the protective iron oxide film on steel.[3] When free chloride ions are present, the electrical conductivity of the concrete is increased, and by chemical reaction, depassivate the steel.[39] The ions may be introduced in the concrete in several ways: as a secondary effect of carbonation (breakdown of chloro-aluminates, releasing Cl^- ions), as an accelerating admixture, chloride-contaminated aggregates, or from deicing salts and seawater spray.

Although chloride ions are an essential catalyst of the corrosion process, the mechanism through which the protective film is destroyed is not fully understood. Three theories have been postulated to explain the electrochemical effects of chloride ions on steel corrosion:[7]

(a) *Oxide Film Theory.* Chloride ions penetrate the protective film through pores or defects in the film. Also, the chloride ions may "colloidally disperse" the film, thereby facilitating penetration of ions.

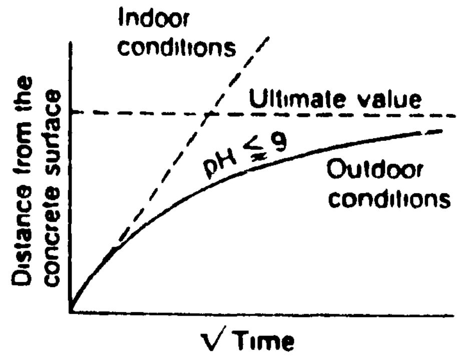


FIGURE 2.23 - RELATIONSHIP BETWEEN DEPTH OF CONCRETE CARBONATION WITH TIME[6]

(b) *Adsorption Theory.* As chloride ions are adsorbed on the surface of the reinforcing steel, metal ions are freed more easily. As a result, the chloride ions react more aggressively with the dissolved oxygen or hydroxyl ions.

(c) *Transitory Complex Theory* "Chloride ions compete with hydroxyl ions for the ferrous ions produced by the corrosion process" to form iron chloride which diffuses away from the anode. The protective ferric oxide layer is destroyed when iron chloride breaks down to form iron hydroxide and releases the chloride ion which removes more ferrous ions from the anode.

2.3.3 THRESHOLD CHLORIDE PENETRATION

For the corrosion process to occur at any appreciable rate, it is clear that a specific chloride ion concentration must be present. This limit is often termed the threshold chloride ion concentration. Exceeding this limit will increase the rate of corrosion.[7] The concept of threshold chloride concentration is shown schematically in Figure 2.24. From the graph it is clear that an increase in chloride concentration will not have an adverse effect on the concrete as long as the pH value is also increased.

Empirical data shows that, when the chloride to hydroxyl ion molar ratio is higher than 0.6, steel is no longer protected against corrosion even at pH values greater than 11.5.[1] The relationship between chloride concentration and pH at the iron-liquid interface is shown in Figure 2.25. A chloride ion concentration of 0.2 percent by weight of cement is normally considered as the threshold limit.[40] For concrete mix proportions typically employed in practice, the threshold chloride content required to start the corrosion process varies between 0.6 to 0.9 kg of Cl per cubic meter of concrete (0.2 to 0.3 lb/cy).[1] However, an exact value has not been firmly established.

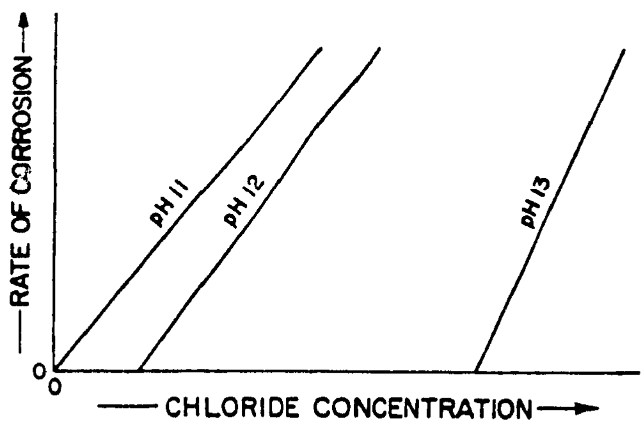


FIGURE 2.24 - RATE OF CORROSION VS. CHLORIDE CONCENTRATION[7]

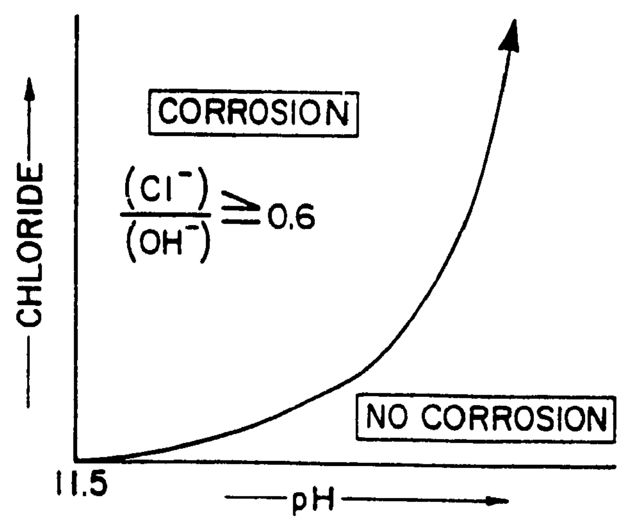


FIGURE 2.25 - CHLORIDE CONTENT VS. pH[7]

2.3.4 PREDICTING CORROSION DAMAGE

There are many techniques available to help predict the start of corrosion, assess corrosion damage, and identify the cause. However, if they are to be used effectively, it is important to understand the various processes that can initiate corrosion, and how this corrosion may affect the structure.

To integrate the various factors influencing reinforcing steel corrosion in concrete structures, Tuutti[41] proposed a model (Figure 2.26) which suggests that corrosion damage can be predicted by assuming two separate rate determining periods: the corrosion initiation period and the corrosion propagation period. Each stage is affected by different parameters. The initiation period is influenced by the rate of CO_2 and chloride ion diffusion, while the propagation period is influenced by the rate of oxygen and water diffusion These parameters control the depassivation of steel and the cathodic reaction, respectively. According to Lin and Jou[42] the second stage is more difficult to predict because it not only depends on the rate of oxygen diffusion through the concrete cover, but also the degree of corrosion the structure can endure. The propagation or deterioration period is influenced by other factors, such as moisture content of the concrete, its quality, strength, and mechanical requirements. A higher temperature and a more rapid loss of moisture of the concrete can produce short propagation periods (from six months to five years).[43] In this case, the effective service life of the structure can be considered as the initiation period and also the design life of the structure.[42]

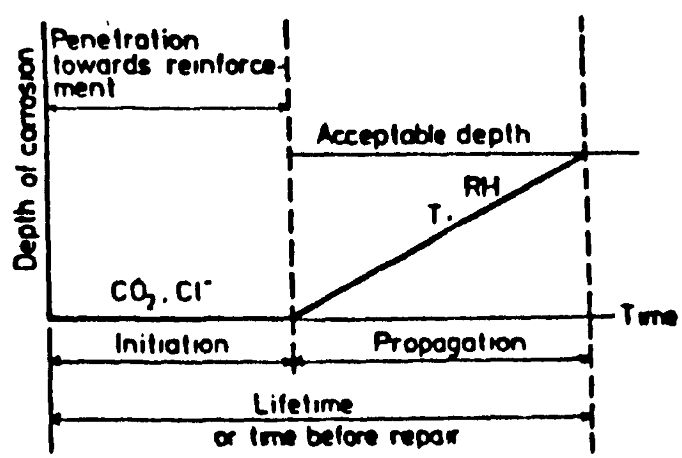


FIGURE 2.26 - SCHEMATIC REPRESENTATION OF STEEL CORROSION IN CONCRETE[30]

2.3.5 TYPES OF REINFORCEMENT CORROSION

Corrosion of metals can occur in many ways. The type and rate of corrosion depends on the materials present in the concrete, and on the moisture and gases available because they will affect its permeability, density, and reactivity.[39] Corrosion may either manifest itself in the form of general rusting, or localized attack known as pitting corrosion. Other forms of corrosion include galvanic corrosion, stress corrosion, hydrogen embrittlement, and bacterial corrosion. These are discussed below.

2.3.5.1 GENERAL RUSTING

General rusting is the most common form of steel corrosion. As previously described, this occurs through a complex electrochemical process in which the metal is oxidized when exposed to air and water Small electrical currents that flow between areas of different voltage potential on the steel transport metal ions from anodic to cathodic areas, thereby reducing the steel cross-section at the anode, and depositing metal at the cathode. As a result, rust occurs uniformly over the entire steel surface. Since rust typically occupies approximately six to seven times the volume of steel, this increase in volume creates stresses that cause cracking and spalling.[39]

2.3.5.2 PITTING CORROSION

Pitting corrosion is a localized form of attack which initiates when the protective film breaks down over small surface areas. This often occurs when there is a large concentration of chloride ions in the concrete in small depassivated areas. This decreases the electrical resistivity of the concrete locally and increases the rate of dissolution of iron at the small anodic pit (Figure 2.27).[39] This often results in a substantial local reduction in steel cross-sectional area at the pit without any external sign of damage to the concrete, because the corrosion products are soluble and are absorbed within the concrete.[44] This form of rusting often leads to sudden failure of prestressed or post-tensioned structures.

Anodic and cathodic areas can create concentration cells which may either be microscopically separated (microcell corrosion) or locally separated (macrocell corrosion).[6,39] These concentration cells are shown schematically in Figure 2.28. When the amounts of chloride ion concentration in concrete varies, electrical potential differences along the steel surface are created which permit corrosion to initiate. Corrosion may also occur as a result of different amounts of oxygen that are

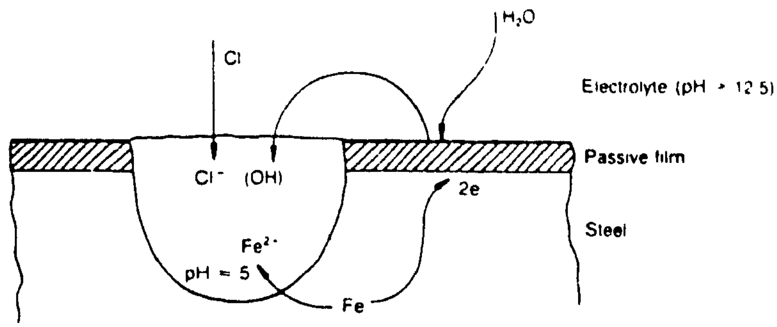


FIGURE 2.27 - PITTING CORROSION CAUSED BY CHLORIDES[8]

available to various areas of the reinforcement, creating what is known as a differential-oxygen cell.[7] For instance, Funahashi et al.[38] suggest that in marine structures contaminated with chlorides, rebar in concrete located under the seawater is anodic to the rebar in concrete exposed to the atmosphere. Research by Okada et al.[3] showed that the ratio of the cathodic area (Ac) to the anodic area (Aa) affects the rate of corrosion in macrocells of reinforcing steel in concrete. Their observations also showed that wetting and drying cycles, as opposed to continuous immersion, increases the corrosion rate by increasing the Ac/Aa ratio (i.e., oxygen supply to the cathode).

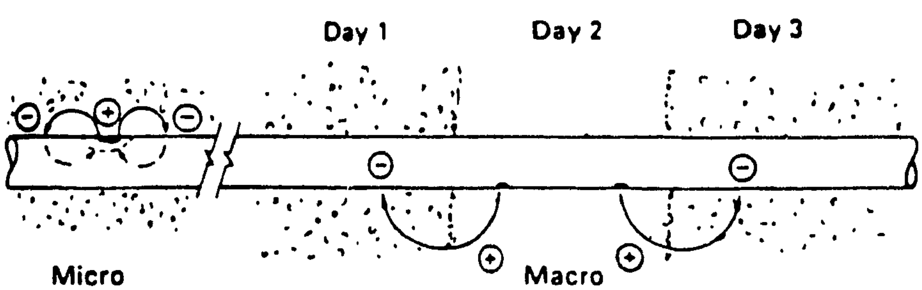


FIGURE 2.28 - CONCENTRATION CELLS IN CONCRETE[7]

2.3.5.3 GALVANIC CORROSION

Galvanic cells are created when the steel is in direct or indirect contact with another type of metal which occupies a different position in the "Galvanic Series"[39] The electrical potential that is developed, and the relative sizes of the two dissimilar metals, will determine the rate and extent of steel corrosion.[7] The further apart the metals are in the galvanic series, the more aggressive is the reaction.[39]

2.3.5.4 STRESS CORROSION CRACKING

This form of corrosion may lead to brittle failure of reinforcing or prestressing steel. Localized anodic processes produce high permanent stresses that can lead to cracking. The anodic process occurs at the root of the crack during the crack propagation stage as shown in Figure 2.29.[6]

2.3.5.5 HYDROGEN EMBRITTLEMENT

Another type of brittle failure which can occur in steel is the result of a cathodic process known as hydrogen embrittlement. Under certain conditions during the cathodic process, atomic hydrogen is produced as an intermediate product and can penetrate into the steel. The hydrogen recombines to form molecular hydrogen within the steel and produces a high internal pressure which usually leads to cracking (Figure 2.30).[6] Both types of failures can be prevented if the steel is encased by sound hardened concrete or cement grout.

2.3.5.6 BACTERIAL CORROSION

In anaerobic (oxygen-free) conditions, bacteria can form on the surface of the concrete and penetrate to the level of the steel. Although oxygen is not present, the bacteria produces iron sulfide which initiates the corrosion reaction. This reaction is often severe and can lead to significant structural damage.[44]

2.3.6 CORROSION PROTECTION MEASURES

Using a good quality concrete of low permeability is essential to control the various mechanisms involved in the corrosion process. Although no conventional concrete is completely impermeable, proper and careful attention to concrete mixture parameters, workmanship, and curing will ensure a good quality concrete with a low permeability. The various parameters are summarized in the following sections.

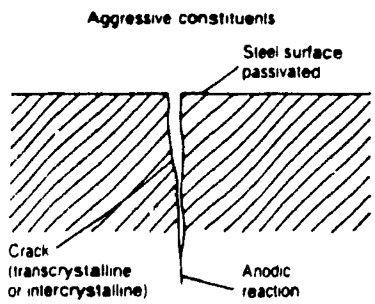


FIGURE 2.29 - STRESS CORROSION CRACKING[6]

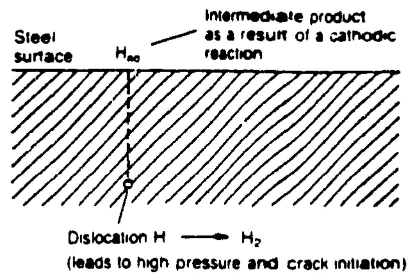


FIGURE 2.30 - HYDROGEN EMBRITTLEMENT[6]

2.3.6.1 WATER-CEMENT RATIO

Low water-cement ratios produce less permeable concrete which in turn provides greater protection against reinforcement corrosion. Figure 2.31 shows the influence of water-cement ratio and the degree of hydration on permeability. Accordingly, the ACI Building Code 318-92[20] specifies a maximum water-cement ratio of 0.40 and a concrete compressive strength of at least 35 MPa (5000 psi) for normal weight concrete exposed to deicing salts, brackish water, or seawater (Table 2.2). However, if the concrete cover is increased by 13 mm (½ in), the code allows a maximum water-cement ratio of 0.45. The ACI Committee 357R-84[45] report for the design and construction of offshore concrete structures recommends similar water-cement ratios for various exposure zones (Table 2.3). When severe deterioration of concrete is anticipated, a 28-day compressive strength of 42 MPa (6000 psi) is recommended.

The ACI Committee 201.2R[15] report recommends that for structures located above the seawater and seawater spray zone for a height of 8 m (25 ft.), or within a horizontal distance of 30 m (100 ft.), the water-cement ratio should be less than 0.50 by weight.

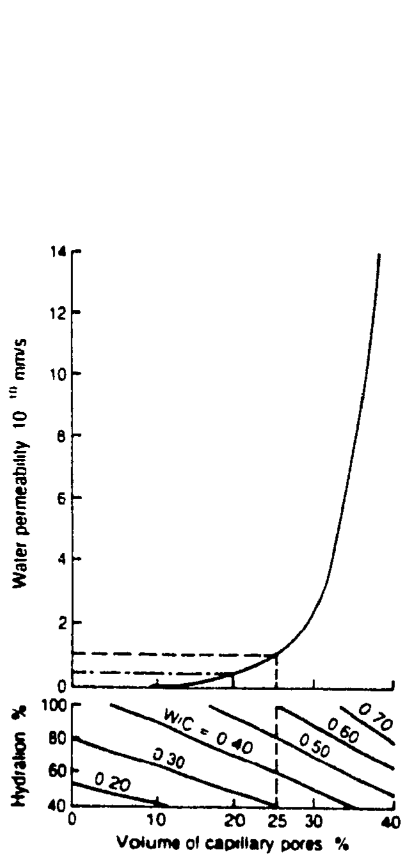


FIGURE 2.31 - INFLUENCE OF WATER-CEMENT RATIO ON PERMEABILITY[6]

TABLE 2.2 - RECOMMENDED WATER-CEMENT RATIOS FOR SPECIAL EXPOSURE CONDITIONS[20]

Exposure condition	Maximum water-cement ratio, normal weight aggregate concrete	Minimum f'_c, light-weight aggregate concrete
Concrete intended to have low permeability when exposed to water	0.50	25 MPa (3750 psi)
Concrete exposed to freezing and thawing in a moist condition	0.45	30 MPa (4250 psi)
For corrosion protection for reinforced concrete exposed to deicing salts, brackish water, seawater or spray from these sources	0.40*	32 MPa (4750 psi)*

*If minimum concrete cover is increased by 12 mm (0.5 in), water-cement ratio may be increased to 0.45 for normal weight concrete, or f'_c reduced to 30 MPa (4250 psi) for lightweight concrete.

TABLE 2.3 - RECOMMENDED WATER-CEMENT RATIOS AND CONCRETE COMPRESSIVE STRENGTHS FOR THREE EXPOSURE ZONES[45]

Zone	Maximum water-cement ratio	Minimum 28-day cylinder compressive strength
Submerged	0.45	35 MPa (5000 psi)
Splash	0.40	35 MPa (5000 psi)
Atmospheric	0.40	35 MPa (5000 psi)

2.3.6.2 CEMENT CONTENT

The capacity of concrete to bind CO_2 and Cl^- will increase as the cement content also increases (Figure 2.32). The rate of carbonation and chloride penetration in concrete are influenced much less by the cement content than by the water-cement ratio, quality of compaction, and curing. However, the cement content will influence the workability of concrete, and to a lesser degree, the curing sensitivity.[6]

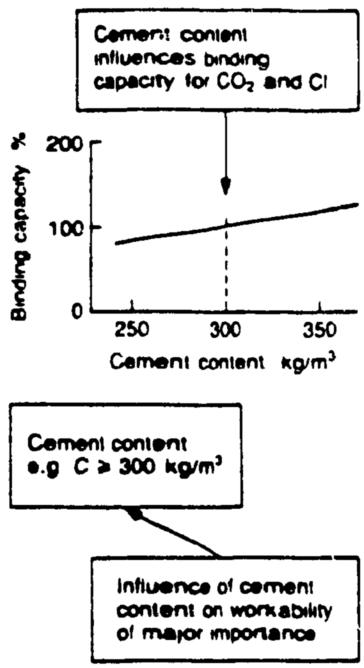


FIGURE 2.32 - INFLUENCE OF THE CEMENT CONTENT ON BINDING CAPACITY[6]

ACI 318-92[20] does not provide any cement content requirement for a marine environment. Normally, a cement content of about 300 kg/m^3 (500 lb/cy) is enough to produce a low permeability concrete with adequate durability, provided the water-cement ratio is below 0.5 to 0.6.[6] The ACI 357R-84[45] report recommends a minimum cement content of 355 kg/m^3 (600 lb/cy) of concrete. If more than 415 kg/m^3 (700 lb/cy) of portland cement is used, special steps must be taken to reduce the likelihood of cracking in thin members due to thermal stresses. Thermal cracking can be reduced by replacing part of the cement with a pozzolan.[46]

2.3.6.3 CEMENT TYPES

The durability of concrete is greatly affected by cement composition. The tricalcium aluminate (C_3A) content in portland cement concrete has a significant effect on the corrosion process. Increasing the C_3A content increases the resistance to corrosion, since the chloride ions react with the hydrated tricalcium sulfoaluminate in the hardened cement paste to form an insoluble Friedel salt. Recent research by Rasheeduzzafar[47] showed "that corrosion initiation time, time-to-cracking of cover concrete, and chloride threshold values increased, whereas loss of metal from reinforcement corrosion decreased as the C_3A content of cement increased". Figure 2.33 shows the effect of C_3A content of cement on time-to-initiation of corrosion of reinforcing steel.

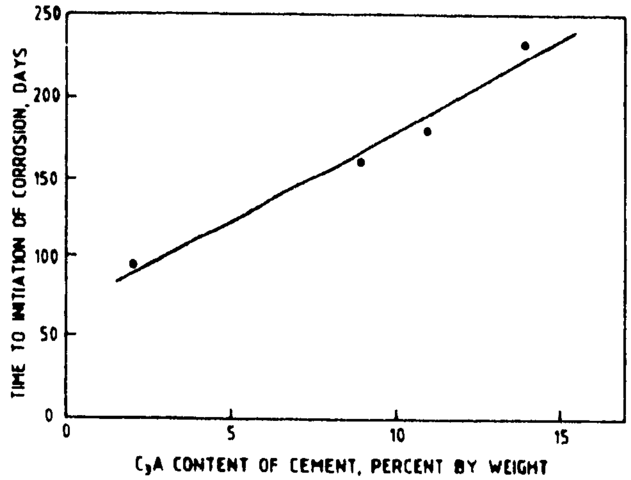


FIGURE 2.33 - EFFECT OF C_3A CONTENT ON TIME TO INITIATION OF REINFORCEMENT CORROSION[47]

Similarly, the loss of metal data for reinforcing bars taken from ASTM Type I (CSA Type 10) and ASTM Type V (CSA Type 50) cement concrete test specimens indicate that the corrosion performance of Type I (C_3A: 9.5%) cement is better than the performance of Type V (C_3A: 2.8%) cement (Figure 2 34).[47] Nevertheless, as the amount of chlorides increases, the benefit of adding more C_3A becomes less noticeable since C_3A in cement combines with only a limited quantity of chloride Furthermore, increasing the C_3A content reduces the resistance of concrete to sulfate attack.[7] In such situations, using Type V cement would provide adequate protection against sulfate attack but it would not remove free chlorides to protect the steel from corrosion. ACI 357R-84[45] permits the use of ASTM Type I, II, and III (CSA Types 10, 20 and 30) portland cements, but recommends that the C_3A content should be between 4 and 10 percent.

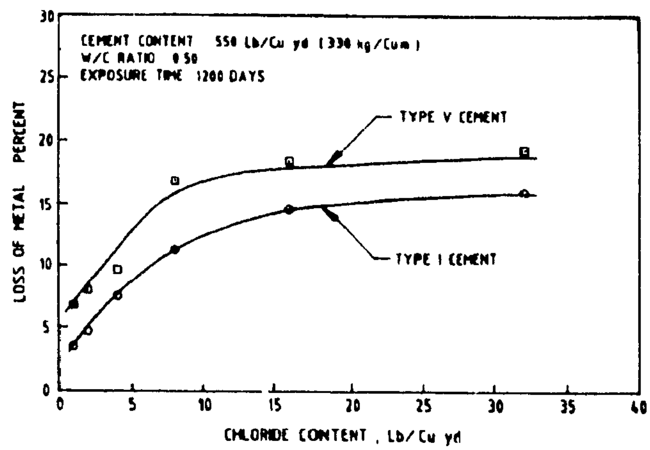


FIGURE 2.34 - EFFECT OF CEMENT TYPE AND CHLORIDE CONTENT ON REINFORCEMENT CORROSION[47]

2.3.6.4 POZZOLANS

In marine environments, the use of pozzolans, such as silica fume, fly ash, and blast-furnace slag, are commonly used to produce a concrete which will simultaneously resist sulfate attack and chloride-induced corrosion. Pozzolans combine with the calcium hydroxide and water in the fresh mix to form a hardened cement paste with a higher strength and a reduced permeability (Figure 2.35). Pozzolans also combine chemically with the lime and reduce the effects of lime leaching. Typical mix proportions include (by weight of cement): 15 to 20 percent fly ash, 50 to 70 percent of granulated blast-furnace slag, or 5 to 10 percent of condensed silica fume.[46]

Research by Rasheeduzzafar[47] on blended cements made by replacing a portion of ordinary Type I (high C_3A) portland cement with 10 percent silica fume, 20 percent fly ash, or 70 percent blast furnace slag, produced concrete with a higher resistance to corrosion and to sulfate attack. Results of this research are shown in Figure 2.36. ACI 318-92[20] requires a Type II cement or a Type I cement plus a pozzolan to resist moderate sulfate attack in seawater. It should be noted that when pozzolans or other cementitious admixtures are used in addition to portland cement, it is more useful to consider the water to cementitious materials ratio rather than simply the water-cement ratio.[48]

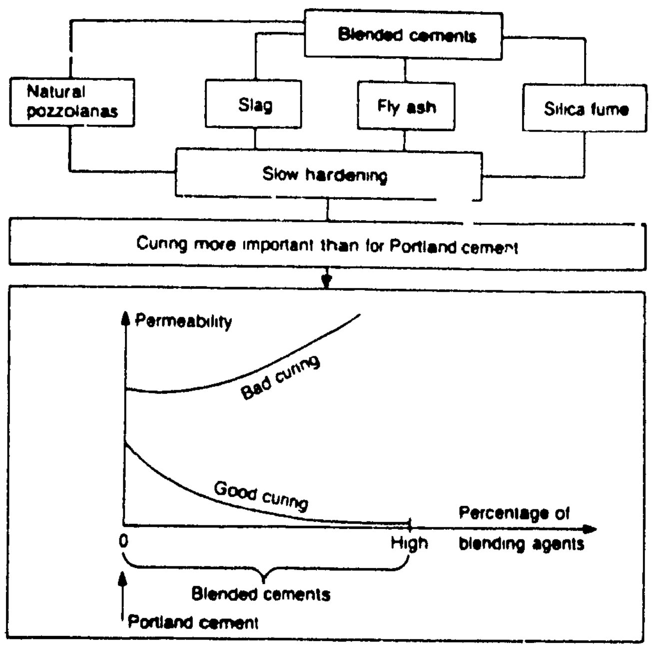


FIGURE 2.35 - INFLUENCE OF CEMENT TYPE ON PERMEABILITY[6]

2.3.6.5 ADMIXTURES

Numerous organic and inorganic chemical admixtures have been used to prevent or reduce steel corrosion in concrete. Water-reducing admixtures and superplasticizers are commonly used to provide workable mixes at low water-cement ratios. To protect reinforcing and prestressing steel from corrosion, calcium chloride ($CaCl_2$) or admixtures containing chlorides should not be used.[46] Chemical admixtures used in portland cement concrete must meet the requirements of ASTM C 494. -

2.3.6.6 AGGREGATES

Since 70 percent of the concrete mix volume is occupied by aggregates, their presence has a significant effect on concrete permeability. For instance, concrete permeability will increase with creasing maximum coarse aggregate size. This is because most mineral aggregates have a permeability 10 to 1000 times greater than that of the cement paste.[7] Therefore, it is essential that the moisture content of aggregates used in making the concrete is included in water-cement ratio computations.

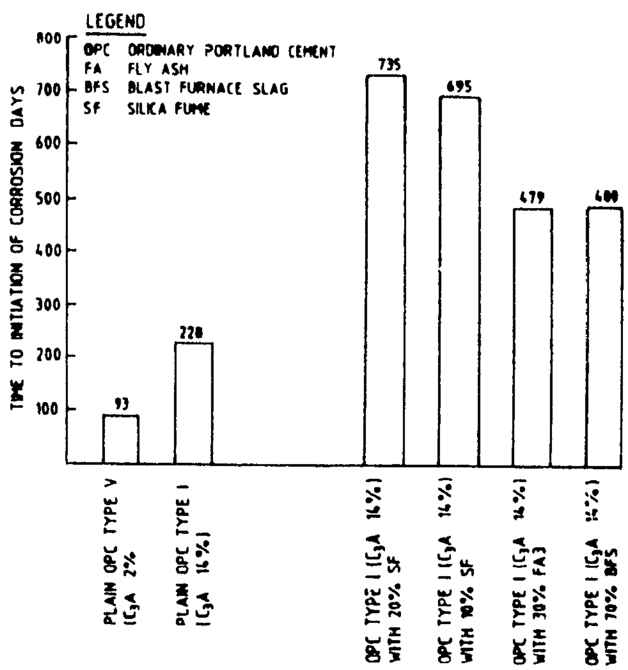


FIGURE 2.36 - INFLUENCE OF PLAIN AND BLENDED CEMENTS ON RESISTANCE TO REINFORCEMENT CORROSION[47]

Aggregates which contain a sufficient amount of chlorides may have a deleterious effect on reinforcement corrosion. Aggregates that conform to ASTM C 33 requirements can be used as well as marine dredged aggregates, provided they are washed with fresh water to reduce the chloride ion content.[46] However, international experience has shown that reducing the chloride ion content in marine aggregates to an acceptable level is very difficult, even after double washing.[49]

2.3.6.7 PERMISSIBLE CHLORIDE CONTENTS

To provide adequate corrosion protection, ACI 318-92[20] limits the maximum water-soluble chloride ion concentration in hardened cement at 28 days to the values shown in Table 2 4

TABLE 2.4 - MAXIMUM CHLORIDE ION CONTENT FOR CORROSION PROTECTION[20]

Type of member	Maximum water soluble chloride ion (Cl) in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0 15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

In cases not covered by the table, the maximum chloride ion content is to be based on a specific test procedure.[48] Results from a recent study,[47] shown in Table 2.5, give threshold chloride contents for various C_3A cements. The results agree very well with the ACI 318-92 limits, for cement containing up to eight percent C_3A.

2.3.6.8 CONCRETE COVER THICKNESS

Concrete cover depth over reinforcing steel is thought by many to be the most important factor influencing reinforcement corrosion. Additional concrete cover delays the ingress of moisture and chloride ions, which in turn increases the time-to-corrosion period.[48] The effect of the concrete cover thickness on reinforcement corrosion is influenced by several parameters, as shown by the expression below:[7]

$$R_t = \frac{41 \times S_i^{1.22}}{K^{0.42} \times (w/c)}$$

where, R_t = time-to-corrosion of concrete exposed continuously to saline water, years

S_i = depth of concrete cover, cm

K = chloride ion concentration, ppm

w/c = water-cement ratio

An example of this relationship is shown graphically in Figure 2.37, below.

TABLE 2.5 - THRESHOLD CHLORIDE VALUES FOR DIFFERENT C_3A CEMENTS[47]

Cement No.	C_3A content of cement, percent by weight of cement	Alkalies as equivalent Na_2O content of cement, percent by weight	Threshold chloride, percent by weight of cement	
			Free Cl^-	Total Cl^-
1	2.04	0.58	0.135	0.40
2	7.59	0.60	0.165	0.62
3	8.52	0.43	0.170	0.65
4	14.00	0.65	0.215	1.00

The ACI 357-R84[45] report on offshore structures provides recommendations for concrete cover for various exposure zones (Table 2.6). Current practice recommends providing a minimum 65 mm (2.5 in.) of concrete cover for conventional concrete and 90 mm (3.5 in.) on prestressing steel for structures in the splash and atmospheric zone exposed to seawater spray. AASHTO recommends 100 mm (4 in.) of concrete cover for such exposure except for precast piles.[15]

Research by Lin and Jou[42] confirmed that chloride ion penetration in concrete marine structures can be predicted by diffusion theory and Fick's second law. Based on test results, the required concrete covers for effective service lives of 10, 30, and 50 years are 51 mm (2 in.), 88 mm (3.5 in.), and 114 mm (4.5 in), respectively, with a survival probability of 0.95.

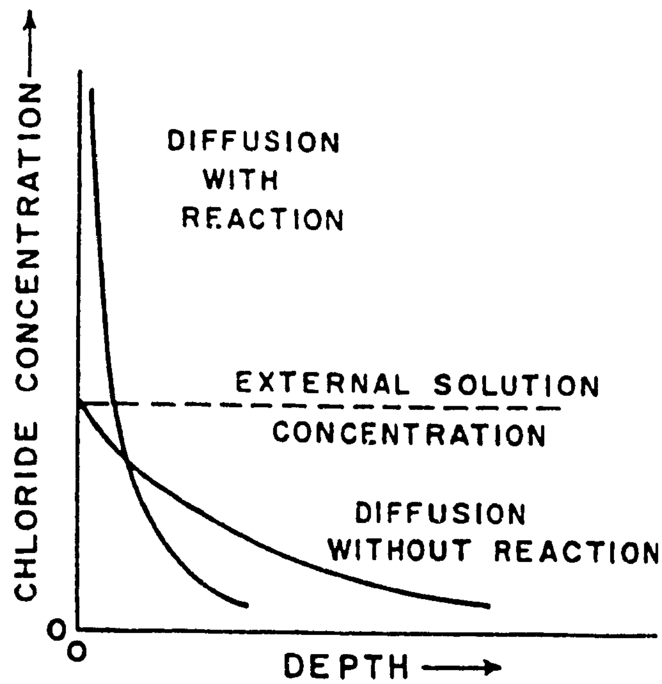


FIGURE 2.37 - CHLORIDE CONCENTRATION VS. DEPTH OF CONCRETE COVER[7]

TABLE 2.6 - RECOMMENDED NOMINAL CONCRETE COVER OVER REINFORCEMENT[45]

Zone	Cover over reinforcing steel		Cover over post-tensioning ducts	
Atmospheric zone not subject to salt spray	50 mm	(2 in.)	75 mm	(3 in.)
Splash and atmospheric zone subject to salt spray	65 mm	(2.5 in.)	90 mm	(3.5 in.)
Submerged	50 mm	(2 in.)	75 mm	(3 in.)
Cover of stirrups	13 mm less than those listed above	(½ in.)		

2.3.6.9 COMPACTION

The quality of concrete compaction directly affects reinforcement corrosion. Inadequate compaction during placement facilitates the ingress of elements conducive to reinforcement corrosion. For instance, reducing the degree of compaction by 10 percent can reduce the concrete compressive strength by 50 percent, reduce bond by 75 percent, and increase chloride permeability by 100 percent.[46]

A study conducted by the Federal Highway Administration (FHWA)[50] in the U S A., demonstrated the importance of proper consolidation The study showed that poorly compacted concrete with a water-cement ratio of 0.32, was less resistant to chloride penetration than well-compacted concrete with a water-cement ratio of 0.60. Good compaction can usually be achieved by using internal or immersion-type (poker) vibrators.

2.3.6.10 CURING

Careful curing, with control of both temperature and moisture, is essential for reducing concrete permeability. If the concrete is inadequately cured, the permeability of the surface layer of concrete may be increased by five to ten times.[6] If the curing period is too short, the protective passive film will not develop before chloride ions penetrate the concrete.[7] Accordingly, ACI Committee 201[15] recommends at least seven days of uninterrupted moist curing, or membrane curing. ACI Committee 308[51] also provides current recommendations for curing concrete. The effect of curing time on concrete permeability is demonstrated in Table 2.7.

TABLE 2.7 - EFFECT OF CURING ON PERMEABILITY[7]

Days of Curing	Coefficient of Permeability
fresh paste	1,150,000,000
1	36,300,000
2	2,050,000
3	191,000
4	23,000
5	5,900
7	1,380
12	195
24	46

If concrete members are cured with low pressure steam, additional moist curing at normal temperatures is usually beneficial.[5] However, unless appropriate precautions are taken, steam-cured mass concrete structures will experience internal microcracking as a result of differential thermal strains.[3] The relationship between steam and moist curing and the corrosion initiation time of embedded reinforcing steel in concrete can be represented by the following empirical expression:[7]

$$P = a \times D^b$$

where,

- P = time to activate corrosion potential for partial immersion in saturated sodium chloride solution, days
- D = time of underwater curing following initial curing, days
- a = 6.33 for steam curing; 6.00 for moist curing
- b = 0.66 for steam curing; 0.90 for moist curing

2.3.6.11 PERMISSIBLE CRACK WIDTH

As previously stated, the thickness of the concrete cover plays an important role with regard to the influence of cracks on reinforcement corrosion.[6] ACI 224-90[5] limits the maximum permissible crack width to 0.15 mm (0.006 in.) at the tension side of reinforced concrete structures which are exposed to wetting and drying cycles or seawater spray (Table 2.8).

TABLE 2.8 - TOLERABLE CRACK WIDTHS IN REINFORCED CONCRETE[5]

Exposure condition	Tolerable crack width, in.	(mm)
Dry air or protective membrane	0.016	(0.41)
Humidity, moist air, soil	0.012	(0.30)
Deicing chemicals	0.007	(0.18)
Seawater and seawater spray; wetting and drying	0.006	(0.15)
Water retaining structures*	0 004	(0 10)

*Excluding nonpressure pipes

The CEB Model Code states that the width of a crack adjacent to the reinforcing steel should be less than 0.1 mm (0.004 in) if the concrete member is exposed to frequent flexural loads, and 0.2 mm (0.008 in) for other structures. The International Prestressing Federation (FIP) recommends a maximum crack width of 0.004 times the nominal concrete cover.[1]

A direct relationship between crack width and corrosion has not been firmly established, however, exposure tests and site inspections seem to indicate that the influence of crack width on the corrosion rate is relatively small for crack widths up to 0.4 mm (0.016 in.).[6] Mehta and Gerwick[3] suggest that by increasing the permeability of concrete and exposing it to numerous processes of deterioration, existing microcracks will eventually cause severe deterioration.

Cracks that propagate transversely to the reinforcement are not as harmful as longitudinal cracks (along the reinforcement). This is because in the case of transverse cracks, corrosion is limited to a small area, so that the risk of spalling of the concrete cover is low. In cases where the horizontal surfaces are in contact with chloride-contaminated water, transverse cracks may cause serious deterioration. Under these circumstances, limiting the crack width will not reduce the risk of reinforcement corrosion.[6] As a result, special protective measures must be implemented.

2.3.6.12 PROTECTIVE COATINGS

Providing good quality concrete with adequate cover is just one of many ways to protect concrete from reinforcement corrosion. Many protective systems have been used with varying degrees of success.[15] Reinforcing bar coatings and cathodic protection are other forms of corrosion protection and are usually more expensive than producing and placing low-permeability concrete.

The two basic types of protective coatings are: anodic coatings (e.g., zinc-coated steel) and barrier coatings (e.g., epoxy-coated steel). Cathodic protection techniques on the other hand, render the concrete environment inconducive to reinforcement corrosion either by forcing the ionic flow in the opposite direction or by using sacrificial anodes.[1] Both systems have been used with mixed results. Other systems include the use of concrete surface sealers or coatings. Reference 52 describes advantages, disadvantages, and cost impact of various corrosion-protection systems.

2.4 DETERIORATION BY CHEMICAL REACTIONS

Another form of concrete deterioration is caused by "chemical interactions between aggressive agents present in the external environment and the constituents of the cement paste".[1] The rate at which these reactions occur will determine the durability of a concrete structure. The ease with which the aggressive substance penetrates the concrete determines the rate at which deterioration progresses. The accessibility of these substances will be determined by the permeability of the originally sound concrete, temperature, or by the passivating layer of the products that are produced as a result of the reaction.[6]

The rate of chemical attack on concrete will also depend on the pH of the aggressive fluid.[1] A well-hydrated portland cement paste, will contain high concentrations of Na^+, K^+, and OH^- ions which produce a high value of pH of about 12.5 to 13.5. Therefore, when portland cement concrete is exposed to an acidic solution (low pH), the alkalinity of the pore fluid will decrease which leads to destabilization of the cement paste constituents. For instance, free CO_2 found in soft and stagnant waters, acidic ions such as SO_4^{2-} and Cl^- in groundwater and seawater, and H^+ ions in some industrial waters will usually lower the pH of concrete to below 6. Therefore, most industrial and natural waters can be considered to be aggressive to portland cement concrete.[1] Acid rain, which has a pH of 4 to 4.5, can etch concrete surfaces.[53]

The chemical reactions that may lead to a decrease in concrete quality can be divided into three subgroups as shown in Figure 2.38.[7] The most important are:[6]

- The reaction of acids, and salts of ammonium or magnesium, and soft water with hardened cement
- The reaction between sulfates and aluminates in the concrete
- Alkali-silica reactivity
- Corrosion of embedded reinforcing steel

The Portland Cement Association (PCA) has published a report on the effects of the various substances on concrete along with a guide to protective treatments.[53] The effects of some of the more common chemicals on the deterioration of concrete are summarized in Table 2.9.

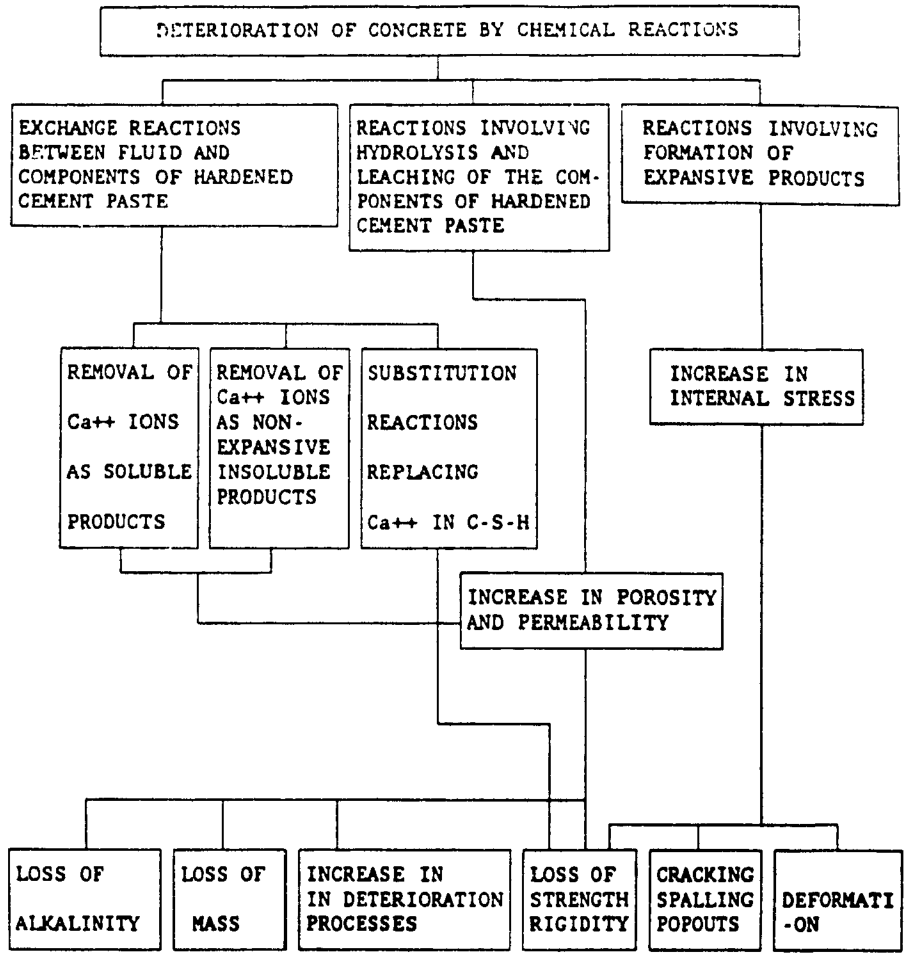


FIGURE 2.38 - CHEMICAL CAUSES OF CONCRETE DETERIORATION[7]

2.4.1 HYDROLYSIS AND LEACHING OF CEMENT PASTE COMPONENTS

Water which conntains chlorides, sulfates, and bicarbonates of calcium and magnesium is generally not aggressive to concrete. On the other hand, water from condensation of fog or water vapor, and water from rain or melting snow and ice, does not usually contain calcium ions and tends to dissolve the calcium-containing products in portland cement paste. When the water is stagnant, the solution in contact with the concrete achieves chemical equilibrium and ceases to dissolve the cement paste. However, when the contact solution is continuously diluted by flowing water, the hydrolysis of the cement paste will continue. The reaction will continue to occur until all or most of the calcium hydroxide has been leached away, leaving behind weak silica and alumina gels. The leaching of calcium hydroxide from concrete interacts with carbon dioxide present in the air and produces efflorescence (white crusts of calcium carbonate).[1]

TABLE 2.9 - EFFECTS OF COMMONLY USED CHEMICALS ON CONCRETE[15]

Rate of attack at ambient temperature	Inorganic acids	Organic acids	Alkaline solutions	Salt solutions	Miscellaneous
Rapid	Hydrochloric Hydrofluoric Nitric Sulfuric	Acetic Formic Lactic	—	Aluminum Chloride	—
Moderate	Phosphoric	Tannic	Sodium Hydroxide > 20 percent*	Ammonium nitrate, Ammonium sulfate, Sodium sulfate, Magensium sulfate, Calcium sulfate	Bromine (gas), Sulfite liquor
Slow	Carbonic	—	Sodium hydroxide 10-20 percent,* Sodium hypochlorite	Ammonium chloride, Magnesium chloride, Sodium cyanide	Chlorine (gas), Seawater Softwater
Negligible	—	Oxalic Tartaric	Sodium hydroxide < 10 percent,* Sodium hyprochlorite, Ammonium hydroxide	Calcium chloride, Sodium chloride, Zinc nitrate, Sodium chromate	Ammonia (liquid)

* Avoid siliceous aggregates because they are attacked by strong solutions of sodium hydroxide

2.4.2 ACID ATTACK (CATION EXCHANGE REACTIONS)

Portland cement is generally not resistant to acid attack. Concrete deterioration by acid attack is caused by the reaction between the acid solution and the calcium hydroxide in the portland cement paste. The chemical reaction produces water-soluble calcium salts which are removed by the erosive action of flowing water (Figure 2.39). In some cases, the resulting calcium salts are insoluble and are not easily removed from the concrete.[15]

The rate at which a reaction occurs with the concrete depends more on the solubility of the resulting calcium salt than on the aggressiveness of the acid. The rate of deterioration will also be much higher in flowing water conditions. It should also be realized that, with acid attack, the hardened cement paste is completely converted to a soluble salt, thereby destroying the entire pore structure. Therefore, unlike other types of attack, the permeability of the originally sound concrete is of minor importance.[6] The three types of deleterious cation-exchange reactions that can occur between acids and concrete are summarized below.

2.4.2.1 FORMATION OF SOLUBLE CALCIUM SALTS

Solutions containing hydrochloric, sulfuric, or nitric acid are often found in industrial practice. The reactions that occur between these acids and the portland cement paste produce soluble calcium salts, such as calcium chloride, calcium acetate, and calcium bicarbonate, which are subsequently removed by leaching. Also, reactions of solutions of ammonium chloride and ammonium sulfate with concrete produce highly soluble products as shown by the reaction below:[1]

$$2NH_4Cl + Ca(OH)_2 \rightarrow CaCl_2 + 2NH_4OH$$

Carbonic acid is also very aggressive to concrete. The aggressiveness of the reaction between carbonic acid and calcium hydroxide [$Ca(OH)_2$] present in hydrated portland cement paste is dependent on the amount of free dissolved CO_2 in the attacking solution. The reaction can be shown as follows:[1,36]

$$Ca(OH)_2 + H_2CO_3 \rightarrow CaCO_3 + 2H_2O$$

$$CaCO_3 + CO_2 + H_2O \rightleftharpoons Ca(HCO_3)_2$$

If there is a sufficient amount of free CO_2 present in the solution, the insoluble $CaCO_3$ is transformed into soluble calcium bicarbonate as shown by the second reaction Any amount of free CO_2 over and above that which is needed for equilibrium would cause the second reaction to move to the right, thus accelerating the hydrolysis of calcium hydroxide. When the pH of groundwater or seawater is above 8, the amount of CO_2 present is negligible. However, if the pH is below 7, it may contain significant amounts of CO_2.

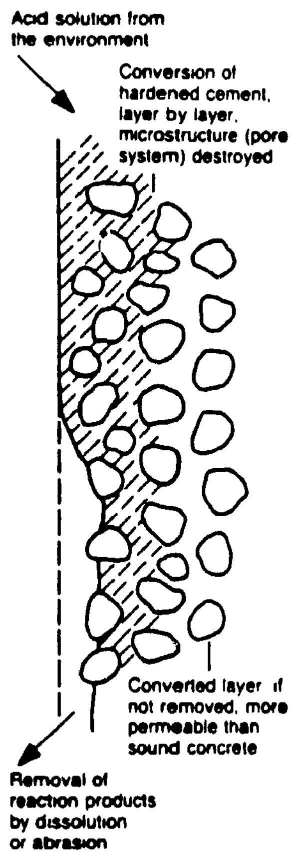


FIGURE 2.39 - EFFECT OF ACID ATTACK ON CONCRETE[6]

2.4.2.2 FORMATION OF INSOLUBLE AND NONEXPANSIVE CALCIUM SALTS

Certain waters which contain agressive anions may react with cement paste to form insoluble calcium salts, which may or may not cause damage to the concrete depending on whether the reaction product is either expansive or is removed by erosion. Chemicals belonging to this category include oxalic, tartaric, tannic, humic, hydrofluoric, and phosphoric acid.

2.4.2.3 MAGNESIUM ION ATTACK

Seawater or groundwater often contain solutions of magnesium bicarbonate and when these waters come in contact with concrete, they react with the calcium hydroxide in portland cement paste to form soluble calcium salts. Magnesium ion attack is considered to be the most aggressive because it eventually extends to the calcium silicate hydrate (C-S-H) in the cement. Prolonged contact with magnesium ions leads to the formation of a weak magnesium silicate hydrate.

2.4.2.4 RECOMMENDATIONS FOR CONTROL

Concrete with a low water-cement ratio and a well-graded aggregate may provide adequate resistance to mild acid solutions. In situations where the acidic solution is stagnant, a "sacrificial" calcareous aggregate may be beneficial. The acid may sometimes be neutralized by replacing the siliceous aggregate with limestone or dolomite having a minimum calcium oxide concentration of 50 percent. In this case, the acid attack will be more uniformly distributed, reducing the rate of attack on the cement paste and minimizing loss of aggregate particles.[53]

No concrete, regardless of its quality, will resist long exposure to high acid concentration. In such cases, it may be possible to apply an adequate protective surface coating to the concrete.[15] ACI Committee 515[54] and the Portland Cement Association[53] provide recommendations for barrier coatings to protect concrete from various chemicals.

2.4.3 REACTIONS INVOLVING FORMATION OF EXPANSIVE PRODUCTS

Five types of reactions that involve the formation of expansive products have been identified as being deleterious to portland cement concrete: sulfate attack, alkali-silica attack, alkali-carbonate attack, delayed hydration of free CaO and MgO, and corrosion of steel in concrete.[1] These reactions can cause closure of expansion joints, deformation and displacements in various parts of the structure, cracking, spalling and pop-outs. A discussion of each reaction follows.

2.4.3.1 SULFATE ATTACK

This form of attack usually occurs when concrete is exposed to solutions containing sulfates of sodium, potassium, calcium, or magnesium. Ammonium sulfate, which is often found in agricultural soil and waters, is also aggressive to concrete. Seawater, which has a high sulfate concentration, can be aggressive to marine structures. Deterioration of concrete as a result of sulfate attack is known to manifest itself in two distinct forms: expansion, and progressive loss of strength and mass (Figure 2.40).[1]

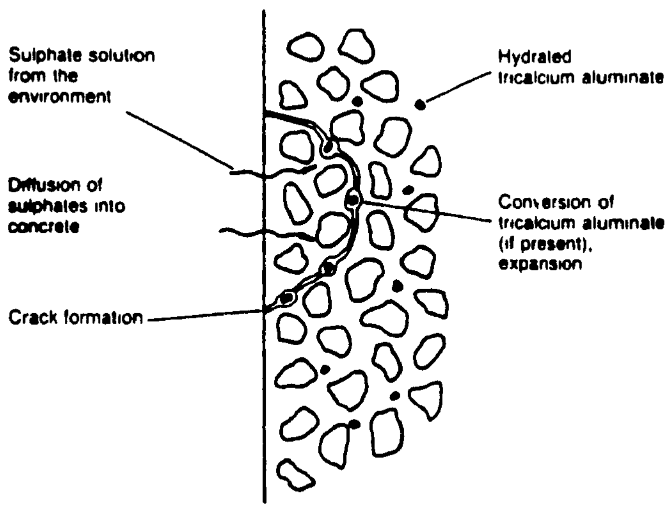


FIGURE 2.40 - EFFECT OF SULFATE ATTACK[6]

It is believed that there are two chemical reactions that occur in sulfate attack on concrete.[15] The first involves the combination of sulfates of sodium, calcium, or magnesium with free calcium hydroxide to form calcium sulfate (or gypsum), as shown by the following reactions:

$$Na_2SO_4 + Ca(OH)_2 + 2H_2O \rightarrow CaSO_4 \cdot 2H_2O + 2NaOH$$

$$MgSO_4 + Ca(OH)_2 + 2H_2O \rightarrow CaSO_4 \cdot 2H_2O + Mg(OH)_2$$

$$3MgSO_4 + 3CaO \cdot 2SiO_2 \cdot 3H_2O + 8H_2O \rightarrow 3(CaSO_4 \cdot 2H_2O) + 3Mg(OH)_2 + 2SiO_2 \cdot H_2O$$

As seen by the second reaction, magnesium sulfate creates the most severe attack on concrete. In addition to the formation of gypsum, the reaction produces a poorly alkaline magnesium hydroxide, which creates an unstable environment around the calcium-silicate-hydrate (C-S-H) binder. In such cases, calcium silicates release calcium hydroxide, converting part of the C-S-H binder into a cohesionless granular mass, in addition to the expansive cracking.[47]

The second reaction involves the combination of gypsum and hydrated calcium aluminate to form calcium sulfoaluminate (ettringite). This is represented by the following equations:[1,15]

$$C_3A \cdot C\overline{S} \cdot H_{18} + 2CH + 2\overline{S} + 12H \rightarrow C_3A \cdot 3C\overline{S} \cdot H_{32}$$

$$C_3A \cdot CH \cdot H_{18} + 2CH + 3\overline{S} + 11H \rightarrow C_3A \cdot 3C\overline{S} \cdot H_{32}$$

It is believed that the formation of ettringite is responsible for the expansion. However, the mechanism by which expansion occurs is not fully understood. Two principal theories which have coexisted for a long time are: exertion of pressure by growth of ettringite crystals, and swelling of "colloidal" ettringite due to adsorption of water. A new theory concerning sulfate expansion in concrete developed by Ping and Beaudoin[55] attempts to explain the process based on the principles of chemical-thermodynamics. It suggests that sulfate expansion is caused by the conversion of chemical energy into mechanical work to overcome the cohesion of the system. The expansive force comes from crystallization pressures which occur as a result of the interaction between the ettringite and the cement paste.

2.4.3.2 CONTROL OF SULFATE ATTACK

The main factors influencing expansion are:[6]

- Amount of aggressive substance present
- Permeability of concrete
- Cement type (C_3A content)
- Amount of moisture available

A reasonable degree of protection against sulfate attack can be provided by using a dense, low permeability concrete with a low water-cement ratio and a high cement content. Proper consolidation and curing of fresh concrete with adequate cover thickness produce a high quality concrete with low permeability.[1] Additional safety against sulfate attack can be provided by using sulfate-resisting cements. The ACI Committee 201.2R report[15] and the ACI Building Code 318-92[56] provide recommendations for the type of cement and water-cement ratio for normal weight concrete for various degrees of sulfate exposure. These are classified into four categories and are shown in Table 2.10.

TABLE 2.10 - RECOMMENDATIONS FOR NORMAL WEIGHT CONCRETE SUBJECT TO SULFATE ATTACK[20]

Exposure	Water soluble sulfate (SO_4) in soil, percent	Sulfate (SO_4) in water, ppm	Cement (ASTM)	Water-cement ratio, maximum*
Mild	0.00 - 0.10	0 - 150	---	---
Moderate	0.10 - 0.20	150 - 1500	Type II, IP(MS) IS(MS)	0.50
Severe	0.20 - 2.00	1500 - 10,000	Type V	0.45
Very Severe	Over 2.00	Over 10,000	Type V + Pozzolan*	0.45

*A lower water-cement ratio may be necessary to prevent corrosion of embedded items
Seawate, also falls in this category
Use a pozzolan which has been determined by test to improve sulfate resistance when used in concrete containing Type V cement.

In general, ASTM Type V (CSA Type 50) portland cement which contains less than 5 percent C_3A provides adequate protection against mild sulfate attack. European standards limit the C_3A content of cement to 3 percent for high sulfate resistance.[6] However, in severe sulfate exposure conditions concrete containing high alumina cements, portland blast-furnace slag cements (with more than 70 percent slag), and portland pozzolan cements with at least 25 percent pozzolan (natural pozzolan, calcined clay, or low-calcium fly ash) have provided a higher resistance to sulfate attack.[1] Pozzolans combine with the free lime resulting from the hydration of the cement, thereby reducing the amount of gypsum formed.[57] The best results have been obtained when the pozzolan is a Class F fly ash meeting the requirements of ASTM C 618.[58]

2.4.3.3 ALKALI-SILICA REACTION (ASR)

Alkali-silica reaction (ASR) is a chemical reaction that can occur between aggregates containing certain forms of silica and sufficient alkalies (sodium and potassium) in the cement paste. The phenomenon, which was first described by Stanton[59] in 1940, is reported by the Strategic Highway Research Program (SHRP) as being one of the major causes of concrete deterioration in the United States. Published literature indicates that ASR is also widespread in other parts of the world such as eastern Canada, Australia, New Zealand, South Africa, Denmark, Germany, England, Iceland,[1] India and Turkey.[15] ASR often occurs in marine structures, such as dams, bridge piers, and sea walls.[1]

ASR involves the breakdown of silica structure of the aggregate by hydroxyl ions to form an alkali-silica gel which can swell by absorbing a large amount of water through osmosis. The hydraulic pressure which develops can cause expansion and cracking of the concrete, leading to a loss of strength, elasticity, and durability.[1] ASR will typically produce an irregular crack pattern commonly referred to as map or pattern cracking (Figure 2.41). Such cracking may also be accompanied by displacements or misalignments of structural members.[60] An excellent handbook for identifying ASR in the field has been produced by Stark (SHRP).[61]

Most of the alkalies in concrete come from portland cement, although they are also found in seawater, ground-water, and deicing salts. Reactive silica is found in a variety of mineral forms in aggregates. Their rate of reactivity depends on their morphology, whether amorphous or crystalline.[60] A comprehensive list of deleteriously reactive rocks, minerals, and synthetic substances responsible for concrete deterioration by ASR has been developed by ACI Committee 201 and is shown in Table 2.11. Several of these rocks, including granite gneisses, metamorphosed subgraywacks, and some quartz and quartzite gravels may react slowly.[15] The quantity and reactivity of the reagents, available moisture, and temperature are all contributing factors to the rate of ASR deterioration. Severe structural damage can occur in as little as three years, although in some cases, ASR can take more than 30 years to create any visible damage.[39]

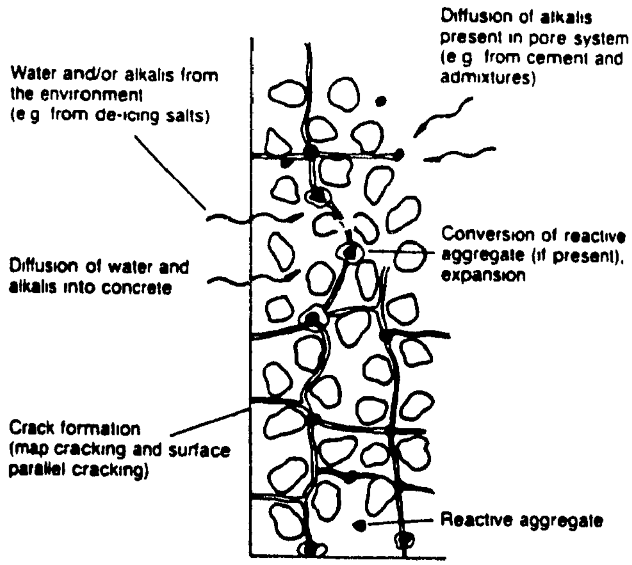


FIGURE 2.41 - EFFECT OF ALKALI-SILICA REACTION[8]

2.4.3.4 CONTROL OF ALKALI-SILICA REACTION

Test procedures for identifying potentially reactive aggregates and specification procedures for preventing or minimizing its effects have been in place for many years. The presence and amounts of reactive constituents in the aggregate can be determined by petrographic examination. Table 2.12, which is adapted from Reference 60, shows the maximum amounts of reactive constituents that can be tolerated in aggregates, and can be compared with petrographic test results. To ensure long-term serviceability of concrete exposed to conditions which promote ASR, Ozol and Dusenberry[63] suggest that quartzose aggregates are to be considered as potentially reactive. In addition, there are certain test methods which can be used to measure the reactivity or the potential to cause expansion. The most commonly used tests include ASTM C 289 and ASTM C 227.[60] The results of these tests are used to verify the findings of the petrographic examination with regard to the reactivity of an aggregate.

TABLE 2.11 - DELETERIOUSLY REACTIVE ROCKS, MINERALS, AND SYNTHETIC SUBSTANCES[15]

Reactive substances	Chemical composition	Physical character
Opal	$SiO_2 \bullet nH_2O$	Amorphous
Chalcedony	SiO_2	Microcrystalline to cryptocrystalline; commonly fibrous
Certain forms of quartz	SiO_2	(a) Microcrystalline to cryptocrystalline; (b) Crystalline, but intensely fractured, strained, and/or inclusion-filled
Cristobalite	SiO_2	Crystalline
Tridymite	SiO_2	Crystalline
Rhyolitic, dacitic, latitic, or andesitic glass or cryptocrystalline devitrification products	Siliceous, with lesser proportions of Al_2O_3, Fe_2O_3, alkaline earths, and alkalies	Glass or cryptocrystalline material as the matrix of volcanic rocks or fragments in tuffs
Synthetic siliceous glasses	Siliceous, with lesser proportions of alkalies, alumina, and/or other substances	Glass

The most important deleteriously alkali-reactive rocks (that is, rocks containing excessive amounts of one or more of the substances listed above) are as follows:

Opaline cherts	Andesites and tuffs
Chalcedonic cherts	Siliceous shales
Quartzose cherts	Opaline concretions
Siliceous limestones	Fractured, strained, and
Rhyolites and tuffs	inclusion-filled quartz
Dacites and tuffs	and quartzites
Siliceous dolomites	Phyllites

Note: A rock may be classified as, for example, a "siliceous limestone" and be innocuous if its siliceous constituents are other than those indicated above.

However, Lane[60] reports that these test procedures may not be effective in detecting all potentially reactive aggregates. To compensate for this shortfall, several new test methods have been developed which are suitable for detecting the reactivity of aggregates which contain microcrystalline or strained quartz. This method is being evaluated by ASTM as P 214 "Proposed Test Method for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction".

Table 2.12 - RECOMMENDED MAXIMUM TOLERABLE PERCENT OF REACTIVE CONSTITUTENTS IN AGGREGATES[60]

Reactive constituent	K & F	U S. CoE
Opal	0.5	0
Chert and chalcedony	3.0	—
Chert with any chalcedony	—	5
Tridymite and cristobalite	1.0	1
Strained or microcrystalline quartz	5.0	—
Highly-strained quartz	—	20
Volcanic glasses (> 55% silica)	0	3

K & F = Kosmatka and Fiorato - U.S. CoE = Unites States Corps of Engineers

Since alkalies in concrete are mostly found in the portland cement, the traditional approach to minimizing ASR has been to use low-alkali cements. The ASTM C 150 specifications recommend using cement with an alkali content not exceeding 0.60 percent Na_2O equivalent (Na_2O + 0.658 K_2O). However, research by Ozol and Dusenberry[63] suggests that a lower value may be more advisable. Tuthill[64] suggests that a maximum limit of 0.40 percent should be used to prevent ASR. Mehta[1] reports that investigations in Germany and England have shown that a total alkali content below 3 kg/m^3, is not likely to cause any damage.

However, since it is difficult for producers to control the amount of alkali content in cement, an alternative measure is to use blended cements made by adding pozzolans or ground granulated blast-furnace slag.[60] Class C and Class F fly ashes and silica fume are the most widely used pozzolans. Silica fume is reported to be the pozzolan which provides the best protection against expansion resulting from ASR. Kosmatka and Fiorato[62] report that Class F fly ashes (low lime) are more effective when provided in amounts of 15 to 20 percent of the total cementitious material. If Class C fly ashes (high lime) are used, a replacement of 35 to 40 percent would provide the same degree of protection. Hogan and Meusel[50] suggest that cements containing 40 percent or more slag can considerably reduce expansion. The standard test method for determining the mix proportions to achieve the desired performance is ASTM C 441.

2.4.3.5 ALKALI-CARBONATE REACTION (ACR)

Certain limestone aggregates, have been reported to be reactive in concrete structures in Canada and the United States.[15] The mechanism of attack, involves "dedolomitization" of the carbonate rocks which leads to the formation of brucite and the regeneration of alkali. This effect is opposite to what occurs with ASR in which the alkalies are consumed as the reaction occurs. However, concrete deterioration by ACR is also characterized by expansion and map cracks and is more severe in areas where there is a continuous supply of moisture. Another difference between the two reactions is that ACR does not exude silica gel.

In addition to expansion, a phenomenon associated with ACR is the formation of rims around the aggregate particles and extensive carbonation of the surrounding paste. This phenomenon is not fully understood, however, ACI Committee 201[15] reports that it occurs as a result of "a change in disposition of silica and carbonate between the aggregate particle and the surrounding cement paste". The rims seem to propagate concentrically toward the center of the aggregate with time. Damage due to ACR usually occurs in less than three years.[39]

2.4.3.6 DELAYED HYDRATION OF CRYSTALLINE MgO AND CaO

Hydration of free crystalline MgO or CaO, can also cause expansion and cracking in concrete if they are present in sufficient amounts. Current ASTM C 150 restrictions require that the MgO content in cement should not exceed 6 percent. However, laboratory tests showed that concrete made with a low-MgO portland cement containing sufficient amounts of CaO can also produce expansion.[1] Current manufacturing practices ensure that the content of free crystalline CaO in portland cement does not exceed one percent.[7]

2.5 CONCRETE DETERIORATION DUE TO BIOLOGICAL PROCESSES

2.5.1 BACTERIOLOGICAL GROWTH

Bacteriological growth such as lichen, moss, algae, and roots of plants and trees penetrating into the concrete at cracks, can cause mechanical deterioration to concrete structures. Microgrowth may also cause chemical attack by producing humic acid which is aggressive to the cement paste.[8]

However, the most common type of biological attack on concrete is found in the sewer systems. Sulfur-oxidizing bacteria, using hydrogen sulfide (H_2S) derived from the sewage, produces high concentrations of sulfuric acid, thus resulting in acid and sulfate attack in the concrete (Figure 2.42). Rigdon and Beardsley[66] noted that this problem commonly occurs in warm climates such as California (U.S.A.), Australia, and South Africa. Pomeroy[67] also observed that this phenomenon occurs at the end of long pumped sewage force mains in the northern (colder) climates.

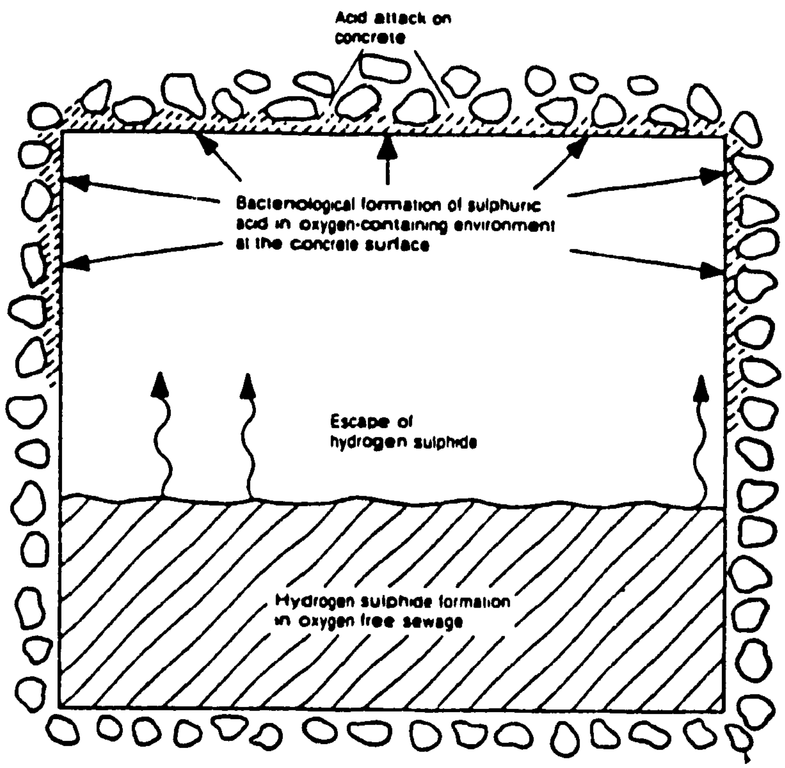


FIGURE 2.42 - BIOLOGICAL ATTACK IN SEWER SYSTEMS[6]

The mechanism by which sulfuric acid is formed involves two distinct processes which will not occur unless certain conditions are met. Generally, a free water surface is required, and a low dissolved oxygen content in the sewage that permit the buildup of anaerobic (oxygen-free) sulfur-reducing bacteria. Some of these bacteria reduce the sulfates or proteins that are present in the sewage to form H_2S. Sulfur-oxidizing bacteria (thiobacillus concretivorus) reduce the H_2S to form sulfuric acid, which lowers the pH of concrete to 2 or less. The destructive effect of the sulfate ions on the calcium aluminates in the cement paste account for the deterioration of concrete, which usually occurs in the crown of the sewer.[29]

Information which may enable the engineer to design, construct and operate a sewer so that the formation of sulfuric acid is reduced is provided in References 67, 68 and 69.

2.5.2 MARINE GROWTH

Marine organisms are commonly found on the surface of underwater concrete structures. This marine growth (or fouling) can have significant adverse effects on the integrity of the structure. Firstly, it increases the surface area of the profile that is exposed to current flow, thereby increasing wave and current forces on the structure. Secondly, marine growth can also cause deterioration of some concrete structures due to galvanic action between the organism and the concrete.[70] In tropical or semi-tropical waters, several types of marine rock borers can penetrate into the concrete, although this damage usually occurs in low-quality concrete.[71]

2.5.2.1 TYPES OF MARINE GROWTH

Marine growth can be categorized into two basic types: soft fouling and hard fouling.[72] Soft fouling is caused by organisms which have the same density as seawater. This type of marine growth creates bulk, but is usually easy to remove. Hard fouling is caused by marine organisms that are denser than water and are more firmly attached to the concrete surface. These organisms are usually more difficult to remove. The following sections have been adapted from a review of Reference 72, and are provided for reader information.

2.5.2.2 SOFT FOULING

Those organisms which cause soft marine growth are as follows:

(a) Algae This is usually the first type of marine growth to appear on the surface of an underwater structure and is usually referred to as slime. Being sensitive to light, algae is not usually found below 20 m (66 ft.) of water depth.

(b) Bacteria. This will also be one of the first organisms to develop on a structure and will grow in water depths in excess of 1000 m (3280 ft.).

(c) Sponges. These are often found on the surfaces of deep offshore structures at depths greater than 1000 m (3280 ft.). See Figure 2.43.

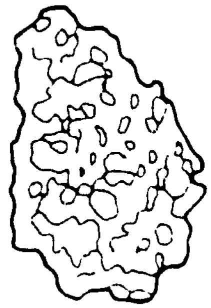


FIGURE 2.43 - A SPONGE[72]

(d) Sea squirts. These organisms have a soft body and are sometimes found in large colonies. See Figure 2.44.

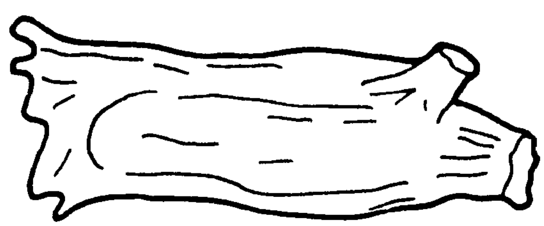


FIGURE 2.44 - A SEA SQUIRT[72]

(e) Hydroids. These are related to the sea anemone and also grow in colonies. They resemble seaweed, and can produce dense colonies to depths of 1000 m (3280 ft.). See Figure 2.45.

(f) Seaweeds. There are several varieties of seaweed that grow on underwater concrete structures. The longest of these is kelp, which can produce fronds up to 6 m (20 ft.) in length, if the conditions are suitable. See Figure 2.46.

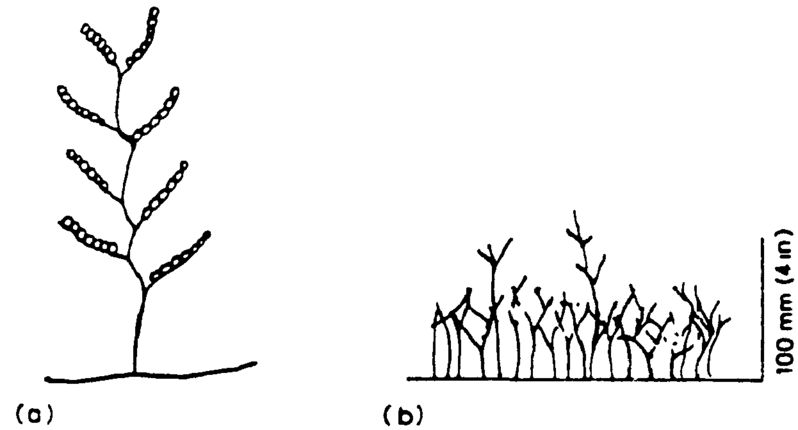


FIGURE 2.45 - HYDROIDS: (a) PROFILE OF A SINGLE HYDROID; (b) COLONY OF HYDROIDS[72]

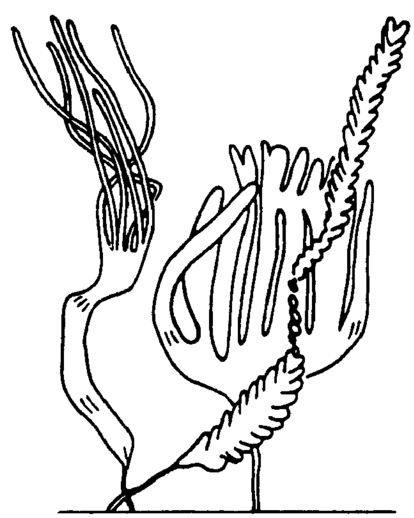


FIGURE 2.46 - SEAWEED: PROFILES OF THREE VARIETIES OF KELP WEED[72]

(g) Bryozoa. This marine growth looks like moss and grows very tall. Bryozoa is actually an animal with tentacles, as shown in Figure 2.47.

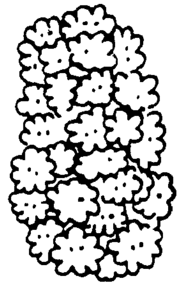


FIGURE 2.47 - BRYOZOA[72]

(h) Anemones. The anemone, sometimes referred to as anthozoan, has a cylindrical shaped body which is surrounded by tentacles. It attaches itself firmly to the concrete surface by a disc-shaped base and is difficult to remove without tearing its body. The species is found in many shapes and colors.

(i) Dead men's fingers (Alcyonium digitalum). These colonies which can grow up to 150 mm (6 in.) in length can be found on pier piles and rocks on waterfront and offshore structures. When they are below water, several small polyps grow out from the finger-shaped body, with each polyp having eight feathery tentacles (Figure 2.48). When submerged, its color is white to yellow or pink to orange. When it is out of the water, it is flesh colored and resembles the human hand.

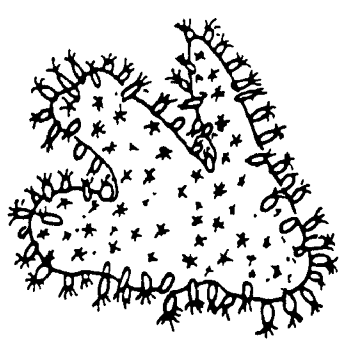


FIGURE 2.48 - DEAD MEN'S FINGERS[72]

2.5.2.3 HARD FOULING

This group of marine growth is comprised of calcareous or shelled organisms and includes the following:

(a) Barnacles This form of marine growth is the one most commonly found attached to waterfront structures. The common species is called *Balanus balanoides* (Figure 2.49). These organisms grow in dense colonies to a depth of 15 to 20 m (49 to 66 ft.), but can also grow at depths of 120 m (394 ft.).

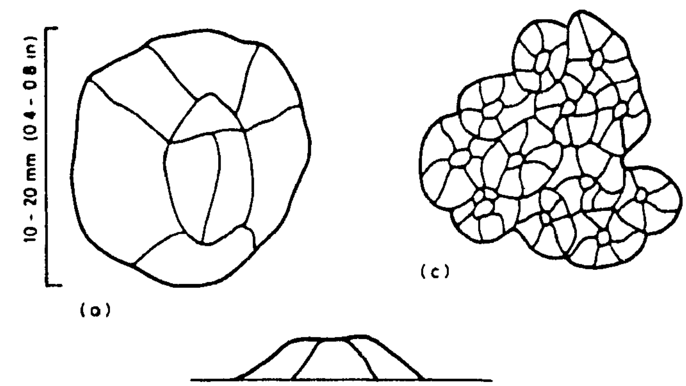


FIGURE 2.49 - BARNACLE: (a) VIEW OF THE TOP OF A BARNACLE; (b) SIDE VIEW; (c) TOP VIEW OF A CLUSTER OF BARNACLES[72]

(b) Mussels. A mussel is a hard-shelled mollusc which attaches itself firmly to the structure by very strong threads located at the hinge of the shell. The main species, *Mytilus edulis*, is usually found in dense colonies to depths of between 20 m (66 ft.) and 50 m (164 ft.)

2.5.2.4 FACTORS INFLUENCING MARINE GROWTH

The type of organism which develops, and its growth rate, will depend on several factors including water depth, temperature, current, salinity, and food supply. In general, the formation of slime or algae on unprotected concrete surfaces occurs in two to three weeks. In some cases, marine growth has been know to develop within 24 to 36 hours after a surface has been cleaned. On the other hand, barnacles and soft fouling can develop in three to six months. Mussel colonies generally take two seasons to develop, and can also grow on top of existing dead fouling. The various factors affecting growth rate are summarized below.

(a) Depth. In general, marine growth density decreases with increasing water depth, since an increase in depth reduces light intensity. This reduction in light intensity reduces the ability of certain organisms, such as algae, to photosynthesize. A generally accepted schematic representation of marine growth density with varying water depth (in British waters) is shown in Figure 2.50. From the diagram it is evident that the highest density of marine growth occurs near the water surface, which is where wave loads are the highest.

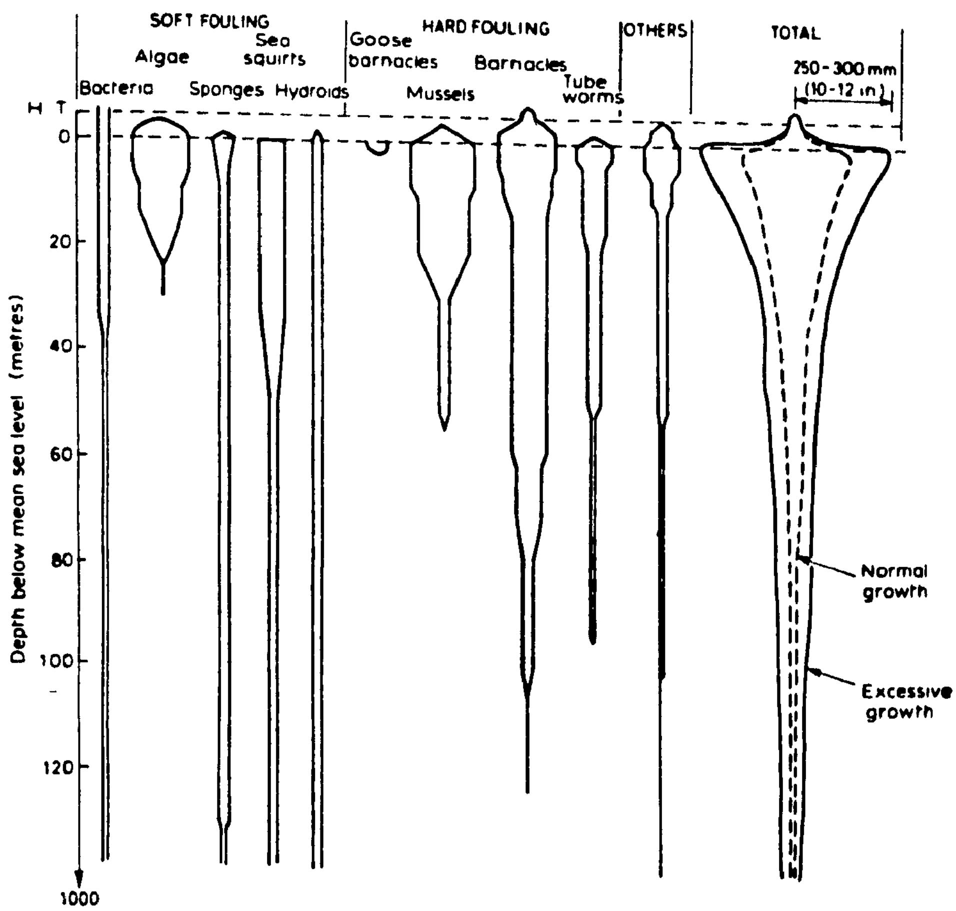


FIGURE 2.50 - SCHEMATIC REPRESENTATION OF THE DISTRIBUTION OF MARINE GROWTH WITH WATER DEPTH[72]

(b) Temperature. An increase in water temperature will typically increase the growth rate of marine organisms. For instance, an increase in temperature of 10°C (18°F) will approximately double the growth rate. However, most organisms stop growing at 30 to 35°C (86 to 95°F). Since the greatest variation of temperature occurs near the water surface, marine fouling colonies near the surface will undergo seasonal growth.

(c) Water current. Water flow velocity is an important factor influencing the type of colony that develops. It appears that many larvae cannot attach themselves to concrete surfaces if the water velocity exceeds 0.5 m/sec (1.6 ft/sec). However, once attached, it can withstand water velocities of more than 3m/sec (10 ft/sec). At higher velocities, fouling which is not firmly attached is easily removed. During slack flow periods, larvae can attach themselves to structures in cracks, where water flow is either slow or stagnant. Once the organism develops, strong water currents will provide more nutrients which will accelerate growth.

(d) Salinity. In fresh water, the only organism whch can grow is marine algae slime. The amount and type of marine growth increases with increasing salinity. For example, the size of mussels will be five times greater as the salinity of water increases from 0.6 percent to the normal salinity of seawater (3 to 3.5 percent). As the salinity increases, hydroids will grow first followed by mussels.

(e) Food supply. Marine growth depends on the amount of nutrient available. Growth rates in coastal areas (shallow water) are higher than those offshore (deep water). Research has shown that marine organisms found in water which circulates around offshore structures have a higher growth rate.

2.6 CONCRETE DAMAGED BY HARD IMPACT

A marine structure may be subjected to several forms of collisions or impact. Examples of these include ship collisions, wave action, and dropped objects. Depending upon the type and behavior of the impactor, the response of the concrete structure may either be global such that little or no local damage will occur, or if the impactor hardness and/or velocity of impact is high enough, local damage will occur.[73] This section focuses on the latter, and is adapted from a review of Reference 73.

As a consequence of several serious accidents involving concrete offshore structures in the North

Sea (both from dropped objects and ship collisions), the Department of Energy in the UK sponsored several studies to investigate the problem further. A report by Wimpey[74] identified the various types of objects that are likely to be dropped on concrete offshore structures. A subsequent report by Brown and Perry[75] identified the various forms of local damage which can occur from fallen objects and developed simple design formulae for assessing the damage. These are discussed below.

2.6.1 FORMS OF LOCAL IMPACT DAMAGE

The report by Wimpey[74] concluded that the objects most likely to cause severe damage are the end-on impact of slender objects and impact of bulky objects. Brown and Perry[75] recognize five forms of damage (Figure 2.51) and have adopted a standardized description for each as defined below:

- Penetration (the depth to which an object penetrates the concrete).
- Spalling (the cratering damage on the impacted surface).
- Scabbing (the fracturing and expulsion of concrete from the opposite face of the impact).
- Perforation (the object passes completely through the concrete).
- Shear plug (formed by inclined cracking through the thickness of the concrete).

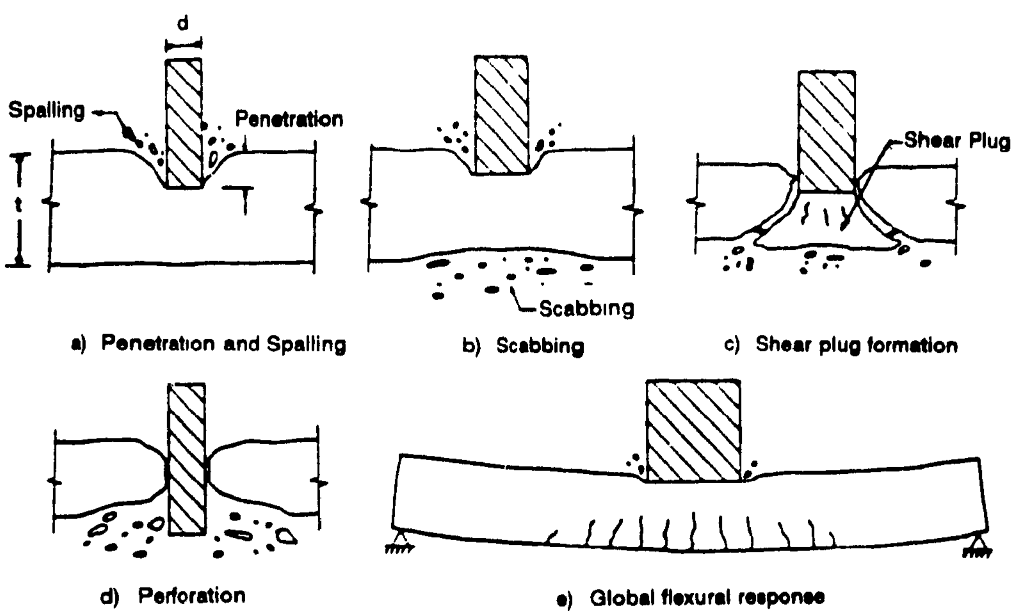


FIGURE 2.51 - FORMS OF IMPACT DAMAGE[75]

The type of damage which will most likely occur depends on the ratio of concrete thickness to the object diameter (t/d). For high t/d values (of the order of 6), penetration and spalling are likely to occur. For t/d values approaching 1, scabbing and shear plug formation are more likely to occur. For very low values of t/d, a global response may cause the major damage, although scabbing can also occur. Factors influencing the response of concrete slabs to impact are discussed in Reference 76.

2.6.2 ASSESSMENT OF LOCAL IMPACT DAMAGE

The report by Brown and Perry[75] concludes that present impact damage assessment methods would provide reliable results, but that the basis for established scabbing formulae was inadequate. As a result, new empirical formulae for scabbing were developed. For solid objects, a non-dimensional number N_1, is given by the following formula:

$$N_1 = \frac{m^{0.5} \times V}{d_o^{0.5} \times t \times (1+t/d_o)} \times \frac{E^{0.5}}{f_r}$$

where,
- M = mass of missile (or impactor)
- V = velocity of missile
- d_o = diameter of missile
- t = slab thickness
- E = modulus of elasticity
- f_r = concrete shear strength

If N_1 is greater than some critical value, the nominal shear stress will be greater than the nominal shear strength of the concrete and inclined cracking, shear plug formation, and scabbing are likely to occur. The same procedure is used for pipe-shaped objects.

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CHAPTER 3

UNDERWATER INSPECTION OF CONCRETE STRUCTURES

3.1 INTRODUCTION

The service life of any structure, such as a dam, navigation lock, bridge, wharf, or other marine structure, depends on the preservation of the physical condition of both the portion of the structure above and below the waterline. Therefore, it is important to develop and implement an adequate inspection, maintenance, and repair program for the *entire* structure. In many cases, however, underwater inspections are seldom performed because the evaluation of the condition of a concrete structure under water is usually more difficult and expensive than evaluating a structure located above the waterline.[1] However, aging structures reaching or exceeding their design life, and the growing concern with concrete durability both contribute to making underwater inspection an essential part of today's infrastructure evaluation and preservation technology.

Various groups, such as Transportation agencies, and Port Authorities in Canada, the United States, and other parts of the world require periodic underwater inspections as part of a preventive maintenance program. They are also undertaken as a requirement prior to the purchase of a facility by a new owner, to evaluate the strength of the structure for new loading conditions, to gather information needed for planning the expansion or modification of a facility or as an initial construction inspection to confirm that a structure has been constructed in accordance with contract documents. Catastrophic events, such as ship collisions, earthquakes, hurricanes, and floods also require underwater inspections for damage assessment.[2] In addition, deteriorated structures that might be dangerous to public safety, or that can cause substantial property damage, need to be continually inspected to determine its capacity to operate safely.[3]

Underwater inspections can be performed by divers, remotely operated vehicles (ROVs), or manned submersibles. The most common method employed is the use of commercial/engineer divers. They are readily available at almost all waterfront locations and can be mobilized relatively quickly. ROVs are becoming used more frequently for visual inspections and have also been used for making repairs. The basic ROVs are usually equipped with cameras, videos, and lights, and are remotely controlled from the surface. ROVs can be very economical in deep-water or long penetration dives. Manned submersibles are used primarily for performing very deep dive inspections of structures such as offshore oil platforms and pipelines.[4] These types of vehicles are rarely used for waterfront inspections. A more detailed discussion of each of the above diving methods is provided later in this chapter.

Evaluation of a structure must take into account several factors, including design considerations, existing operating, inspection, and maintenance records, condition surveys, in-situ testing, instrumentation, and determination of the phenomena causing the deterioration.[3] This section provides a summary of techniques and equipment currently used by divers to visually inspect concrete in existing underwater structures or underwater portions of concrete structures. The planning and preparation required for an underwater inspection program, and methods of documenting and presenting the results are included along with a description of the recommended underwater inspection procedures. The material in this chapter has been adapted from a review of different available references, especially 3, 5 and 18.

3.2 INSPECTION OBJECTIVE

The objective of an inspection is to obtain the necessary information to assess the structural condition of the structure to determine whether it meets current design and future performance criteria. However, the primary reason for conducting an inspection is the structural safety of the structure. Although the nature of the inspection will determine the extent of information to be provided, the general objective of a condition survey should involve the following:[3,5]

- Identifying and describing all major damage and deterioration
- Identifying the phenomena or materials causing the deterioration
- Determining the extent and rate of deterioration
- Determining the structure's performance characteristics under future service conditions
- Documenting the types and extent of marine growth, water depths, water visibility, tidal range, and water currents which will help plan future inspections
- Determining conformance with contract documents and verifying as-built conditions
- Identifying any potential problems which may occur with mobilization of equipment, personnel, and materials needed to make repairs
- Making recommendations for suitable methods of repair and maintenance
- Obtaining and developing data needed for making cost estimates of these repairs and maintenance
- Recommending the types and frequencies of future inspections

3.3 LEVELS OF INSPECTION

Underwater inspection of marine structures can be grouped into three basic types or levels. They are differentiated by the amount of preparation work required and the means by which the work is to be performed as described below. The level of inspection to be used for a particular inspection must be chosen early in the planning phase.[3,5,6] Table 3.1, which was developed by the U.S. Navy, summarizes the general purpose of each inspection and the type of damage that each level will detect.

TABLE 3.1 - CAPABILITY OF EACH LEVEL OF INSPECTION FOR DETECTING DAMAGE TO MARINE STRUCTURES[5]

Level	Purpose	Detectable Defects		
		Steel	Concrete	Wood
I	General visual to confirm as-built condition and detect severe damage	Extensive corrosion Severe mechanical damage	Major spalling and cracking	Major losses of wood due to marine borers Broken piles Severe abrasion
II	Detect surface detects normally obscured by marine growth	Moderate mechanical damage Major pitting	Surface cracking and crumbling Rust straining Exposed rebar	External pile diameter reduction due to marine borers Splintered piling Loss of bolts and fasteners Early borer and insect infestation
III	Detect hidden and beginning damage	Reduced thickness of material	Location of rebar Beginning corrosion of rebar Change in material strength	Internal damage due to marine borers (internal voids) Decrease in material strength

3.3.1 LEVEL I - GENERAL VISUAL INSPECTION

This type of inspection is the most rapid of all three because it does not require cleaning of the element being inspected. The various purposes of a Level I inspection include: to confirm as-built conditions; provide initial information for developing an inspection program; and detect obvious damage or deterioration caused by overstress, impact, corrosion, or biological attack.

3.3.2 LEVEL II - CLOSE-UP VISUAL INSPECTION

This type of inspection requires cleaning of the concrete surface either before or during the inspection. This level is needed for detecting and identifying surface damage which may be hidden by marine growth. A limited amount of information can be obtained to enable a preliminary assessment of the load carrying capacity of the structure or element of the structure. Since cleaning is time consuming, it is usually done to critical areas of the structure. The amount and thoroughness of cleaning is dictated by the amount of information needed to make a general assessment of the structure.

3.3.3 LEVEL III - HIGHLY DETAILED INSPECTION

A highly detailed inspection is primarily conducted to detect damage which is hidden or damage which is about to occur, and to determine material homogeneity. This type of inspection will usually require prior cleaning. This level often involves the use of Nondestructive Testing (NDT) and sometimes Destructive Testing (DT) techniques. The NDT techniques are usually performed at critical structural areas which may be suspect or are representative of the underwater portion of the structure. Destructive or partially destructive testing, such as coring or physical material sampling, is usually performed to obtain specimens for laboratory testing. Generally, the equipment and test procedures will be more sophisticated, and should be performed by qualified engineer divers or testing personnel.

3.3.4 FACTORS INFLUENCING PRODUCTIVITY

The time, equipment, and effort required to perform the three different levels of inspection are considerably different. The time required for each level depends on environmental factors such as visibility, water depth and water currents, water temperature, wave action, amount of marine growth, and the skill and experience of the inspector/diver.[5] A guide was developed by the U.S.

Navy for estimating the time required to conduct Level I and Level II inspections and is provided in Table 3.2.

TABLE 3.2 - PRODUCTION RATE FOR SURFACE AND UNDERWATER INSPECTION OF STRUCTURAL ELEMENTS[5]

Structural Element	Inspection Time Per Structural Element (minutes)			
	Level I		Level II	
	Surface	U/W	Surface	U/W
300 mm steel H-pile	2	5	15	30
300 mm wide strip of steel sheet pile	1	3	8	15
300 mm square concrete pile	2	4	12	25
300 mm wide strip of concrete sheet pile	1	3	8	15
300 mm diameter timber pile	2	4	10	20
300 mm wide strip of timber sheet pile	1	3	7	15

The information in the Table is based on a water depth of 9 to 12 m (30 to 40 ft.); visibility of 1.2 to 1.8 m (4 to 6 ft.); warm, calm water; moderate marine growth about 50 mm (2 in.) thick; and an experienced engineer diver supervised by an engineer at the surface. Inspection times for Level II assume that approximately 1 m (3 ft.) of the element in the splash zone, 0.3 m (1 ft.) at mid-depth, and 0.3 m at the bottom, will be cleaned of marine growth using the most efficient method.[5] Since Level III inspections depend on the extent of existing damage and can vary significantly for structure to structure, estimates of time are not provided in the Table.

3.4 UNDERWATER DIVING TECHNOLOGY

Various methods are presently in use or currently under development for the underwater inspection of concrete structures. In the last 15 years, significant technological advances have been made by the offshore oil industry. Various individual skills, equipment, and techniques are used in underwater inspections and can be grouped into three basic categories: manned diving missions; remotely operated vehicles (ROVs); and manned submersibles[2]. These are described below.

3.4.1 MANNED DIVING MISSIONS

Manned diving is the most common method used for performing underwater inspections. The breathing gas is provided either by self-contained underwater breathing apparatus (SCUBA) or by an umbilical hose which extends from the surface (surface-supplied/tended air or mixed-gas)[1]. These are the two methods of diving operations most suited for underwater inspection of relatively shallow marine substructures. The main advantage of using divers is that it is a versatile system which in most cases relatively inexpensive. On the other hand, this diving mode is limited by time and dive depth. In addition, since the diver's sense of perception is quite different under water than in air, his observations will be more susceptible to error[3]. Appendix C provides some general characteristics of these types of diving modes.

3.4.1.1 SCUBA DIVING

In SCUBA diving operations, the diver's breathing tanks are typically mounted on his back. The SCUBA diver has the highest degree of movement than in all the other types of diving methods because he is not connected to the surface or an umbilical cable. This diving method is highly mobile and is especially suited for performing short duration dives[1].

The disadvantages of SCUBA diving are: depth limitation, limited air supply, and difficulty in communication with topside personnel. In SCUBA diving the maximum sustained depth at which a diver can work is about 18 m (60 ft.). An experienced diver can dive to depths of 37 m (120 ft.) for short periods of time without experiencing any difficulty. Air requirements will be different for each diver. Normally, a 2 m^3 (72 ft^3) tank is sufficient to allow a SCUBA diver to work at a depth of 9 m (30 ft.) for approximately one hour. As a rule, the amount of time which a diver can remain submerged will decrease with increasing water depth or level of exertion[1].

3.4.1.2 SURFACE-SUPPLIED/TENDED AIR DIVING

In this method of diving, the diver breaths the air or mixed-gas through an umbilical hose supplied from the surface. The breathing medium is forced through the hose by a surface mounted compressor. The diver is also attached to a communication cable, a lifeline, and a pneumofathometer. The diver can use either a hard hat with a dry suit or a face sealing mask with a wet, dry, or hot-water suit[1].

The main disadvantage of surface-supplied/tended air diving is the significant decrease in diver productivity. The diver is considerably less mobile than the SCUBA diver due to the extra weight and diving gear he must carry. Another disadvantage is this diving method needs a considerable amount of additional diving equipment. However, the main advantage with this diving mode is that the diver is in continuous contact with the surface personnel.[1]

3.4.1.3 COMPARISON OF EQUIPMENT REQUIREMENTS

Lamberton et al.[1] have made a comparison between SCUBA diving and surface-supplied/tended air diving and is summarized below. A typical SCUBA diving mission will require the following equipment:

- "Van or truck to transport the gear
- Boat, motor and trailer
- Anchors, mooring line, outboard ladders, and life jackets
- SCUBA tanks, wet suit, fins, weights, masks, regulator, etc.
- Dive flag
- Chipping hammers, picks, pry bars, probing rods, and scrapping tools;
- Underwater lights
- Writing boards, drafting equipment, and underwater slates for recording data;
- Underwater still or video camera
- Rulers, tapes, calipers, or other measuring devices"

For a similar inspection performed by surface-supplied/tended air diving, the following additional equipment is required:

- "Larger vehicle to transport the gear
- Larger boat to support diving operation
- Diving compressor and receiver tank
- Diver umbilical with air hose, communication cable, lifeline, and pneumofathometer;
- Surface-supply head gear
- Diver's radio"

3.4.1.4 SAFETY HAZARDS

In both diving methods, the diver is subjected to the hazards and conditions of the underwater environment which directly affect his performance and safety. In order to avoid serious accidents or injury, the diver must have a clear understanding of these factors and must be able to recognize and handle them[2].

The various hazards and accidents to which a diver may be subjected to are listed below[1]. A detailed explanation of the causes, effects, and treatments of these hazards and accidents is provided in Reference 7.

- Decompression sickness or nitrogen narcosis (the bends)
- Oxygen poisoning
- Bleeding
- Overexertion and exhaustion
- Hypothermia
- Squeeze
- Gas expansion
- Blowup
- Loss of communication
- Fouling
- Polluted water
- Noxious air
- Tides and currents
- Marine traffic
- Marine life
- Floating debris

3.4.1.5 DIVE DEPTHS AND DURATION

A number of organizations have developed dive tables to help divers coordinate dive depth with duration so they can minimize the possibility of their developing decompression sickness. The tables contain time limits for a dive to a given depth. For instance, the U.S. Navy Standard Diving Tables indicate that a healthy 22 year old Navy diver can stay at: a 9 m (30 ft.) depth of seawater for an unlimited time; approximately one hour at 18 m (60 ft.); and 30 minutes at 27 m (90 ft.).[2]

In deeper dives, these limits are significantly shorter. In these cases, inspections are usually performed by more than one diver in succession, or by using an on-site recompression chamber (saturation and nonsaturation diving). The recompression chamber is an air chamber which gradually adjusts the diver's body to the pressure at which he will be working. Depending on the depth of the dive, the adjustment period can vary from a few hours to several days. For dives exceeding 40 m (130 ft.), a specially formulated mixed breathing gas is usually used to avoid the potentially hazardous effects related to nitrogen absorption by the body.[2]

The diving industry has recently developed a rigid-shell "one atmosphere suit" fitted with a set of pincers for hand actuators. The suit permits the diver to work in an environment of one atmosphere (ambient air pressure), which eliminates the possibility of the diver developing decompression sickness. However, the suit is expensive, difficult to work with, and requires special surface support.[2]

3.4.2 ROVs and ROBOTICS

Remotely operated vehicles (ROVs) are being used more often for underwater inspections. They are similar to robots and are used extensively for inspecting deep structures such as offshore pipelines, deep bridge foundations, and hydraulic structures. They are usually connected to a support vessel or to the surface by a flexible communication cable. The vehicle is maneuvered by ballasting and propulsion equipment, and is equipped with video and still cameras mounted on the frame. Some ROVs are equipped with mechanical arms (manipulators) which can operate equipment or retrieve physical samples. The vehicle is remotely controlled from the surface without the use of divers.[2]

Some ROVs, such as MANTIS owned by International Underwater Contractors (IUC), can be operated by a pilot under a one atmosphere condition. There are three modes of operation: as a surface controlled ROV system; with a pilot as an observer in partial control assisted by a surface operator; or with a pilot in full control.

3.4.3 ROV TYPES

ROVs can range from small, relatively inexpensive systems to highly capable but expensive systems. The particular ROV system used for a project depends on the nature and the depth of the underwater inspection being conducted.

According to a report by the U.S. Army Corps of Engineers,[3] there are towed vehicles, bottom crawlers, self-propelled vehicles and vehicles remotely controlled from the surface. There are six basic types of ROVs and these are briefly summarized below. The requirements of a specific diving operation will dictate the most efficient, cost effective, and safest system to be employed. Examples of some commercially available ROVs are provided in Appendix D.

3.4.3.1 TETHERED/FREE-SWIMMING

This vehicle operates in midwater, is equipped with closed-circuit television (CCTV) cameras, and can maneuver in three dimensions. Most of these vehicles receive their power from a support platform, but many are self-powered by batteries carried on board. These vehicles can operate in water depths ranging from 30 m (100 ft.) to 3050 m (10,000 ft.). Vehicle dimensions range from "basketball size to that of a small automobile" and weighs approximately from 32 kg (70 lb) to 5455 kg (12,000 lb) in air.[3]

3.4.3.2 TOWED

These vehicles are propelled and powered by a cable connected to a surface ship. Real-time or slow-scan CCTV and photographic cameras are typically carried on board. Two types of towed vehicles are described below:[3]

(a) Midwater These types of vehicles operate in midwater, but can also make contact with the bottom periodically. Maneuverability in the horizontal direction is controlled by the ship's heading, and the vertical direction is controlled by a reeling cable. These vehicles are designed for long range, long duration dives and can operate in water depths of 6100 m (20,000 ft.).

The Remote Underwater Manipulator (RUM III) is an example of this type of vehicle which usually operates in a towed mode, but can also operate in a bottom-crawling mode to retrieve samples and perform detailed work. Another example is the Towed Unmanned Submersible (TUMS) which operates in a towed mode and can be used to perform detailed investigations as a tethered, free-swimming vehicle by using on board thrusters.

(b) Bottom - and structurally - reliant. These vehicles are towed in contact with the bottom of the sea (bottom-reliant) or are structurally supported by a pipeline (structurally reliant). They are unique in that they are designed for a specific purpose. They are very large structures and are

usually used for cable or pipeline burial.

3.4.3.3 BOTTOM-RELIANT

This type of ROV is powered and controlled by the surface support ship and is equipped with CCTV cameras to monitor the work in progress. The vehicle is propelled by wheels or tracks which are in contact with the seabed. These vehicles are specifically built to perform tasks such as pipeline/cable trenching, bottom excavation and backfilling, maintenance, inspection, soil investigation, or nodule collection.[3]

A good example of such a vehicle is that developed by the Dutch for underwater inspection work at the Eastern Scheldt Storm Surge Barrier. The ROV (PORTUNUS) is a 6 m (20 ft.) long by 4 m (13 ft.) wide by 2 m (6-½ ft.) high, tracked, bottom-crawling vehicle which can perform inspections in turbid water. It can travel at velocities of up to 2.5 m/s (8.2 ft/s) at depths from 15 to 45 m (49 to 148 ft.). The vehicle carries six television cameras, four high frequency transducers, and sonar.[8] This vehicle can be used for inspection of stilling basins.

3.4.3.4 STRUCTURALLY RELIANT

These vehicles are similarly powered and controlled from the surface ship and are propelled by wheels, tracks, or push-pull rams which are connected to a structure. Some of these vehicles are capable of operating in midwater and can travel to and from the structure. Most vehicles are equipped with CCTV, and are specifically designed for pipeline trenching, oil tank sounding, and ship's hull cleaning and inspection.[3]

3.4.3.5 UNTETHERED

These are self-powered vehicles which are not physically connected to the surface vessel. They can maneuver in three dimensions and operate within a preprogrammed schedule. In some cases, their direction and altitude can be modified by commands given from the surface by an acoustic link. These vehicles can operate in depths ranging from 30 m (100 ft.) to 6,100 m (20,000 ft.) and can dive for four to six hour durations.[3] An example of this type of vehicle has been developed by the Dutch to inspect the deep foundations of the precast piers for the Eastern Scheldt Surge Barrier project. The body of the vehicle (TRIGLA) is tubular in shape and measures 900 mm (36 in.) long by 42 mm (1-⅝ in.) in diameter. It is self-propelled, free-floating, and is equipped with lights, cameras, and pressure sensors.[8]

3.4.3.6 HYBRID

Hybrid vehicles are a combination of ROVs that are remotely controlled from the surface or support vessel, and directly controlled by the diver or pilot. These vehicles are a recent addition to underwater work and can perform a variety of tasks, such as pipeline trenching, diver assistance, structure inspection and maintenance, pipeline anchoring, and cable burial.[3]

Further details regarding the structural aspects of the vehicles, tools, sensors, personnel, supporting systems, applications, and operational and navigational considerations, etc., are provided in Reference 9.

3.4.4 ADVANTAGES OF USING ROV SYSTEMS

The advantage of using ROV systems is that they can be used in environments which are considered to be unacceptable for diver safety. They can also be used for very deep and long duration diving operations. In addition, these systems can continuously and repeatedly conduct inspections without performance degradation or concern with diver decompression.[3]

3.4.5 DISADVANTAGES OF USING ROV SYSTEMS

According to Popovics and MacDonald,[3] the disadvantages of using ROV systems are:

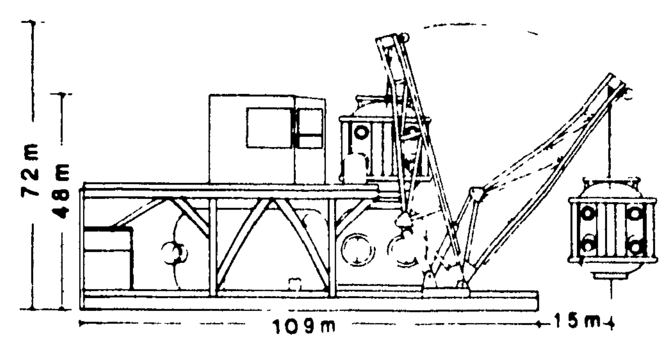
- They are very expensive to use
- They are less flexible and less reliable than using divers
- ROV systems usually require greater maintenance than diver systems
- Video cameras on ROVs may provide distorted views of the extent of deterioration or damage

3.4.6 MANNED SUBMERSIBLES

Manned submersibles or minisubmarines are used to perform deep underwater inspections of offshore oil platforms and salvage operations and have limited use with nearshore marine structures. They are similar to ROVs except that they are larger, more cumbersome and require significant surface support equipment.[2] They are controlled by a pilot inside the vehicle and can follow a preprogrammed schedule via a highly sophisticated acoustic transponder navigation system. A manned submersible is typically equipped with viewports for forward and downward viewing, CCTV cameras, video tape recorders, still cameras, underwater telephone for diver communication, mechanical manipulators, navigation sonar, and acoustic tracking systems. Examples of manned submersibles are provided in Appendix E.

3.4.7 DIVING BELLS

Diving bell systems can be rapidly mobilized to perform deep diving services at low cost. The skid mounted "bounce dive" system consists of a double-lock deck decompression chamber, transfer-under-pressure locks, and a closed-bottom diver work bell. A typical closed-bottom diving bell system is illustrated in Figures 3.1 and 3.2. A closed-bottom bell diving system has several advantages over an open bell system, such as: it can bring an engineer down as an observer to get a first hand look at the work area, it provides more time for inspection repairs or material retrievals, it can surface rapidly without the need for making decompression stops, and is safer for launch and recovery maneuvers in rough waters. The work bell mates with the main deck decompression chamber from above using a standard transfer-under-pressure hydraulic mating clamp.



System Width 4 1 m

FIGURE 3.1 - DEPLOYMENT OF A DIVING BELL
(INTERNATIONAL UNDERWATER CONTRACTORS, INC.)

The diving bell can accommodate two divers and is equipped with an enclosed control console cabin, standard life support equipment, internal/external depth gauges, temperature monitoring equipment, internal/external lighting and several communication systems. The bell is also equipped with several auxiliary emergency life support and communication systems. The deck decompression chamber consists of sleeping quarters, an environmental control unit, and medical lock. Typical examples of deep diving or saturation systems are included in Appendix F.

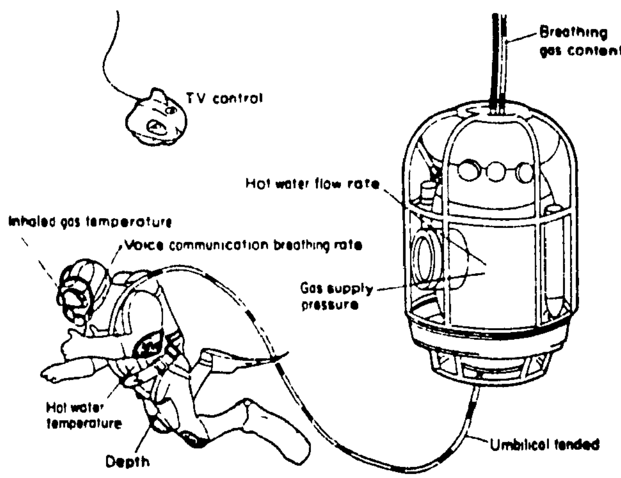


FIGURE 3.2 - SCHEMATIC DIAGRAM OF A DIVER/BELL SYSTEM[27]

3.4.8 SUPPORT VESSELS

Support vessels are an important part of the inspection equipment and can greatly influence the quality of the inspection and repair work. Highly sophisticated projects often require some type of support vessel. Typically used support vessels are described in Reference 8 and are summarized below.

3.4.8.1 FLOATING PONTOONS AND BARGES

This is the basic support vessel which provides a working platform. These vessels must be stabilized during rough waters, especially during placement of tremie or pumped concrete (Chapter 5). The vessel is stabilized by several anchor lines, a good metacentric height, a wide area at the waterline, or buoyancy tanks.

3.4.8.2 JACK-UP BARGES

These barges are used for shallow water work and can provide several advantages for underwater repair and inspection work: they provide stability during surface preparation, repair, and inspection operations; they can be used for guiding and controlling surface cleaning equipment, and monitoring equipment; and are suitable for placing prefabricated concrete panels.

3.4.8.3 SHIPS

Large ships have been designed to support several diving systems. The ship carries communication and navigation equipment and other gear which are needed to support diver activities. A typical ship has living quarters which can accommodate a large crew for as long as 30 days at sea. The ship is equipped with a machine shop for repair and building of equipment. The ship can accomplish several types of missions including inspection and survey, search and recovery, diving support, oceanographic, geophysical, and bathymetric surveys, medium and long-range research programs, and installation and/or retrieva of sensor packages and systems. More details on a typical support vessel are provided in Appendix G.

3.5 INSPECTION PERSONNEL

The evaluation of a concrete structure under water is more complex than that of a structure above water and must be carried out by a team of experienced divers and engineers. The reason for this is because although engineers have the necessary technical background for making a reliable evaluation, they are not trained divers or ROV operators and vice versa.[3]

The diver performing the underwater inspection must be well informed of the proper use and care of diving equipment, safety requirements, communication techniques, and diving operations. There are agencies, such as the American Association of State Highway and Transportation Officials (AASHTO) and the Federal Highway Administration (FHWA) in the United States, that require divers to be certified and well-trained in structural inspection.[1] Organizations that certify divers include:

- Professional Association of Diving Instructors (PADI)

- National Association of Underwater Instructors (NAUI)
- American Diving Contractors Association (ADCA)

Engineer divers are being used more often to conduct underwater inspection of marine structures. Many agencies in the United States, such as the U.S. Navy, require that inspections be conducted by professional engineering divers. Engineer divers are capable of evaluating the extent and effects of deterioration on the structure.[4] A report by Buehring[10] states that:

> "The engineer trained to dive can become more familiar with the underwater environment and can also obtain pertinent information first hand ..." and goes on to state, "... The professional engineer who dives is likely to make more judgmental decisions than the commercial diver. Also because the engineer's authority exceeds that of the commercial diver, his observations and conclusions are more likely to be accepted at face value...".

The majority of agencies use commercial divers supervised by professional engineers. The engineer possesses the necessary technical expertise and can provide guidance to the diver or divers during the inspection, permitting a faster, efficient, and more accurate inspection.[3] The inspection team should have a good background in civil or mechanical engineering, as well as in design, construction, maintenance, and operation of underwater concrete structures.[11]

3.6 FREQUENCY OF INSPECTION

There are no firm guidelines on the frequency of inspection of concrete structures. Public works are usually inspected regularly at short intervals.[12] Some agencies in the United States conduct underwater inspections of bridge substructures every two years, while others schedule inspections every five years. Others inspect their bridges infrequently or only after indications of underwater problems. In some cases, underwater inspections are performed immediately after each major storm where scour problems are anticipated.[1]

The frequency of inspection will depend on the expected rate of deterioration and damage to the facility or structure. The U.S. Navy recommends that piles above the waterline, including the splash and tidal zones, should be inspected annually. The underwater portions should be inspected every six years starting from the splash/tidal zone and proceeding downward. The level and frequency of inspection should be adjusted according to the extent of the observed deterioration.[5] The ACI Committee 357[13] recommends that concrete offshore structures should be surveyed annually for damage or deterioration and the inspection findings should be carefully reviewed every five years.

Frequent and well organized inspections are an effective method of keeping maintenance costs to a minimum. They are also useful for obtaining base-line data on a specific type of structure. The rate of deterioration can be monitored and decisions can be made when repair becomes economical.[12] The International Prestressing Federation (FIP) Guide to Good Practice[14] includes a table which provides guidance on the intervals that might be appropriate for "routine" and "extended" inspections. A routine inspection is visual and does not require special equipment or access. Extended inspection, on the other hand, is a more detailed investigation which requires special access and remote viewing techniques. Intervals for three "classes" of structure and for three environmental and loading conditions are illustrated in Table 3.3.

TABLE 3.3 - RECOMMENDED INSPECTION INTERVALS FOR VARIOUS ENVIRONMENTAL AND LOADING CONDITIONS[12]

Environmental and loading conditions	Classes of structure					
	1		2		3	
	Routine	Extended	Routine	Extended	Routine	Extended
Very severe	2*	2	6*	6	10*	10
Severe	6*	6	10*	10	10	--
Normal	10*	10	10	--	Only superficial inspections	

*Midway between extended inspections

The intervals shown in the table should be regarded as absolute maxima and in most cases inspections should be performed more frequently. Inspection frequency is based on engineering and economic judgement, and should be established to suit the structure with regards to use, siting, construction, and design.[12]

3.7 INSPECTION PROCESS

A report by Dr. G. Watson[15] on underwater inspection of offshore structures recognizes three phases to any inspection process: defining the requirement implementing the inspection, and assessing the inspection results. He further states that:

"There is a close interaction between the requirement phase and the implementation phase, in that the implementation cannot generally be defined until the requirement detail is available. The assessment phase interacts with both the requirement and implementation, by providing feedback via the results themselves, and the degree of attainment of the defined requirement by the prescribed implementation."

There is no set procedure for conducting an underwater inspection since each will be different and will require a process of data collection. However, it is important to define how a structure will be assessed as this can have a significant effect on visual inspection in the implementation phase. Although inspections vary widely in type, scope, objective, and complexity, most require the basic activity shown in Table 3.4. The order in which these activities are listed is not necessarily the sequence in which these activities will take place. The activities of the inspection process outlined in the table are described in more detail in the following sections.

TABLE 3.4 - COMMON STEPS IN THE INSPECTION PROCESS*

Collection of background information; preliminary document review
Initial reconnaissance site visit; eyewitness interviews
Formulation of investigative plan; formation of project team
Comprehensive collection of documents; document review
Site investigation; sample collection
Theoretical analyses
Laboratory analyses
Development of failure hypotheses, analysis of data, synthesis of information, and formation of conclusions
Report writing

*Adapted from Reference 25

3.7.1 AVAILABLE INFORMATION

The initial phase of any inspection program should be the collection and review of all available information on the structure.[4] The planning and implementation of an inspection cannot be properly and efficiently done without consideration of information related to the design, construction, operation, and maintenance of the structure or facility.[3] Review of the available information often provides an indication to what caused or might be causing the defect. This information can help save considerable time in the field and result in a more accurate inspection. Bell[25] has developed a list of project specific documents, and their sources, often used in an inspection process. These are shown in Tables 3.5 and 3.6, respectively.

TABLE 3.5 - PROJECT SPECIFIC DOCUMENTS USED IN AN INSPECTION*

- Contract drawings (including all revised issues)
  - Structural (including progress prints)
  - Architectural (including progress prints)
- Contract specifications
  - Technical sections of interest
  - General conditions
  - Special conditions
- Contracts
  - Owner/Architect
  - Architect/Structural Engineer
- Contract revisions
  - Addenda
  - Field directives
  - Change orders
- Shop drawings and other submissions
  - Reinforcing steel
  - Product data
- As-built drawings
- Field and shop reports (including construction photos)
  - Concrete inspection laboratory (reinforcing steel, formwork, concrete)
  - Concrete mix designs
  - Construction supervisor's daily log
  - Owner's or developer's field inspectors
- Materials strength reports or certification
  - Concrete compressive strength
  - Steel mill certificates
  - Results of special load tests
- Project correspondence[a]
  - Owner/consultant
  - Owner/contractor
  - In-house memoranda
  - Records of meeting notes
  - Records of telephone conversations
- Consultant reports
  - Feasibility studies
  - Progress reports
  - Soils consultant reports (including boring logs)
- Calculations
  - Primary structural engineer
  - Reviewing structural engineer
  - Specific subcontractor's engineers
- Maintenance and modification records

[a]The scope will vary depending on the inspector's assignment
*Adapted from Reference 25

Geotechnical investigations can also provide important information in the inspection and evaluation of marine structures, especially if the structure has undergone any settlement or movement. If there is evidence of settlement or movement, a geotechnical investigation should be performed to determine if subsurface conditions will affect the structure in the future.[4] A geotechnical investigation can also provide an indication whether further settlement will occur. This may have a direct impact on the repair solution.

TABLE 3.6 - SOURCES OF PROJECT DOCUMENTS[25]

Architects and engineers involved in original design, modification, or repair of facility
Past and present owners
Past and present tenants
General contractor and/or construction manager for original construction, modification, or repair of facility
Subcontractors involved in original construction, modification, or repair of facility
Developer of facility
Construction mortgagee of facility
Materials or systems suppliers for original construction, modification or repair of facility
Previous or other current investigators
Building department
Testing agency involved in original construction, modification, or repair of facility

3.7.2 PLANNING THE INSPECTION

Once all of the available information on the structure has been obtained and reviewed, an inspection plan should be developed. Since surveys are both expensive and time consuming, the inspection must be carefully planned and implemented to obtain the greatest amount of information in the shortest time possible. The purpose of the inspection and the desired technical results will determine the specific information that is needed, the level of detail required, and the format of the final report. Site logistics will often decide the form of diving mode to be used. For instance, shallow structures are typically inspected by traditional diving methods while deeper structures may be inspected using ROVs or minisubmarines.[2]

3.7.2.1 ENVIRONMENTAL FACTORS

Environmental conditions that may hamper the efficiency of the inspection should be considered. These may include atmospheric temperature range, water temperature range, tidal range, water depths, water visibility, and currents. Any condition which will directly affect the time required to perform an inspection, such as the extent of marine growth, ice, or seasonal flooding should also be considered.[5]

3.7.2.2 INSPECTION AREAS

The proper and adequate selection of areas to be inspected is crucial to the effectiveness of the condition survey. A sufficient number of inspection areas must be selected to provide representative information on the underwater portion of the structure. Thus, it is important to know the areas of the structure which will be subjected to maximum stress. It is also very helpful to know the processes involved with concrete deterioration. A useful flow chart for developing an effective inspection plan for evaluating underwater concrete structures was developed by Brackett et al.[28] and is shown in Figure 3.3. Another report by Brackett,[16] sponsored by the U.S. Naval Civil Engineering Laboratory (NCEL), provides a good guide on sampling criteria for inspection of pile supported wharf structures.

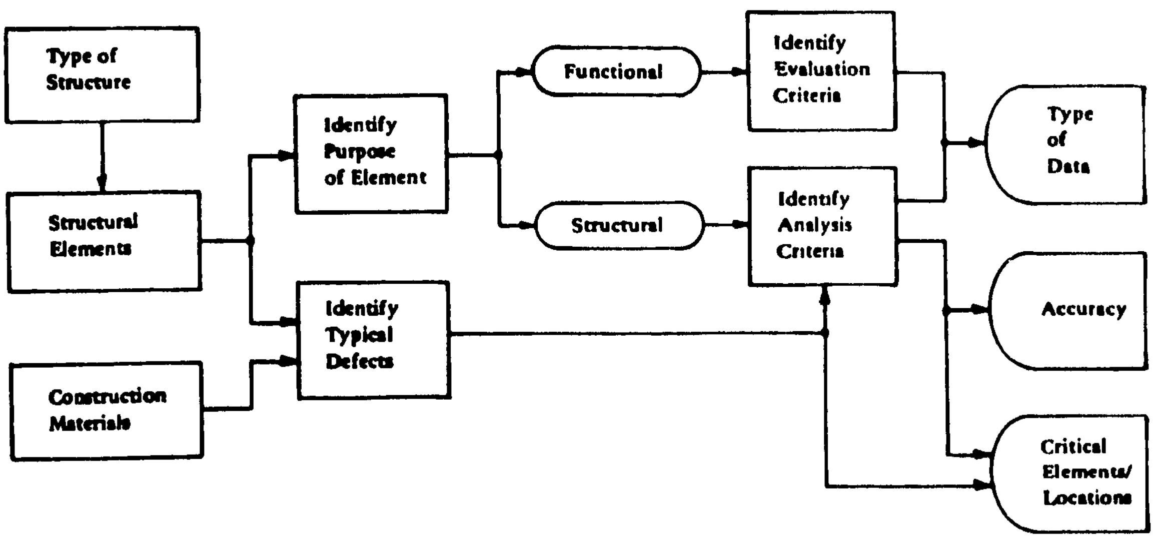


FIGURE 3.3 - FLOW CHART FOR DEFINING INSPECTION DATA REQUIREMENTS FOR UNDERWATER STRUCTURES[28]

3.7.2.3 MANPOWER AND EQUIPMENT

Availability of manpower and equipment are key factors that should be considered when planning an underwater inspection. The skill of the diver or ROV operator as an inspector is also an important factor that should be considered. Most divers (or pilots of ROVs and minisubmarines) are well trained in the use of diving techniques but do not have the technical background to conduct structural investigations. Some inspections can be performed by using relatively unskilled divers. However, if the inspection calls for "interpretive skills" an engineer diver will usually provide the desired results.[2] The inspection plan should also indicate the type of inspection equipment to be used for each inspection task.[3]

3.7.2.4 SCHEDULING

As previously stated, periodic inspections are essential to the implementation of an effective maintenance program. Underwater inspections should also be conducted during and at the end of the construction phase of the structure to provide baseline data for future inspections. Underwater inspections should be scheduled during favorable conditions such as periods during low water, low pollution levels, minimum ice, no flooding, or good underwater visibility.[3]

3.7.2.5 RECORDING AND DOCUMENTATION

The documentation and form of the report required is an important consideration when planning an underwater inspection. For example, the documentation that is needed for a research project will not be the same as that required for repair or damage assessment. In addition, the level of detail needed for a facility purchase baseline survey is quite different from that required by a damage repair document. If the diving report will be issued as a repair construction document, the damage noted during the inspection should be properly "quantified and qualified."[2] Photographs or video cassette recordings should be used whenever possible as they can provide a more detailed and accurate description of inspection results. A more detailed discussion on methods of recording and documentation is provided later in this chapter.

3.8 INSPECTION PROCEDURE

Inspection of marine structures should generally be conducted in two parts: the first part is an above-water survey, and the second part is an inspection of the underwater portions of the structure.[4] It is important to coordinate the two surveys so that no area of the structure within the tidal zone is left uninspected due to tide elevation changes. The underwater inspection is usually performed along the same guidelines as for the above-water inspection. Therefore, this section discusses the inspection of the underwater portion of concrete structures only.

The underwater survey should also be conducted in two phases whenever possible.[4] The first phase involves a quick visual (Level I) inspection to detect visible damage or major deterioration which could be used to make a preliminary assessment of the condition of the underwater portion of the structure. Information obtained during this phase should provide guidance for developing and performing a final (Level II or Level III) inspection. The second phase, or final inspection, is a complete inspection of the structure from the splash zone down to the mudline as shown in Figure 3.4.

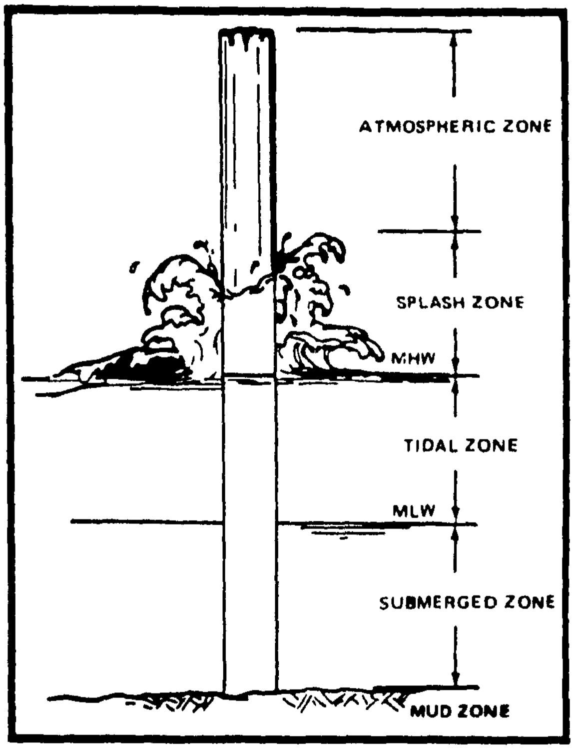


FIGURE 3.4 - INSPECTION RANGES ON PILING[5]

Since the tidal zone is the area where most of the mechanical damage or deterioration usually occurs, it should be inspected at several locations, with most of the inspection points concentrated immediately below the low water zone. Inspection and documentation of the structure in the submerged zone should be spaced uniformly to provide efficient data gathering.

At or near the mudline it is recommended to increase inspection and documentation to evaluate the potential for scour and abrasion type damage.[4] A typical underwater inspection procedure is outlined below:[3,5]

(a) Inspect the structure starting at the splash/tidal zone.

(b) Remove all marine growth from a section approximately 450 to 600 mm (18 to 24 in.) in length.

(c) Visually inspect this area for cracks with rust stains, spalling, and exposed reinforcing steel. The location, length and width of the defects should be recorded.

(d) Sound the cleaned area with a hammer to detect loose concrete or hollow areas in the structure.

(e) While proceeding downward, visually inspect the structure where marine growth is minimal, and sound with a hammer.

(f) At the bottom, record (on a Plexiglass slate) the water depth along with any observations of damage. Carefully inspect the base of mass concrete structures such as retaining walls and footings to detect any scour damage.

(g) After returning to the surface, record all observations and measurements immediately into the inspection log. If voice communication between the diver and topside personnel is available, the data can be directly relayed to the surface as the inspection progresses.

More detailed procedures for inspecting underwater bridge substructures are provided in Appendix H.

3.9 SURFACE CLEANING EQUIPMENT

To facilitate inspection, so that a thorough and accurate visual examination of the structure can be conducted, some form of surface cleaning will almost always be required. The extent of the cleaning is dependent on the amount of marine growth present, and the type of inspection being made. For instance, routine inspections generally require only minor cleaning, whereas detailed inspections require thorough cleaning of certain structural elements. According to Lamberton et al.,[1] "indiscriminate cleaning should be avoided" because it is not only time consuming and expensive, it can also cause further damage to weakened areas.

There are four basic categories of underwater surface cleaning tools and each category offers a selection of techniques giving several methods. The performance of each type of equipment, such as cleaning rates, depends upon many factors.[5,18] These include:

- The physical and operational characteristics of the cleaning tool
- The operator experience
- The extent and type of marine growth
- Water visibility
- Accessibility of the surface to be cleaned
- Underwater working conditions

Assuming good working conditions, an average experienced diver can achieve the cleaning rates given in Table 3.7. The following sections describe some of the tools listed in the table that are appropriate for cleaning underwater concrete structures.

TABLE 3.7 - MARINE GROWTH REMOVAL FROM CONCRETE SURFACES[5]

Method	Production Rate, m^2/min (ft^2/min)		Safe Work Duration (hr)
	Preliminary Cleaning	Final Cleaning	
Hand Scraper	.02 (0.2)	--	2
Cavitation Pistol	0.13 (1.4)	0.06 (0.6)	2
Reactionless Jet	0.21 (2.3)	0.07 (0.8)	2
Sand injection Jet	0.13 (1.4)	0.04 (0.4)	2
Barnacle Buster	--	0.06 (0.6)	1
Reaction Jet	0.33 (3.6)	0.10 (1.1)	1

3.9.1 HAND TOOLS

The effectiveness of these types of tools depends on diver effort. These tools are not powered, are usually used for light cleaning, and are unlikely to cause any damage to the structure itself. Examples of these tools include scrapers, diver's knife, chipping hammers, probes, wire brushes, chains, and wire ropes. Chains and wire ropes are wrapped around the member (such as a pile) and are then pulled back and forth to remove any fouling.[5]

Hand tools are small, lightweight, highly portable, and are also the least hazardous for underwater use. The main disadvantage is that they are slow and time consuming. The highest cleaning rate that can be achieved is approximately 0.1 m^2 (1 ft^2) per minute. When heavy marine growth is present, cleaning rates are as low as 0.02 to 0.03 m^2/min (0.2 to 0.3 ft^2/min). For this reason, hand tools are not usually used for cleaning large areas or for removing heavy marine growth from concrete structures.[18]

3.9.2 PNEUMATIC AND HYDRAULIC TOOLS

Pneumatic and hydraulic tools are used for cleaning thick marine growth from large areas and are more efficient than hand tools, but can cause damage to the surface of the structure. Hydraulic tools are usually preferred since they are safer and easier to use, and they are not limited by water depth. They also cause less diver fatigue. Examples of these tools include chippers, grinders, needle guns, and rotary brushes. Brush systems are available in various sizes. Brush sizes up to 400 mm (16 in.) in diameter are suitable for cleaning bridge substructures.[1,17]

To effectively remove heavy, calcareous marine growth from concrete surfaces, a rotary cleaning tool, such as the "Whirl Away or Barnacle Buster" is recommended. This tool can be attached to and operated by most standard hydraulic grinders, disc sanders, and polishers. The attachment consists of several hardened steel cutters that rotate in a direction opposite to that of the tool shaft. The Barnacle Buster is the most effective and safest tool for removing marine growth from concrete surfaces because it is easy to use and does not require high-pressure water.[5]

There are several models available ranging from 80 to 170 mm (3-¼ in. to 7 in.) in diameter. The largest diameter tool can clean more than 75 mm (3 in.) of hard shell growth and 150 mm (6 in.) of soft growth at rates of 0.28 to 0.56 m^2 (3 to 6 ft^2) per minute. The smaller models cannot remove heavy fouling effectively and should be used only to remove marine growth less than 50 to 75 mm (2 to 3 in.) thick.[18]

3.9.3 HIGH-PRESSURE WATER JETS

High-pressure water jets are used widely and are very effective for tough cleaning jobs and produce some of the highest cleaning rates. Water jets can be used to displace loose sediment and debris, remove low quality concrete, provide a rough surface for better bonding, and eliminate feathered edges around the perimeter of an eroded area. Water jets can be either used by divers or remotely controlled from the surface.[8]

There are many types of underwater water jet tools available on the market. They all generally consist of a surface pump, a high pressure hose and a gun (Figure 3.5). Water jet cleaning tools generally fall into one of two categories: high-flow devices or low-flow devices.[18] High-flow cleaning systems operate between 27 and 80 MPa (4,000 and 12,000 psi) and between 45 to 97 m^3/min. (12 to 26 gpm). Most high-flow tools are fitted with a retrojet to counter the reaction force generated by the water jet. Low-flow cleaning systems operate at approximately 68 MPa (10,000 psi) and 8 to 20 m^3/min. (2 to 5 gpm). This tool does not develop enough backthrust to require a retrojet.

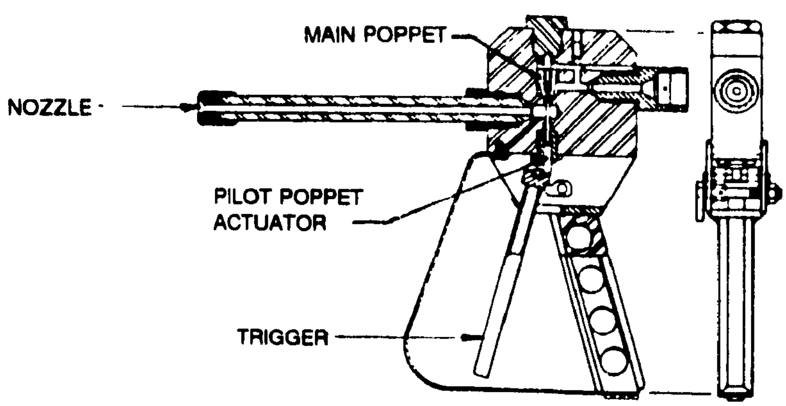


FIGURE 3.5 - WATER JET GUN[18]

Most water jet tools have interchangeable fan and straight-jet nozzles (Figure 3.6).[18] Fan-jet nozzles will clean a wider surface area, while straight-jet nozzles provide a higher cleaning intensity over a very small area. According to research by Parker et al.,[19] the most effective nozzle size in removing marine growth is 0.80 mm (0.031 in.) in diameter. The fan-jet nozzle can clean an area up to ten times faster than typical straight jets. However, the intensity of the fan-jet reduces sharply as the distance from the work surface is increased.[18] The straight-jet nozzle is very effective for cleaning the interior of cracks.

FIGURE 3.6 - FAN AND STRAIGHT-JET NOZZLES[18]

The main disadvantage of the water jet cleaning tool is that it is potentially hazardous and if not properly controlled, an unskilled operator can damage the concrete surface.[17] The high pressure and velocity of the water jet will easily remove human flesh and bone. All personnel should be aware of the hazards involved and should be properly trained before using the device. High-pressure water jets can also produce excessive noise under water which, over time, can be harmful to the diver.[18]

3.9.4 SELF-PROPELLED VEHICLES

Self-propelled cleaning vehicles are used for removing marine growth and corrosion from large underwater surfaces which are readily accessible. Although they have been designed for use on steel surfaces (hulls of ships and oil tankers), they can also be used for cleaning concrete. For instance, the sides of lock walls, faces of dams and stilling basins are areas where these vehicles can be used effectively.[18]

A typical underwater self-propelled vehicle consists of three large rotating brushes and travels on traction wheels. Depending upon conditions, it can clean a path about 1.2 to 1.5 m (4 to 5 ft.) wide and can travel up to 27 m (90 ft.) per minute, giving a cleaning rate of 42 m^2 (450 ft^2) per minute. The vehicle is connected to a surface control console with an umbilical cable and can be either remotely controlled or steered directly by a diver. The control console shows where the vehicle is located with respect to orientation, depth, and the distance travelled.[18]

The disadvantage of using a self-propelled cleaning vehicle is that it can only clean flat surfaces which are unobstructed. Also, due to its size and weight, deployment and recovery of the vehicle requires the use of large handling equipment. These vehicles are also more expensive to operate than most other types of cleaning equipment.[18]

3.10 VISUAL INSPECTION

Visual inspection is the most common method used to inspect underwater concrete structures.[1] It is quick, usually uncomplicated, and nondestructive. It is usually performed to detect severe damage (Level I inspection), and to detect surface damage (Level II inspection).[3] There are three basic methods for gathering visual information underwater and are listed in decreasing order of resolution: diver's eyes, photography and video.[15] In some cases, underwater observations can be made from the surface by an underwater scope. For example, underwater inspection during erosion repair at Chief Joseph Dam was performed through a 10.6 m (35 ft.) long scope equipped with a 150 mm (6 in.) diameter bottom glass and a high-power telescope.[20]

3.10.1 VISUAL INSPECTION TOOLS

There are several hand tools available for performing an underwater visual inspection. Hammers, picks, pry bars, and probing bars are used for performing soundings of the concrete (acoustic ringing). This method can detect voids in the concrete and delamination of the concrete cover. Chipping tools are effective for prodding the surface of the concrete to determine the depth of deterioration.[1,3,5] These methods are economical but qualitative in nature, and should be used only as a guide in evaluating the condition of underwater concrete.

Flashlights for improving visibility are almost always necessary. The use of quartz iodide and thallium iodide lamps have been successfully used by divers to improve visibility underwater.[3] Visibility can also be improved by attaching a clear-water mask to the face plate of the diving helmet.[1]

3.10.2 MEASURING DEVICES

The measurement of physical dimensions provides a partial quantitative measure of the member's strength or degree of deterioration. This method is nondestructive, economical, and requires a minimal amount of time and equipment. However, this method does not provide information on the condition of the remaining concrete.[3] More sophisticated (Level III) techniques for taking internal defect measurements and assessing the condition of the remaining concrete are discussed in Chapter 4. Based on a review of Reference 17, below is a summary of techniques most commonly used by divers to measure defects in underwater concrete structures.

3.10.2.1 LINEAR MEASUREMENTS

(a) Ruler. A ruler is used for measuring crack length, spall length and width. Measurements with a ruler can be taken with an accuracy of plus or minus 0.5 mm (0.02 in.).

(b) Tape Measure. For measurements up to 100 m (328 ft.), tape measures are usually employed. This is not as accurate as the ruler, due to problems with positioning the end and tape sag. Accuracy is plus or minus 5.0 mm (0.2 in.).

(c) Magnetic Tape. This can be used for measurements up to 3 m (9.8 ft.) and is often used to take circular measurements. Measuring accuracy is plus or minus 1.0 mm (0.04 in.).

(d) Scales. These are often made from a special vinyl embossing tape (DYMO) and can be used in conjunction with photography. Accuracy can be plus or minus 5.0 mm (0.2 in.).

(e) Comparator. A comparator is used to measure crack widths and consists of a small hand-held microscope with a scale on the lens closest to the surface being inspected. Crack widths can be measured with an accuracy of about 0.025 mm (0.002 in.) when used in dry conditions.[21] However, the use of this device for measuring crack widths under water may be severely limited if water visibility is poor.

3.10.2.2 CIRCULAR MEASUREMENTS

(a) Calipers. These instruments can be used for taking measurements up to 2 m (6.6 ft.) in diameter. If carefully used, measurements can be made with an accuracy of plus or minus 0.5 mm (0.02 in.).

(b) Special Jigs. Various jigs are available for measuring the ovality of members. Accuracy can be plus or minus 5 mm (0.2 in.).

3.10.2.3 DEFORMATIONS/SPALL DEPTH

(a) Profile gauge. The profile gauge can record a mirror image of the defect with a possible accuracy of plus or minus 0.5 mm (0.2 in.).

(b) Taut Wire. This is used for large spalled or deformed areas. It can also be used to measure the out-of-plumbness of a member. Accuracy depends on the conditions, but is approximately plus or minus 5 mm (0.2 in.).

(c) Casts. These materials are also used to obtain a mirror image of a defect. The most recently developed product for underwater use is produced by BP Chemicals Ltd., and is marketed as "AQUAPRINT". The moulding agent (supplied in a cartridge) is applied to the surface by a pressure dispenser and cures in about 15 minutes. The flexible, but non-stretchable cast is peeled off and taken to the surface for visual analysis.

3.10.3 VISUAL INSPECTION LIMITATIONS

Although visual inspections are quick and relatively inexpensive, there are several limitations which must be considered. Environmental limitations include marine growth, which obscures surface defects unless cleaned, poor visibility in turbid water and strong water currents which ninders the divers capacity to work. Limitations which are related to the diver himself vary from inexperience as an inspector to poor observation and performance resulting from underwater environmental conditions.[3]

3.10.4 TACTILE INSPECTION

Underwater inspections must often be conducted in turbid water which severely reduces visibility. In many cases, visibility is zero. In these cases, a tactile inspection is required in which the diver must use his sensory perception capabilities, such as touch and feel, to detect flaws, damage and deterioration. The task is usually difficult to perform and requires more preparation than when working in clear water. The diver must have a good understanding of the structure by performing an in-depth study of the existing drawings of the structure. During the inspection, good communication between the diver and surface personnel is essential.[1,3] This type of inspection most often requires prior cleaning if the structure is covered with marine growth.[1]

3.11 RECORDING AND DOCUMENTATION

For an inspection to be useful, a clear, concise, and complete record must be established. The inspection information should be documented using standard forms and report formats. Recording devices are necessary to provide a complete and permanent record of the condition of the structure. A Plexiglass slate and a grease pencil is useful for making notes and sketches under water. These notes can be transcribed into the inspection log when the diver returns to the surface.[3,5] Current methods of recording inspection findings include drawings, inspection forms, closed-circuit television (video), flash photography, and photogrammetry.[17] All of these methods are used frequently and are an essential part of the final report.

3.11.1 DRAWINGS

Inspection findings are often recorded on existing drawings of the structure after the diver has completed the inspection. When drawings of the structure are not available, they are drawn from memory by the diver at the end of the inspection. More often, these drawings are developed by using surface data recorders at the time of inspection and are verified by the divers.[17]

3.11.2 INSPECTION FORMS

Recording inspection data can be simplified by using appropriate inspection forms. They are also useful for comparing past, present, and future inspection results. Inspection information should be recorded during the inspection, or as soon as the inspection is completed, and in accordance with generally understood terminology.[3,5] A standard form which may be used for reporting the condition of concrete piles is shown in Figure 3.7. An explanation of the condition ratings used on the form is provided in Figure 3.8.

3.11.3 CLOSED-CIRCUIT TELEVISION (VIDEO)

This method of recording is used extensively for underwater inspections. Video can expedite major underwater inspections and has the advantage of "real-time" display to the surface and "real-time" quality control of the video image, and can be used to monitor diver performance.[3] Video systems can be used as diver-deployed (hand-held or head-mounted), or as remotely operated, mounted on ROVs, or used in piloted minisubmarines. Presently, these systems are available in monochrome (black and white), color, and low light.[17]

PILE INSPECTION RECORD

LOCATION		DATE	DIVERS
PIER NAME/NO		PILE TYPE ☐ BEARING ☐ FENDER ☐ SHEET	PILE MATERIAL ☐ TIMBER ☐ STEEL ☐ REINFORCED CONCRETE
WATER DEPTH	TIME OF DAY	TIDE	DEPTH OF DAMAGE FROM DATUM-GAUGE DEPTH TIDE

BENT NO	PILE NO	NI	PILE CONDITION					TYPE DAMAGE			GAUGE DEPTH DAMAGE	DIMENSIONS OF DAMAGE			COMMENTS
			NO	MN	MD	MJ	SV	MECH	BIO	FUNC		HGT	WIDTH	PENETR	

FIGURE 3.7 - STANDARD PILE INSPECTION REPORT FORM[5]

3.11.3.1 VHS AND U-MATIC

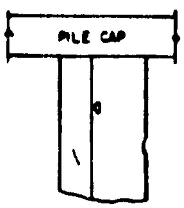
There are two methods of recording video inspections: video home systems (VHS) and U-Matic. The VHS equipment is smaller, more portable, and is more readily available. VHS equipment made from different manufacturers have good compatibility. The main disadvantage is that the quality of the reproduced picture can be poor, and copies of the tapes are always of poorer quality. The U-Matic system uses a wider tape which can provide highly detailed photographs and better quality video tape reproduction.[17]

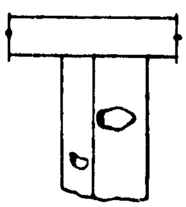
The "VIDICON" is probably the most widely used image sensor for underwater inspections. It is a tube-type device which provides high quality images. It is sensitive to bright light and is insensitive to low light levels. However, the standard VIDICON has been modified to provide better performance in these areas.[3] In some cases, "down-hole" cameras can be used to perform visual inspections from the surface. For instance, in the case of repairs at Chief Joseph Dam in

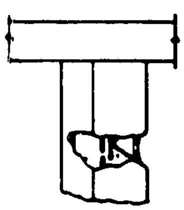
Washington D.C., (U.S.A.), a down-hole camera was used to inspect the vertical monolith joints of the dam.[22] Similarly, a down-hole camera was used for inspecting vertical grout holes during repairs at Big Eddy Dam in Ontario, Canada.[23]

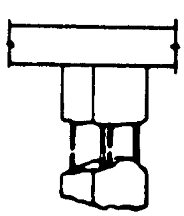
CONCRETE PILES

CONCRETE PILE CONDITION RATING	EXPLANATION
NI	NOT INSPECTED, INACCESSIBLE OR PASSED BY
ND	NO DEFECTS. - fine cracks - good original surface, hard material, sound
MN	MINOR DEFECTS - good original section - minor cracks or pits - surface spalling that exposes course aggregate - small chips or popouts due to impact - slight rust stains - no exposed re-bar - hard material, sound
MD	MODERATE DEFECTS - spalling of concrete - minor corrosion of exposed re-bar - rust stains along re-bar with or without visible cracking - softening of concrete due to chemical attack - surface disintegration to one inch due to weathering or abrasion - reinforcing steel ties exposed - popouts or impact damage
MJ	MAJOR DEFECTS: - loss of concrete (10-15%) - one or two re-bars badly corroded - one or two ties badly corroded - large spalls six inches or more in width or length - deep wide cracks along re-bar
SV	SEVERE DEFECTS: - two or three re-bars completely corroded - no remaining structural strength - significant deformation









NOTE: Explanation of defect should be placed in the comments column.

FIGURE 3.8 - PILE CONDITION RATINGS FOR CONCRETE PILES[5]

The important aspect of using video recordings is that comments can be recorded on the tape as the inspection progresses, either by the diver performing the inspection or by surface personnel viewing the monitor. When the diver is breathing a helium gas mixture, the voice changes pitch and a special voice unscrambler must be used [17] Video tapes should be provided with a title, a brief description of what is on the tape, and the date the inspection was performed. The description should include the name, location, type, and size of the structure being inspected and any other pertinent information.[24]

3.11.3.2 IMAGE ENHANCEMENT

Image enhancement is a common technique used with video recording. Video recordings are electronically enhanced to produce a better quality image. Enhancement is available in color, electronic video enhancement systems, and stereo video systems. The latter is very useful where an assessment of depth is desirable.[15] However, since currently available camera systems can take extremely accurate and highly detailed photographs, the use of image enhancement systems may not be necessary.[17]

3.11.3.3 FIBER OPTICS (ENDOPROBES)

Endoprobes can improve visual observation and are useful for inspecting the inaccessible areas of a structure. The system, which consists of fiber-light guide cables, a light source, and an optical system, provides a 135° field of vision. The lenses are color adjusted so that an accurate photograph can be obtained. The inspection observations made with an endoprobe can be recorded by the use of "Reflex" cameras, "Polaroid" cameras, and television monitors. Size and depth perception can be achieved through the use of graticules.[17]

3.11.4 FLASH PHOTOGRAPHY

Flash photography is used extensively for recording inspection findings. The photographs can be in color or monochrome, and can vary from general stand-off coverage to 100 percent close-up mosaics.[15] Water turbidity and monochromatic marine growth will most often make photographic color distinction difficult. This problem can be reduced by prior cleaning of the structure, fitting a clear-water box to the lens of the camera, or by using proper lighting.[3]

There are three basic ways to use a flash: as a single unit on a stalk as two units on either side of the camera, each on its own stalk and as a ring flash built around the camera lens. The latter usually requires the use of specialized close-up cameras. Back scatter from suspended particles in the water and reflected light from the object are the problems usually encountered when using the single or double unit flashes.[17] This problem can usually be minimized by placing the flash at an angle as shown in Figure 3.9.

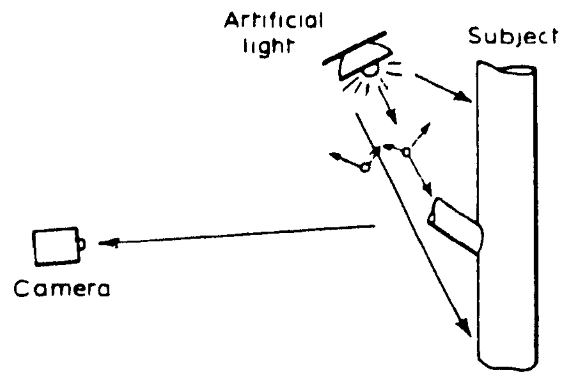


FIGURE 3.9 - AVOIDING BACK SCATTER[17]

All photographs should be numbered and labelled with a slate. If color photographs are used, a color chart indicating color distortions should be attached to the slate.[3]

3.11.4.1 CAMERA TYPES

There are several types of specialized cameras available for performing underwater inspections. Specialist companies have created a variety of camera types and can be grouped into the following categories:[17]

(a) The Nikonos System. This system is a totally waterproofed camera with interchangeable lenses. It can take close-up photographs, using either adapters or close-up lenses and can also use wide-angle or telephoto lenses. The camera uses standard 35 mm film cassettes and either dedicated or nondedicated flash units.

(b) Hydroscan. This is currently the most widely used 35 mm underwater camera. This camera, which has been specifically designed to take close-up photographs, consists of a close-up lens,

fixed prods to provide the correct stand-off distance, a ring flash, a 250-frame film cassette, an automatic film advance, and a special waterproof underwater housing. It is also available with digitalized electronics for printing information on each frame.

(c) *Hasselblad*. These cameras use medium format 6 cm x 6 cm (2.4 in x 2.4 in.), single lens reflex equipment. Several of these systems are housed in waterproof units. This camera system provides a very large negative which provides a good quality print.

3.11.4.2 FILM STOCK

The selection of a particular type of film is based on three factors: film speed (how quickly it reacts to light) whether it is monochrome or color and whether it is small or medium format. These are summarized below.

(a) *Film Speed* Several numerical rating systems exist for categorizing film speed and are loosely grouped into slow, medium, and fast films (Table 3.8). The American Standards Association (ASA), the Deutsch Industries Norm (DIN), and the International Standards Organization (ISO) systems rate 25 and 50 ASA films as slow films, 100 and 200 ASA as medium, and 400 and 800 ASA as fast films.[17]

TABLE 3.8 - RATING SYSTEM AND FILM SPEED[17]

System	Film speeds					
ASA	25	50	100	200	400	800
DIN	15	18	21	24	27	30
ISO	25/15	50/18	100/21	200/24	400/27	800/30

(b) *Color*. Monochrome films are rarely used for underwater inspections. Therefore, selecting a film type usually involves a choice between color prints or color positives (slides). The advantages of color slides are that they can be viewed without having to process prints, are easy to develop on site, and are readily available. One disadvantage, however, is slides cannot be presented in a report. Also, slides are very sensitive to exposure errors. On the other hand, color prints are less affected by exposure errors. Cclor reprints are easy to obtain and are easily presented in reports.[17] To speed up the processing of color prints, a 35 mm Polaroid film with a portable processor is used. This equipment is not expensive but does not provide the same degree of photograph resolution that can be obtained by using standard 35 mm color film.[3]

(c) *Format.* There is a choice between two basic formats: small (35 mm) or medium (6 cm x 6 cm). The small format can be enlarged to a greater degree than the medium format, resulting in less loss of quality in the final print.[17] Selecting a particular film type for a particular application is summarized in Table 3.9.

TABLE 3.9 - SELECTION OF FILM[17]

Requirement	Type of film	Suggested ASA rating
Flash photography	Slow/medium	50 or 64
Available light photography	Medium/fast	200 or 400
Requirements for good enlargements	Medium format 6 cm x 6 cm	Depends on type of light

3.11.5 PHOTOGRAMMETRY

Photogrammetry or stereo photography is used when depth or size perception is desirable. Photogrammetry involves analyzing "information contained in two (stereo) pictures of the same scene taken from different angles".[17] Photogrammetry was first developed for use on land but modified equipment for underwater application has been available for quite some time.

The underwater method produces stereo pictures by using two cameras which are usually based on the Hasselblad equipment. The system is usually equipped with calibration devices to allow for various camera angles and water conditions. The stereo photographs are analyzed by a computer which can provide printouts to suit specific needs, and can be interfaced with other computer systems. Measurements in all three dimensions can be obtained from the photographs with an accuracy of plus or minus 0.33 mm (0.01 in.).[17]

3.12 STRUCTURE EVALUATION

An effective repair and maintenance program cannot be selected until the basic cause of the deterioration is determined. Underwater inspection findings can be used to make an engineering assessment of the structure to determine the cause, extent, and rate of deterioration, as well as its structural capacity and safety. This information may also be used to predict the remaining useful life of the structure and to develop a repair and maintenance program which will allow the safe operation of the structure. Similarly, the inspection results can be used to determine the required manpower and financing needed to perform the repair work. Once the findings of the engineering assessment are available, a management decision must be made whether to allow deterioration to continue, to make repairs that will keep the structure in its present condition, to make repairs that will strengthen the structure or to replace the structure.[12]

The decision-making process can be simplified by developing a condition rating index of the structure which will indicate the urgency of corrective action. The urgency index is developed to assess the condition of the structure and the possibility for further deterioration. The index is used to keep the facility operating at a specific load and safety level. The rating index decreases with increasing deterioration.[4] An example of an urgency index rating system is shown in Table 3.10. The rating can also be modified based on engineering judgement of the deterioration. The modification is shown in Table 3.11.

TABLE 3.10 - URGENCY INDEX RATING SYSTEM[4]

Maintenance Urgency Index	Conditions Noted Based on Maintenance Rehabilitation Level	Maintenance Urgency Definitions Based on Perceived Need for Rehabilitation
9	New Condition	No work required
8	Good Condition	Work required only for cosmetic repairs. Items should be tracked for future maintenance.
6 to 7	Maintenance Required	Items should be considered for maintenance work during next work cycle.
4 to 5	Rehabilitation Required	Allowable load capacity is below design capacity. Rehabilitation work should be performed within next year or sooner.
2 to 3	Major Repair Required	Load capacity of structure is severely impaired. Repairs to be accomplished as soon as possible.
0 to 1	Facility Closed	Facility is unsafe for intended operations. Major reconstruction necessary

TABLE 3.11 - MODIFICATION TO URGENCY INDEX SYSTEM[4]

Modification	Description
+2	No threat for 5± years. Conditions have stabilized, and condition is cosmetic in nature.
+1	No threat for 2 to 3 years. Conditions are worsening slowly, and has little structural effect.
0	No immediate threat. Conditions are deteriorating at normally expected rate, and deficiency does not lead to reduction of structure capacity.
-1	Threat likely within 1 year. Conditions are worsening and are causing reduction in structure capacity.
-2	Threat is imminent. Conditions are worsening rapidly and severe loss of structure capacity is occurring.
Note: Modifiers cannot increase rating above '8'.	

Management decisions must be based on the consideration of several important factors, such as safety, the operational need for the structure, the adequacy of the structure to meet the needs of its intended purpose, economic considerations, appearance, environmental conditions, and other factors having an impact on the structure[3]. A flow chart developed by Brackett et al.[28] summarizes the decision making process for a maintenance and safety program of waterfront facilities, and is shown in Figure 3.10.

3.13 FINAL REPORT

The final phase of any inspection program should be the preparation of the final report. In some cases, the reporting procedure is dictated by the client. In other instances, the form of the report is not specified and it is up to the engineer or inspector to develop a format. There are no set rules for presenting reports as they vary from one company to another.

The basic requirements for good report writing are grammar skills, syntax, style, punctuation, usage, and organization.[25] It is essential for the writer to be clear, concise, and to the point. Technical aspects should be kept separate from qualitative discussion. Most clients do not have the technical background to understand engineering principles.

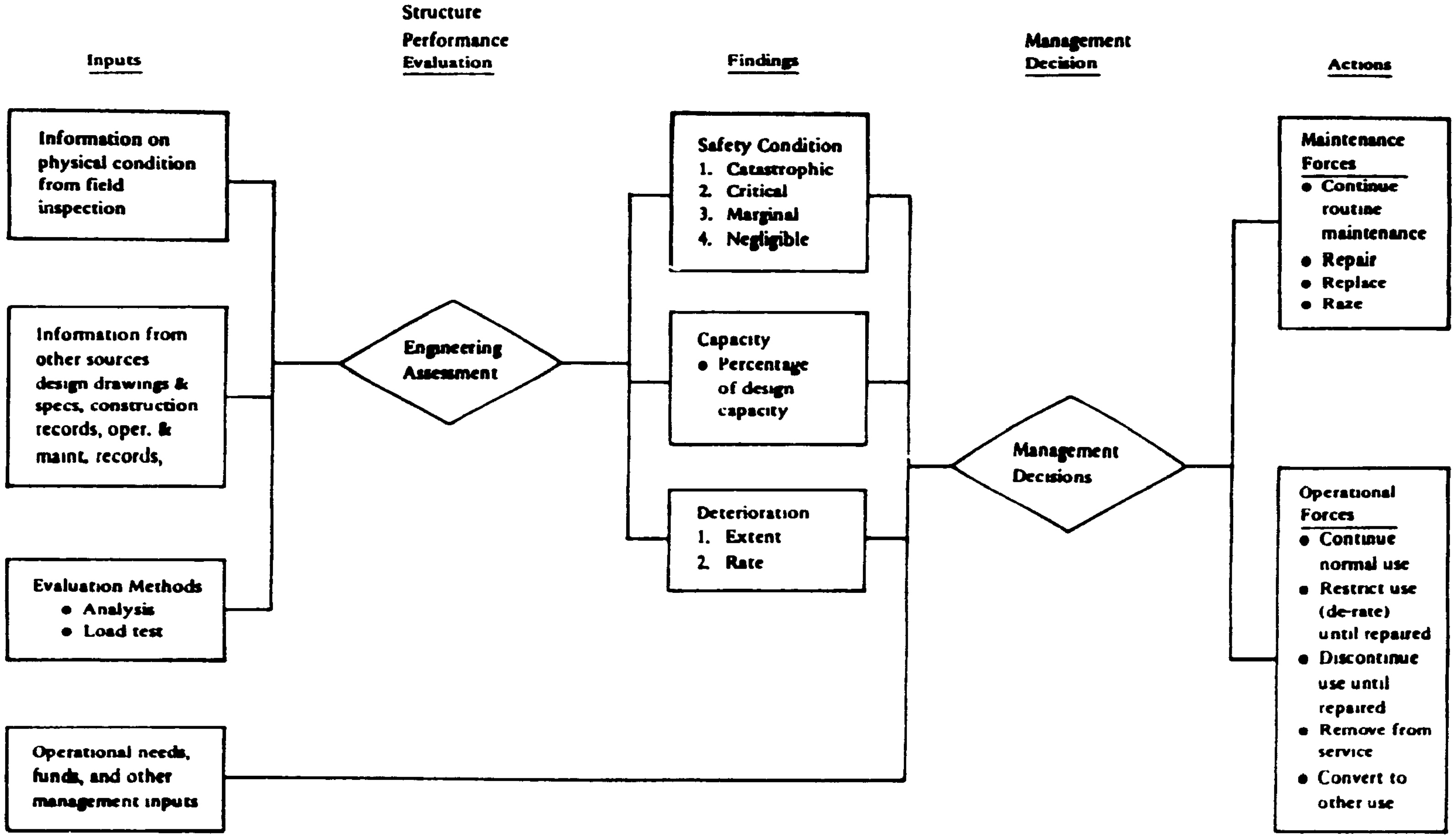


FIGURE 3.10 - DECISION MAKING PROCESS FOR MAINTENANCE AND SAFETY PROGRAM OF MARINE STRUCTURES[28]

3.13.1 FORMAT OF THE REPORT

The report should be typed and bound. The report should have a title relevant to the project as well as a table of contents, and executive summary. All pages should be numbered, with the latest issue number indicated, and dated. Sections within the report should also be numbered. Any drawings or photographs pertaining to the inspection must be included in the report and should be cross-referenced. Any relevant video recordings of the inspection should also be referenced, and should indicate the place where they can be viewed.[17]

If required by the client, the report should provide recommendations for repairs including drawings or sketches showing the work. Construction cost estimates for the proposed repair work should also be provided for decision making purposes. Lastly, the report must be carefully reviewed and signed by an authorized representative within the company. A general outline which can be used for most inspection reports is shown in Table 3.12. Some of these topics are summarized below.

TABLE 3.12 - GENERAL REPORT OUTLINE*

Letter of Transmittal
Executive Summary
Table of Contents
1. INTRODUCTION
1.1 Objective
1.2 Scope
1.3 Background Information
1.4 Available Information
2. INSPECTION FINDINGS
3. LABORATORY TESTING
4. THEORETICAL ANALYSES
5. DISCUSSION
6. CONCLUSIONS and RECOMMENDATIONS

*Adapted from Reference 25

3.13.2 CONTENTS OF THE REPORT

3.13.2.1 EXECUTIVE SUMMARY

This section should be limited to one page whenever possible. It should describe in concise terms what the report covers, what was done, and give a summary of the major conclusions and recommendations.

3.13.2.2 SCOPE AND OBJECTIVE

This section should outline the specific purpose for the inspection and what work was performed. It should identify the structure that was inspected, as well as its location. The scope should also identify the parties who sponsored (or authorized) the investigation, and the personnel who performed the inspection and testing.

3.13.2.3 BACKGROUND INFORMATION

A brief description of the structure or facility should be provided including vicinity, locality, and historical background if available. Whenever possible, plan-view maps, elevations, sections of the structure,and geological maps should be included along with operational information of the facility. All information obtained from other sources which has not be verified by the writer should also be included.

3.13.2.4 AVAILABLE INFORMATION

All existing documents and engineering data which was reviewed and referenced in the report should be listed, with their title, date and origin. The documents may include original construction drawings of the structure and previous inspection and repair reports.

3.13.2.5 INSPECTION FINDINGS

This section of the report will provide a detailed description of the observed conditions of the structure including a brief explanation of the various in-situ testing techniques used. It is important that this section only include facts and observations, and not opinions. All relevant photographs should be included and referenced. Each photograph should be numbered and should provide a brief description of its contents. The degree of deterioration should be diagrammatically illustrated whenever possible. Any inspection forms used for documenting the inspection results should be referenced and included as an appendix to the report. Any relevant video recording should also be referenced and made available for viewing.

3.13.2.6 LABORATORY TESTING

A description of the laboratory tests that were performed should be provided. As with inspection findings, only facts should be reported in this section. Interpretation of test results is included in the discussion section of the report. Photographs or drawings of where cores or physical samples were removed should also be provided.

3.13.2.7 THEORETICAL ANALYSES

This section should describe any structural analyses or calculations that were used to determine the service load capacity of the deteriorated structure or member. The analyses should include the effects of deterioration as determined by field and laboratory investigations. Based on these, the reduced capacity of the structure or element is developed.

3.13.2.8 DISCUSSION

This section provides an interpretation of field investigations, laboratory tests, and engineering calculations. All discussion must be based on the facts already presented in previous sections of the report. If the discussion is extensive, dividing it into appropriate subsections may be useful.

3.13.2.9 CONCLUSION AND RECOMMENDATIONS

This is the most important section of the report which provides a general summary of the findings and characterizes the condition of the structure. If an unsafe condition exists, it should be identified and methods for temporary bracing should be suggested[26]. It should specify the adequacy of the structure or facility based on current design, and operational criteria. The remaining useful life of the structure should also be estimated. Recommendations for possible repair techniques should be provided along with construction cost estimates. Recommended repair procedures should be schematically illustrated showing the major features of the repair work.

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CHAPTER 4
EVALUATION TECHNIQUES FOR IN-SITU CONCRETE

4.1 INTRODUCTION

Organizations or owners concerned with the durability and continued operation of concrete structures must perform periodic visual inspections. During these inspections, the observed damage or deterioration may require a more critical examination. In other cases, concrete structures may experience internal deterioration (e.g., corrosion of reinforcing steel) which becomes visible only after the deterioration has progressed significantly. The qualitative data obtained from visual inspections is generally inadequate to accurately assess the condition of the structure. In such cases, these inspections must be supplemented with more sophisticated methods for obtaining quantitative data to determine the cause, extent, and rate of deterioration.

A wide range of techniques and apparatus are available to the engineer to examine and evaluate concrete structures above water. Some of these methods have been adapted for inspecting concrete structures under water. These test methods can either be nondestructive or partially destructive in nature. The latter is often used for retrieving samples for laboratory testing, thus requiring some form of repair to the concrete after the testing is completed. Depending on the information which is sought, the inspector will often be required to use a combination of these methods to be able to determine the primary cause or causes of deterioration.

Generally speaking, these tests can be used to evaluate the following four areas of concern regarding the deterioration of reinforced concrete structures:[1]

- Concrete quality and composition
- Concrete strength
- Corrosion of embedded reinforcing steel
- Structural integrity and performance

Although not intended to be a complete guide, the following sections provide a summary of the most commonly used methods for testing and evaluating existing concrete structures. Though some of the methods described here were developed in Europe, they are all available in the USA and Canada through specialist companies. An excellent review of in-situ/nondestructive testing of concrete is provided in Reference 2. A list of in-situ inspection techniques and common laboratory tests for concrete are also provided in Appendices I and J, respectively. The material in this chapter has been adapted from different available references, especially 1 and 5, and are presented here for completeness.

4.2 CONCRETE QUALITY

4.2.1 REBOUND HAMMER TEST

The rebound hammer, also known as the Schmidt or Swiss Hammer, is a surface hardness tester that determines the uniformity of in-situ concrete, so that areas of inferior quality can be detected. It measures the rebound distance of a spring-driven mass after it impacts the concrete surface with a standard force. It can be used above or below the water and is useful in new construction to assist contractors in determining stripping times for formwork.[1] A cutaway view showing the various parts of a typical rebound hammer is shown in Figure 4.1.

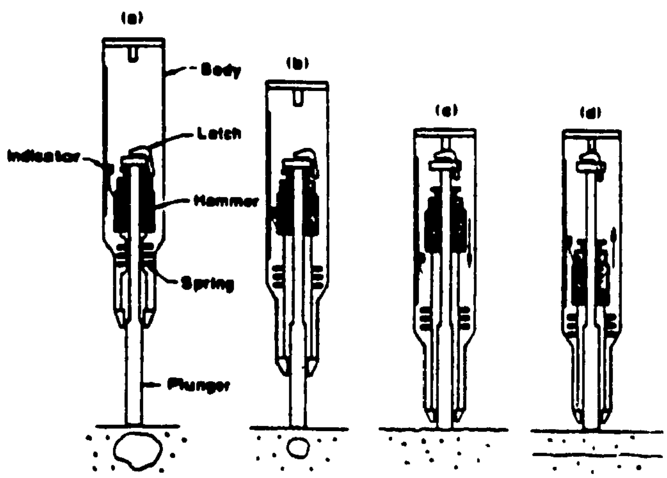


FIGURE 4.1 - CUTAWAY VIEW OF A TYPICAL REBOUND HAMMER[1]

To carry out the test (ASTM C 805), the impact plunger is pressed firmly against the concrete surface, thereby releasing the spring-loaded mass from its locked position. The mass then strikes the steel plunger which is in contact with the concrete surface. The resulting 'Rebound Number' or rebound distance of the hammer is read on a linear scale attached to the instrument. A recording type of Schmidt hammer is available which records the rebound numbers automatically on a roll of paper.[3] To obtain representative data, it is advisable to take a minimum of twelve readings at each test location and averaged (excluding the minimum and maximum values).[4] This number can be used to check concrete uniformity by comparing test data from various parts of the structure. In general, the higher the rebound number the better the concrete quality. If calibrated with laboratory tests on concrete cubes or cylinders, the hammer can give an indication of the in-situ compressive strength.[5]

Although using the rebound hammer is a quick and inexpensive way of determining concrete uniformity, it has many limitations which must be recognized by the user. For example, the results of the rebound number are affected by smoothness and moisture condition of concrete surface, and type of coarse aggregate.[2] Smoother surfaces usually give higher rebound numbers with less scatter in the data.[4] Thus, if the concrete surface is rough, it should be smoothed with a medium-grained silicon carbide stone.[6] On the other hand, saturated concrete tends to give rebound numbers slightly lower than when tested dry.[4] If repair patchwork has been done, test locations away from the patches should be chosen, since the characteristics of the repair material may differ from those of the structure. Processes that harden the concrete surface such as curing membranes or carbonation, may also give higher rebound numbers. Since the velocity of the impact plunger is affected by gravity and friction, rebound numbers obtained in the vertical and horizontal plane of the same test location will be different.[5] Figure 4.2 shows the typical effect on the rebound number when the impact plane is not horizontal.

4.2.2 WINDSOR PROBE TEST

This instrument, although less useful than the rebound hammer, may also be used to determine uniformity by measuring the penetration resistance of concrete at different locations of the structure. The Windsor Probe which is standarized by ASTM C 803, consists of a gun loaded with a hardened alloy probe which is driven into the concrete. The length of the probe which remains exposed provides a measure of the penetration resistance of the concrete. To ensure that the test is performed with some degree of uniformity, ASTM C 803 specifies a maximum probe velocity variation of three percent, based on a minimum of ten tests.[1] As with the rebound hammer, a calibration chart must be made to correlate probe penetration with concrete compressive strength,[6] because calibration curves supplied by the manufacturer are not always reliable. This method is partially destructive and requires repairing the concrete surface after the probe is removed. The test results are also affected by the hardness of the aggregate.[2]

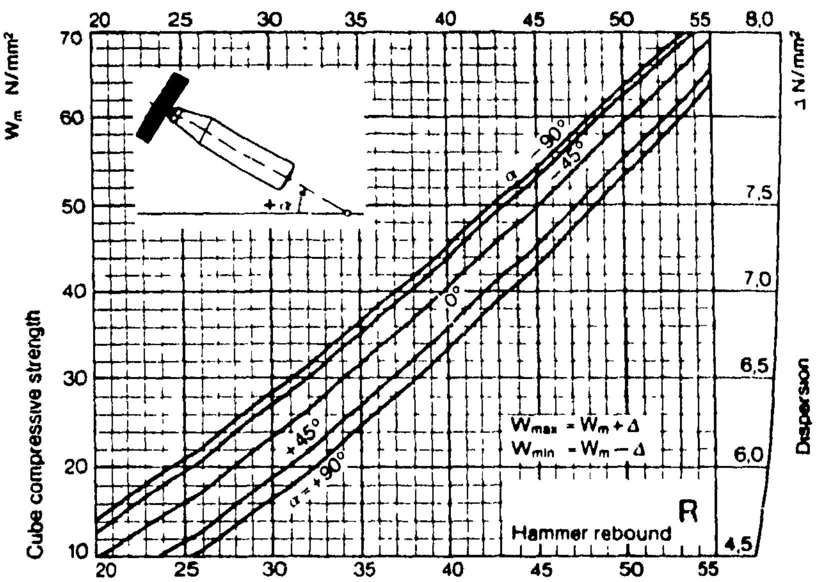


FIGURE 4.2 - EXAMPLE OF REBOUND HAMMER CALIBRATION CURVES OF CONCRETE FOR VARIOUS INCLINATION ANGLES (PROCEQ SA ZURICH)[3]

4.2.3 ULTRASONIC PULSE VELOCITY (UPV)

The UPV method consists of measuring the time it takes a direct compression wave to pass through the concrete (Figure 4.3). The time of travel between the initial propagation and reception of the pulse is measured by an electronic trigger/timer device. The average wave velocity is then computed by dividing the measured path length of the wave by the time of travel. Ultrasonic test procedures are standardized by ASTM C 597.

Since UPV is a function of the modulus of elasticity and density of the material through which it travels, this method provides comparative data for assessing concrete uniformity, as well as locating defects (i.e., cracks, voids, etc.). In some cases, it has been used for estimating in-situ compressive strength. However, the relationship between pulse velocity and concrete strength are affected by several variables, including age of concrete, moisture conditions, aggregate to cement ratio, type of aggregate, surface finish, and location of steel reinforcement.[2] Table 4.1 shows the relationship between pulse velocity and concrete condition, as suggested by Whitehurst,[7] and the minimum acceptable velocities for specific structure types (in Great Britain) are presented in Table 4.2, as suggested by Jones.[8]

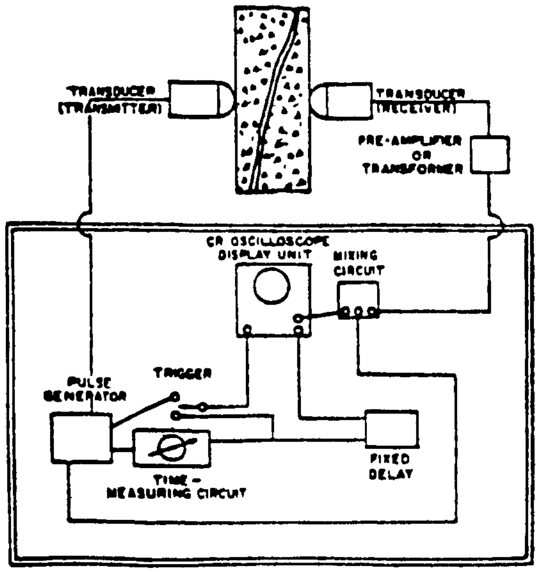


FIGURE 4.3 - SCHEMATIC DIAGRAM OF ULTRASONIC TESTING DEVICE[1]

TABLE 4.1 - RELATIONSHIP BETWEEN PULSE VELOCITY AND CONCRETE CONDITION[8]

Pulse Velocity		
ft/sec	m/sec	General Condition
above 15,000	above 4570	excellent
12,000-15,000	3660-4570	good
10,000-12,000	3050-4570	questionable
7,000-10,000	2135-3050	poor
below 7,000	below 2135	very poor

TABLE 4.2 - MINIMUM VELOCITIES ACCEPTABLE FOR SPECIFIC STRUCTURAL PURPOSES[6]

Types of work	Minimum value of wave velocity for acceptance	
	ft/sec	m/sec
Prestressed concrete, T-sections	15,000	4570
Prestressed concrete, anchor units	14,300	4360
Reinforced concrete frame building	13,500	4115
Suspended floor slab	15,000	4725

The UPV method has traditionally been performed by passing ultrasonic pulses through the concrete between fixed points as illustrated in Figure 4.4. The most common and most accurate method is direct transmission when the transducers are on opposite, parallel faces of the test location. Semi-direct transmission is less accurate and not normally used because it is difficult to duplicate the transmission path. The least accurate is indirect transmission and is used when only one surface of the concrete is accessible, such as a retaining wall.[6]

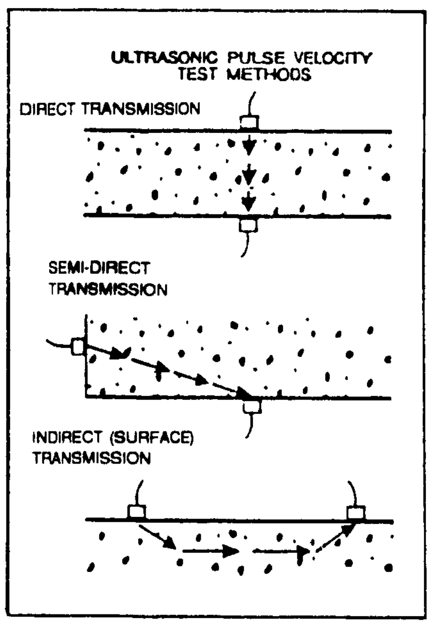


FIGURE 4.4 - TRADITIONAL UPV TEST METHODS[9]

With the recently developed scanning system, shown in Figure 4.5, the source and receiver transducers are free to move while obtaining UPV measurements. The UPV scanners consist of coated piezoelectric source/receiver ceramics in a cylindrical shape which allows them to roll while transmitting and receiving wave pulses. The signals are first amplified and filtered, and then transmitted into a data acquisition system Typical scanning speeds range from 0.15 to 0 31 m/s (0.5 to 1 ft./s), and can collect data at rates of 5 to 10 pulses/second. This speed provides test data every 2.5 to 5 cm (1 to 2 in.) along the entire path. With current data acquisition systems, it is possible to make scans of over 9 m (30 ft.) in length at a time. After scanning, the data is analyzed by a computer system which computes the concrete characteristics at each test location.[9]

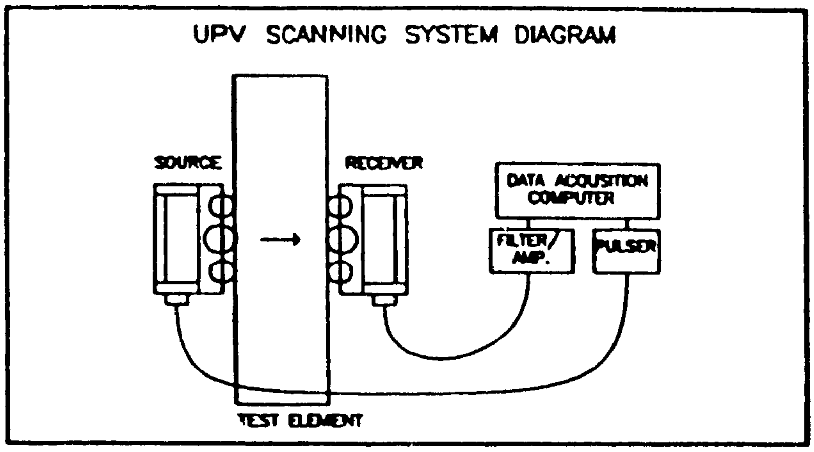


FIGURE 4.5 - UPV SCANNER SYSTEM[9]

4.2.4 CROSSHOLE SONIC LOGGING (CSL)

Sonic logging is used to perform the UPV test in areas which are inaccessible or under water Logging consists of transmitting an ultrasonic pulse through the concrete between source and receiver probes which are placed in water-filled tubes (Figure 4.6). The probes are lowered into preplaced access tubes (PVC sleeves) or coreholes by cables that are pulled over a special winch, which measures and records the probe depth. Sonic logging is performed as the probes are withdrawn simultaneously, thus providing a continuous profile of travel time The method is capable of taking measurements at 25 mm (1 in.) spacings.[10] It can also be used in coreholes drilled through the base of the foundation to assess the integrity of the interface between bedrock and the foundation concrete.[5]

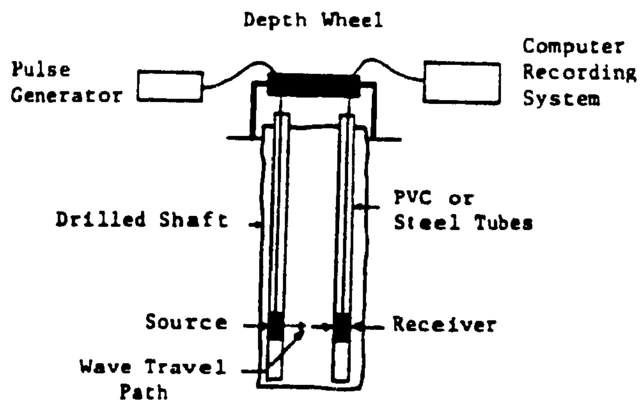


FIGURE 4.6 - CROSSHOLE SONIC LOGGING METHOD[10]

The measured travel times between probes and the corresponding wave velocities are used to evaluate the concrete quality. For example, longer travel times indicate irregularities in the concrete, while complete loss of signal is indicative of a defect in the concrete between the tubes.[10]

4.2.5 DIAGRAPHIC DRILLING

Diagraphic drilling is a destructive exploratory technique used to determine the mechanical characteristics and quality of large concrete structures. During the drilling, several parameters are recorded, such as the instantaneous penetration rate of the drilling tool, the compressive force (thrust) on the drill rod, and the torque applied on the drilling tool. These parameters are related to the mechanical strength of the concrete and to its cohesion. Diagraphic drilling was used for the in-situ investigation of an old concrete quay wall in Zeebrugge, Belgium.[11] A typical diagraphic drilling record is shown in Figure 4.7. Interpretation of the diagrams are typically supported by video inspection of the boreholes, and supplemental laboratory testing of concrete cores. The advantages of diagraphic drilling include ease of application, rapid execution, and the relatively low cost.

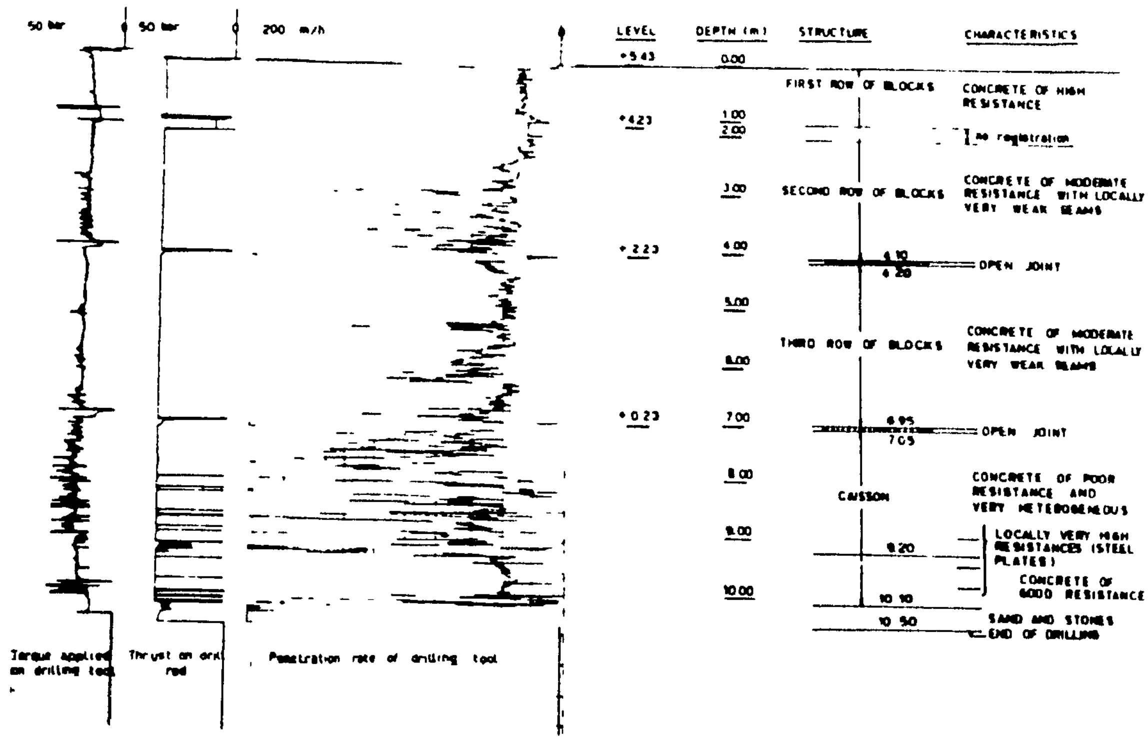


FIGURE 4.7 - A TYPICAL DIAGRAPHIC DRILLING RECORD[11]

4.2.6 RADIOMETRY

Radiometry is primarily used to measure in-situ density and thickness of concrete members. The Smith and Whiffin method, and the Brocard method are the methods most commonly used for determining the in-situ density of concrete. The Smith and Whiffin test consists of drilling holes into the concrete surface and lowering a radioactive source, such as cobalt or radium, down into the holes. Geiger counters, placed in heavy lead sheath, are then positioned on the outside vertical face of the concrete member at the same depth as the radioactive source. The Geiger counters are calibrated by taking readings on samples of known densities. From these readings, charts are prepared which are then used for determining the in-situ density of the concrete by comparison with subsequent Geiger readings.[1] The Brocard method is virtually the same, differing only in the radioactive source used and the thickness of the concrete member which can be tested (up to 410 mm/16 in.).[12]

4.2.7 IN-SITU PERMEABILITY

Concrete durability is closely related to its ease with which water (or other aggressive fluids) can move through its pore structure The rate at which fluids penetrate the concrete determines its permeability, and hence, its rate of deterioration. The permeability of in-situ concrete can be measured by a series of tests presently in use which are covered by the British Standards Institution (BSI), BS 1181, Part 5 The most commonly used include the Figg Hypodermic Test and the Initial Surface Absorption Test.[1] These are summarized below.

4.2.7.1 FIGG HYPODERMIC TEST

This test simply measures the time it takes for a change in pressure to occur in a fluid (air/water) sealed within an evacuated void in the concrete (Figure 4.8). The test procedure consists of drilling a 10 mm (⅜ in.) diameter by 40 mm (1-½ in.) deep hole into the concrete, and plugging it with a polyether foam and sealing it with a silicone sealant. For a water permeability test, a hypodermic needle is pushed through the plug and connected to a water source (100 mm/4 in. head) and a manometer. The time it takes the water to move a distance of 50 mm (2 in.) down a capillary tube is noted.

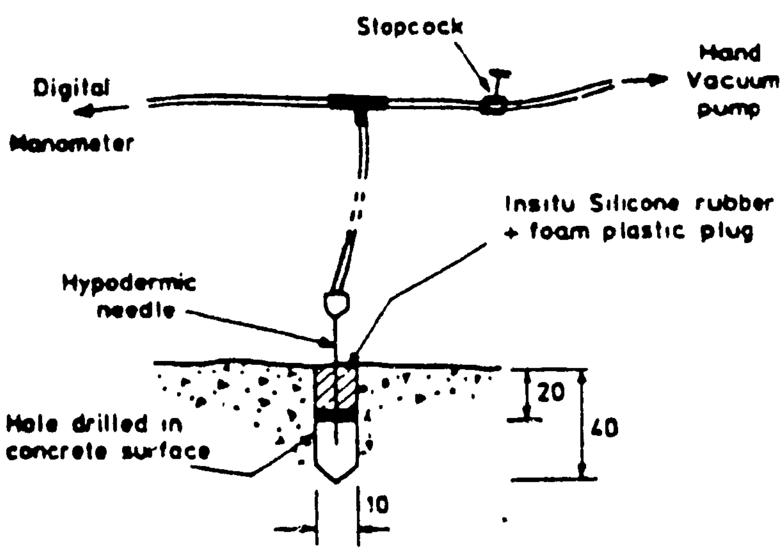


FIGURE 4.8 - SCHEMATIC REPRESENTATION OF AIR PERMEABILITY TEST SET-UP[1]

For the air permeability test, the hypodermic needle is connected to a vacuum pump and the time taken for a pressure drop of about 55 to 59 kPa (8 to 8.5 psi) within the sealed void is recorded. The permeability of concrete to water can be judged qualitatively according to the infiltration times shown in Table 4.3.

TABLE 4.3 - RELATIONSHIP BETWEEN PERMEABILITY AND TIME OF INFILTRATION[1]

PERMEABILITY	TIME OF INFILTRATION (Minutes)
GOOD	> 200
AVERAGE	50 - 200
POOR	< 50

4.2.7.2 INITIAL SURFACE ABSORPTION TEST

The initial surface absorption test measures the amount of water absorbed by the concrete per unit area under a constant pressure head (Figure 4.9). The pressure head (200 mm) is applied through a flexible inlet tube attached to a watertight cap which is clamped to the test area. An outlet tube is connected to a calibrated capillary scale which measures the water penetration into the concrete after the water source is closed. Measurements are taken at time intervals of 10 min., 30 min., 1 hour, and 2 hours from the start of the test. The expected durability of the concrete is classified in accordance with the time of infiltration rates shown in Table 4.4.

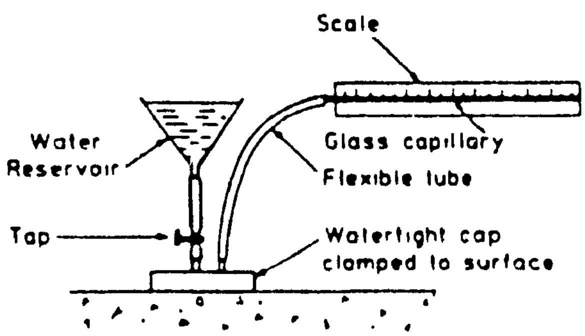


FIGURE 4.9 - SCHEMATIC OF INITIAL ABSORPTION TEST[1]

TABLE 4.4 - RELATIONSHIP BETWEEN CONCRETE DURABILITY AND TIME OF INFILTRATION[1]

DURABILITY	TIME OF INFILTRATION		
	10 Mins.	30 Mins.	1 Hr.
GOOD	< 0.25	< 0.17	< 0.10
AVERAGE	0.25 - 0.50	0.17 - 0.35	0.10 - 0.20
POOR	> 0 50	> 0.35	> 0.20

The following is a list of other permeability test methods currently being developed:[1]

- VTI test[13]
- ISE capillary test
- Water absorption test
- Pressure differential water permeability test
- Ionic diffusion test
- Gas diffusion test

4.3 CONCRETE STRENGTH

4.3.1 CONCRETE CORE TESTING

Concrete core testing is still considered to be one of the most reliable methods to determine concrete compressive strength. Coring is also useful for investigating a variety of other in-situ characteristics, such as the depth of surface deterioration, and the presence and size of visible cracks. Concrete coring is often used to verify the results of other in-situ tests and to provide a physical specimen for supplemental laboratory testing (Section 4.6). The cores should be taken in areas that are representative of the structure. Methods for achieving random sampling are also described in ASTM C 823.

Mather[8] suggests that if the concrete is in deteriorated condition or when drilling operations are questionable, better core recovery will be achieved with a 150 mm (6 in.) diameter diamond bit and barrel than with smaller ones. On the other hand, when the concrete is in fairly good condition, the driller is highly skilled, and the rig is operating efficiently, cores can be satisfactorily retreived using 55 mm (2-⅛ in.) diameter (Nx) bits.

Cores should be logged as they are removed from the hole and core holes should be accurately located on appropriate construction drawings. Cores should be properly packaged so that they will not be damaged or mixed up during shipment to the laboratory. In some cases, it may be necessary to place the cores into plastic sleeves, or wrapping them in cheesecloth dipped in liquid wax to preserve the field moisture content. Preserving the field moisture content is usually very important if some deleterious chemical reaction is suspected.[6] Examination of the cores once they are received at the laboratory are summarized in ASTM C 856 and preparation of a concrete cylinder for compression testing is described in ASTM C 39 specifications. It basically consists of capping the ends to achieve a smooth bearing surface to minimize or eliminate eccentricity during load application. The core is usually dimensioned so that the length is at least twice the diameter. After the specimen is prepared, it is placed into the testing machine and the load is slowly and continuously applied until failure. At the end of the test, the type of failure and general appearance of the concrete is noted on the test log. The compressive strength is calculated by dividing the maximum load at failure by the average cross-sectional area of the core.[1] Typical concrete failure modes are shown in Figure 4.10. In cases where the samples do not meet the specified length to diameter ratios, applicable correction factors are applied to the calculated compressive strengths (Table 4.5).[14]

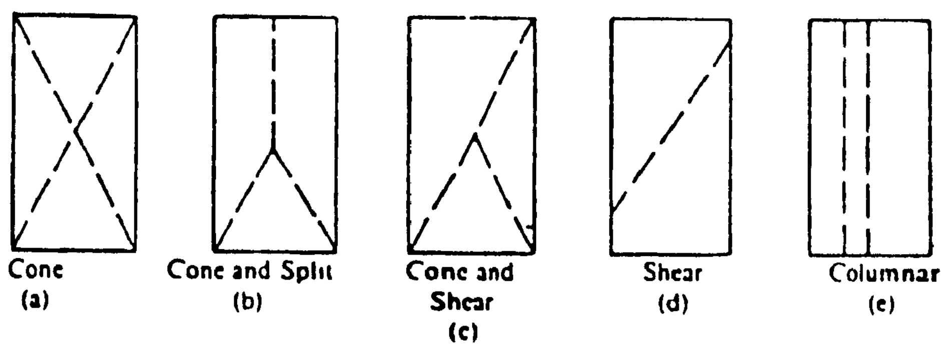


FIGURE 4.10 - TYPES OF FRACTURE OF CONCRETE CYLINDERS[1]

TABLE 4.5 - CORRECTION FACTORS FOR CONCRETE CORES[1]

Ratio of Length of Cylinder to Diameter (l/d)	Strength Correction Factor*
1.75	0.98
1.50	0.96
1 25	0.93
1.00	0.87

*These correction factors apply to lightweight concrete weighing between 1600 and 1920 kg/m^3 (100 and 120 lb/ft^3) and to normal weight concrete They are applicable to concrete dry or soaked at the time of loading. Values not given in the table shall be determined by interpolation. The correction factors are applicable for nominal concrete strengths from 13 8 to 41 4 MPa (2000 to 6000 psi) Correction factors depend on various conditions such as strength and elastic moduli Average values are given in the table

4.3.2 PULL-OUT TEST

The pull-out test is an in-situ method of determining concrete compressive strength by measuring the maximum force required to pull an embedded insert from the concrete mass. The concept was initially suggested by Skramtajew in 1938 and investigated further by Kierkegaard-Hansen. Current ASTM C 900 standards are based on tests conducted by Malhotra, Richards, and Rutenbeck in the early 1970s.

In general, a pullout test consists of pulling out a specially shaped steel insert from concrete (Figure 4.11) The required pullout force required is measured using a dynamometer. Due to its shape, a cone (frustrum) of concrete is pulled out with the insert, generating failure planes at approximately 45° to the direction of the pull. The pullout strength is approximately 20 percent of the concrete compressive strength.[2]

In a recent study, Collins and Roper[15] used pullout tests to evaluate concrete spall repairs. In the study, it was determined that pullout test methods can be used to simulate spalling concrete. All laboratory specimens were subsequently damaged by pullout testing, repaired with epoxy mortar and subjected to a second pullout test. The test program showed that the major factor governing the success of a repair to concrete is the soundness of the repair plane.

Due to the nature of the test, pullout techniques cannot be used on hardened concrete. To overcome this shortfall, new techniques have been developed in which a set of standard anchors are pulled out of standard drilled holes. The anchors can either be normal pullout inserts or split-sleeve wedge anchors. In the former case, a cone of concrete is pulled out while in the latter case, internal cracking of concrete is produced. Tests involving pulling out bolts set by means of epoxy in drilled holes have also been reported [2]

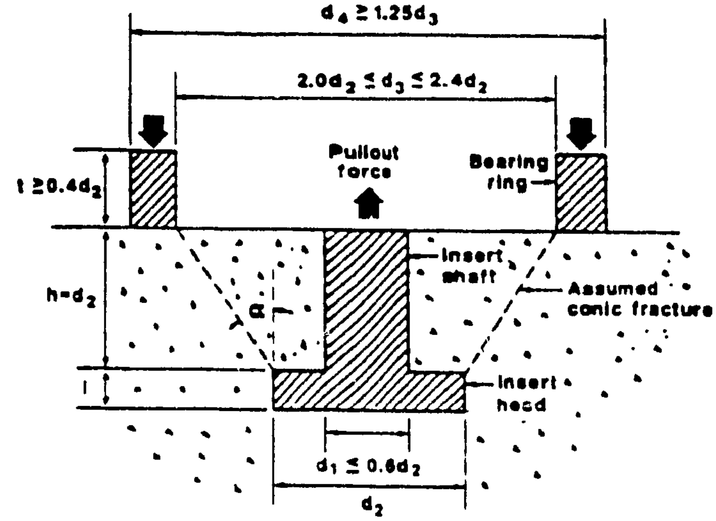


FIGURE 4.11 - SCHEMATIC CROSS-SECTION OF PULLOUT TEST[1]

4.3.3 PULL-OFF TEST

This method provides a means for determining concrete compressive strength by measuring the force required to pull free a steel probe which is bonded to the concrete surface with a high-strength adhesive (epoxy resin). An equivalent cube compressive strength is obtained using a calibration graph.

Typically, the bond strength between the probe and the concrete surface is considerably higher than the tensile strength of the underlying concrete, thus eliminating the possibility of failure at the interface. Failure usually occurs within the underlying concrete mass. The approximate tensile strength of the concrete is computed by dividing the load required to break off the concrete mass by the area of the probe. The compressive strength of the concrete is obtained by dividing the cross-sectional area of a specimen by the load required to crush it.[1]

In cases where the concrete surface is too smooth to allow a good bond between the epoxy/concrete interface, the concrete surface is partially cored to expose a rough finish so that a better bond could be achieved for the probe.

4.3.4 BREAK-OFF TEST

This method is used for determining the flexural strength of concrete at some distance away from the surface [2] This is done by breaking a 55 mm (2-⅛ in.) diameter cantilevered core formed by cutting a circular slot in the existing concrete member. The core is broken off at its base by applying a force at the top with a hydraulically-operated jack which is coupled to a load cell. The force required to break off the core is correlated with compressive strength by means of calibration charts, developed by cylinder compression testing of retrieved cores.[1]

4.3.5 INTERNAL FRACTURE TEST

The Internal Fracture test (BSI 1986B) consists of drilling a 6 mm (¼ in.) diameter hole into the concrete test location to a depth of approximately 35 mm (1-⅜ in.), installing a 20 mm (3/4 in.) expanding wedge anchor into the hole, and pulling the anchor with a torque meter fitted on a 75 mm (3 in.) reaction tripod. The maximum torque value, averaged over a minimum of six readings, is correlated with the compressive strength of the concrete by means of calibration curves.[1]

4.4 REINFORCEMENT CORROSION

4.4.1 HALF-CELL POTENTIAL TEST

The half-cell potential test is used for determining the probability of active corrosion of steel reinforcement in concrete.[3] The test, which is standardized by ASTM C 876, measures electrical potential differences between anodic and cathodic areas that exist in an active corrosion process by means of standard half-cells. Since the corrosion reaction in the concrete is dependent on the ambient temperature, it is reported that useful readings are usually obtained at temperatures in excess of 5°C (41°F.)[1]

The test is performed by connecting the negative terminal of a high-impedance millivoltrneter to the embedded reinforcing steel and the positive terminal to a half-cell (Figure 4.12). The half-cell consists of a copper electrode which is immersed in a copper sulfate electrolyte solution. The electrode is connected to the voltmeter by a lead wire. The other end of the cell consists of a permeable pad through which the copper sulfate solution can make electrical contact with the concrete. Other types of half-cells such as silver/silver chloride can be used, but copper/copper sulfate is the most common.

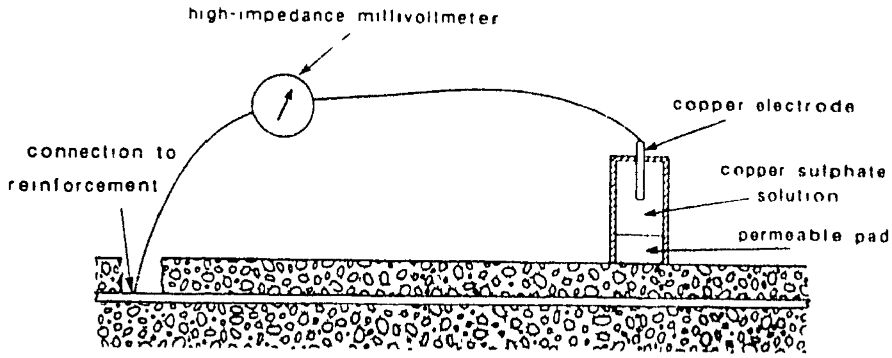


FIGURE 4.12 - SCHEMATIC DIAGRAM OF EQUIPMENT FOR HALF-CELL POTENTIAL TEST[3]

By taking readings at multiple locations, an evaluation of the corrosion activity of embedded steel or other metals can be made. If sufficient readings are taken in a predetermined pattern, equipotential lines can be drawn to create a diagram which resembles a contour map (Figure 4.13) The isopotential lines are created by connecting points of equal electrical potential The general pattern of equipotential contours can readily identify areas of high corrosion activity and areas which are on the verge of developing corrosion activity.[3,16] Recent developments in survey techniques has made this process much quicker and less tedious to use. An example of this is the potential wheel which gives a continuous print out of electrochemical potentials rather than spot readings on a fixed grid (Figure 4.14).[17]

According to the current ASTM standards, areas that show potentials more negative than -350 mV are said to be actively corroding with a probability of more than 90 percent Corrosion is negligible (less than 10 percent probability) in areas where the potential is less negative than -200 mV. At intermediate potentials (between -200 mV and -350 mV), the state of corrosion activity is uncertain (Table 4.6).

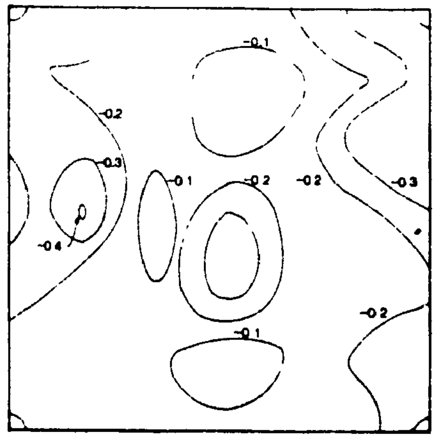


FIGURE 4.13 - TYPICAL ISOPOTENTIAL LINE DIAGRAM CONSTRUCTED FROM HALF-CELL POTENTIALS[16]

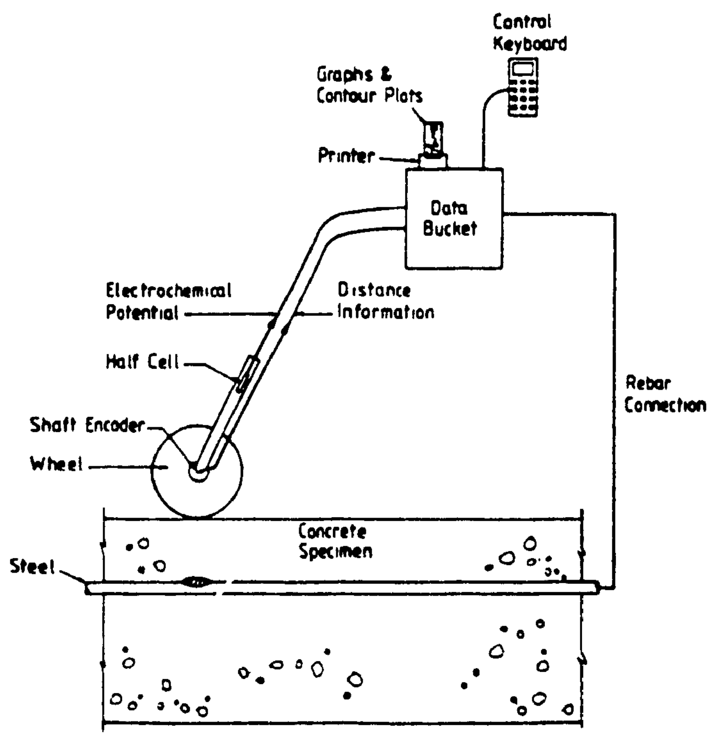


FIGURE 4.14 - POTENTIAL WHEEL FOR RAPID POTENTIAL SCANNING[17]

TABLE 4.6 - PROBABILITY OF OCCURRENCE OF CORROSION[1]

VOLTAGE OF HALF-CELL (Copper Sulfate)	COMMENTARY
U < 200 mV	The probability that there is no corrosion in this area is higher than 90 percent
200 mV < U < 350 mV	One cannot state with certainty that there is active corrosion in this area
U > 350 mV	The probability that there is corrosion in this area is higher than 90 percent

Different investigators have assigned different values to these criteria. For example, recent work by Pfeifer et al.[18] defined the threshold limit of corrosion to be -230 mV. Also, work from Concrete-In-The-Ocean projects in the U.K.[17] found that the corrosion risk criteria described in the current ASTM C 876 standard is not applicable to all corroding structures. On some of the structures surveyed during the project, very negative potentials were found with no sign of deterioration. Table 4.7 shows some of the results obtained and the associated risks found by breakout and examining cracking.

TABLE 4.7 - POTENTIAL MEASUREMENT AND ASSOCIATED CORROSION RISK RESULTS (CONCRETE-IN-THE-OCEANS, U.K.)[17]

Risk	ASTM Bridge Decks (USA)	Royal Sovereign Lighthouse (internal below sea water)	Precast Columns (UK)	Reinforced Concrete Specimens at 300 mm in sea water
Low	0 to -200 mV	-262 to -400 mV	0 mV or more	-900 to -1200 mV
Medium	-200 to -350 mV		0 to -100 mV	
High	-350 mV or less		-100 mV or less	

It is important to realize that, while potential measurements give an indication of corrosion activity, it does not show the extent or rate of corrosion. Half-cell potential readings should be correlated with data from other test methods (described elsewhere in this chapter) to determine the extent and rate of corrosion activity.

4.4.2 CONCRETE RESISTIVITY

As described earlier, the presence of a conductive medium or electrolyte is one of the necessary requirements for the corrosion process to initiate. Therefore, the rate of reinforcement corrosion in concrete depends on, among other factors, the capacity of the concrete to resist the flow of electrical currents. Since flow of electrical current is inversely proportional to resistivity, a measure of the concrete resistivity is indicative of the likelihood of corrosion in the reinforcing steel. Experience has shown that a high resistivity is usually associated with a low corrosion risk and vice-versa.[5]

Measuring the bulk resistivity of concrete is usually done by Wenner's method which is standardized by the British Standards Institution BS 1881 Part 5. An array of four electrodes placed against the concrete surface pass a current through the outer two electrodes using the concrete to complete the electrical circuit (Figure 4.15). The voltage drop which occurs across the inner two electrodes is recorded. The resistivity is calculated by using an empirical expression relating current, voltage drop, and spacing of the electrodes.[1]

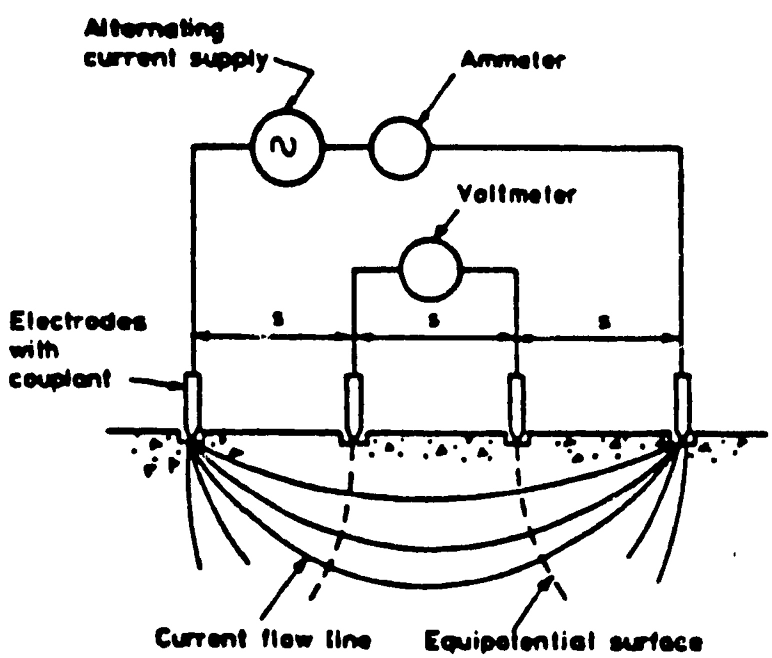


FIGURE 4.15 - SCHEMATIC OF RESISTIVITY TEST SET-UP[1]

It should be noted that resistivity measurements are affected by moisture conditions. In dry conditions, half-cell or resistivity tests may indicate no corrosion activity even though corrosion may be well advanced. Tests should therefore be conducted during wet seasons or after the structure is wetted thoroughly . Caution must be exercised when using these methods on post-tensioned structures, as test results may not be pertinent to the condition of the tendons themselves.[5] A state-of-the-art review of electric potential and resistivity test methods is given by Figg and Massden.[19]

4.4.3 PACHOMETER SURVEY

A pachometer survey is usually performed as part of the detailed inspection. The pachometer (or covermeter) is an instrument used to locate and map embedded reinforcing steel and to measure the depth of concrete cover over the rebar. These instruments are commercially available for use in dry environments and are easily adapted for underwater use.

The pachometer typically consists of coils wrapped around U-shaped magnetic cores. A magnetic field is produced by sending an alternating current to one of the coils and measuring the current which is developed in the other coil. The magnitude of the measured current is affected by both the diameter of the rebar and the distance from the coils. The concrete surface is scanned with the probe until a maximum meter reading is obtained, giving the location and orientation of the embedded rebar. A maximum meter reading will be obtained when the axes of the probe poles are parallel to and directly over the axis of a reinforcing bar.[4] A display dial is graduated to indicate the depth of the steel.[16] In general, pachometers are calibrated for rebars ranging from 10 M to 45 M (ASTM Nº 3 to Nº 16) in size, and can be used to measure depths of concrete cover ranging from 6 to 200 mm (¼ to 8 in.) in thickness.[4]

Other magnetic objects in the vicinity of the rebar where the measurement is being taken will affect the pachometer survey. It may be unable to distinguish individual bars if the rebar is bundled or too close. The effects of parallel 25 mm (1 in.) diameter rebars, located 50 mm (2 in.) below the concrete surface, is shown schematically in Figure 4.16. It is reported that, if the center to center distance of two parallel rebars is at least three times the thickness of the concrete cover, this effect will be negligible.[4]

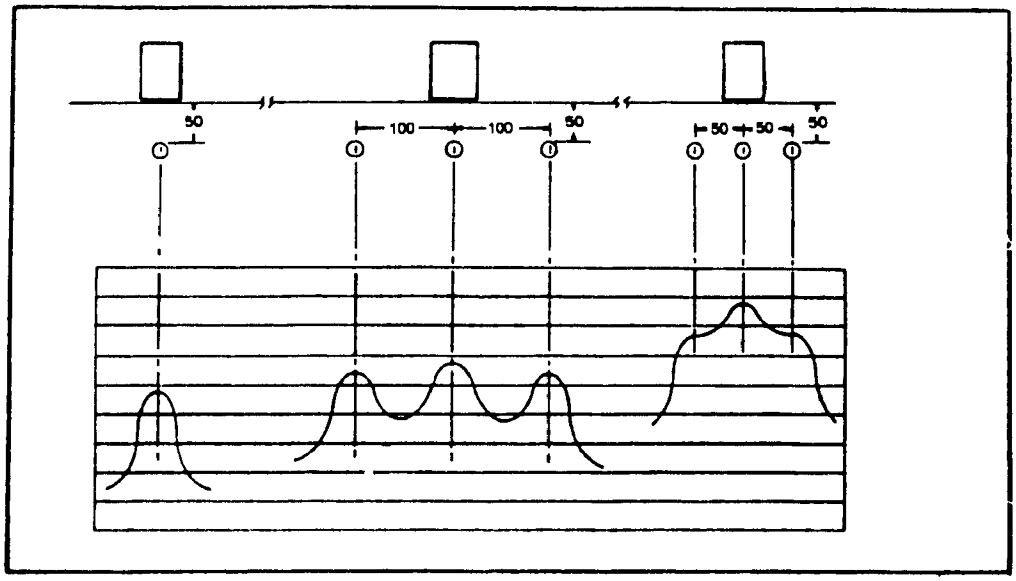


FIGURE 4.16 - EFFECT OF PARALLEL 25 MM DIAMETER REBARS LOCATED 50 MM BELOW THE SURFACE[4]

4.4.4 CARBONATION DEPTH

The corrosion of embedded steel reinforcement in concrete is affected by the pH value of the surrounding hydrated cement paste. Concrete normally provides a high degree of corrosion protection to embedded reinforcing steel due to the stabilizing effect of the high alkaline (high pH) environment. However, the passivity of the protective iron oxide film, which forms on the steel surface, can be disrupted by a reduction in the pH value of the pore fluid within the concrete. This usually occurs as a result of carbonation, or by the penetration of sufficient amounts of chloride ions.

A relatively simple means of determining the depth of carbonation, is by the use of a chemical indicator. The difference in alkalinity (or pH value) between carbonated and uncarbonated concrete is indicated by a change in color. This requires spraying the indicator on a freshly broken concrete surface by using a chisel to chip off the side of a drilled or cored hole.[5] Also, it is essential to ensure that the carbonated surface is not contaminated with dust from uncarbonated concrete. The indicator most commonly used is a solution of phenolphthalein in diluted ethyl alcohol which remains colorless for carbonated concrete and changes to purple-pink when contacting uncarbonated concrete (pH > 10).[3] Though it is not as accurate as laboratory testing, it provides a good site indication of carbonation depth.

4.4.5 CHLORIDE ION CONTENT

The effects of chloride ions on corrosion of reinforcing steel in concrete is well documented in the literature. Free chloride ions increase the electrical conductivity of moisture in the carbonated concrete, and depassivates the reinforcing steel, thus promoting corrosion. Therefore, a measure of the chloride ion content in concrete is indicative of the likelihood of corrosion activity.

The presence of chloride ions in concrete can be detected and measured in the laboratory by chemical analyses of powdered concrete samples, although some simple chemical tests have been developed in the U.K. for site use.[20,21] Powdered concrete samples are usually obtained from several depths, extending from the concrete surface to beyond the outer reinforcing steel. The samples are then dissolved in a chemical solution. Strips of special indicator paper are dipped into the solution and the height to which a color change rises gives the chloride content in percentage by mass of concrete. In order to obtain the chloride ion content in percentage by mass of cement, the cement content must be determined.[3]

Values of 0.20 and 0.40 percent chloride by mass of cement are generally taken as chloride threshold limits for prestressed and reinforced concrete structures, respectively.[22] A survey[23] by the Building Research Establishment in the U.K. has suggested that corrosion is not likely to occur if the chloride ion content of reinforced concrete is consistently less than 0.40 percent by weight of cement and highly probable if it exceeds one percent. Laboratory procedures available for determining chloride ion content in concrete include the VOLHARD method and the X-Ray Florescent Spectrometry Method.[1] The former is a relatively simple chemical test which is standardized by British Standard BS 1881 Part 124. The x-ray method requires specialized testing equipment.

The depth of carbonation affects the levels of chloride content in the concrete. For example, the chloride ion profile obtained from a coastal structure in the Middle East (Concrete-In-The-Oceans Projects) indicates that the chloride content peaked at the maximum depth of carbonation within the concrete cover rather than at the surface (Figure 4.17). According to the report, it appears that the ability of the concrete to bind chlorides is severely reduced by carbonation.[17] Theophilus and Bailey[24] discuss the importance of carbonation tests and chloride levels in durability analysis of concrete structures.

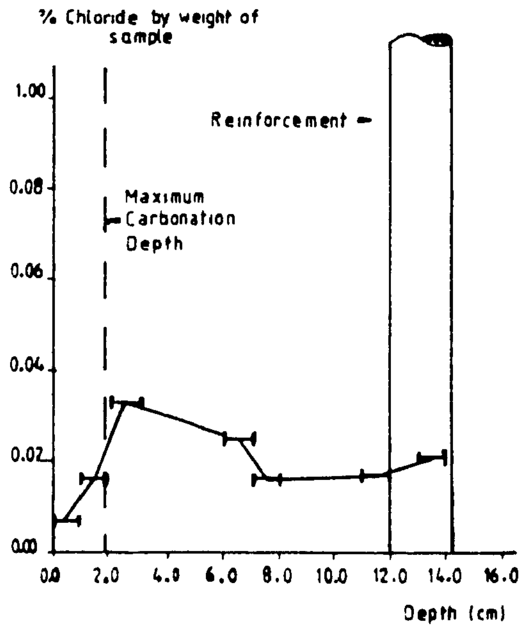


FIGURE 4.17 - CHLORIDE ION PROFILE SHOWING PEAKING CAUSED BY CARBONATION[17]

4.4.6 STEEL SAMPLING

This is a destructive test which requires removing the concrete cover to expose and visually inspect the reinforcing steel. This allows the observer to make a visual assessment of corrosion damage. Samples of the reinforcing steel may be removed for laboratory testing to determine various properties and characteristics, such as steel type, tensile strength and corrosion resistance.[5]

4.5 STRUCTURAL INTEGRITY AND PERFORMANCE

4.5.1 TAPPING TEST

The tapping test is a simple but labor intensive nondestructive method for locating delaminated concrete. It requires the investigator to strike the concrete surface at predetermined grid locations. Delaminated areas are easily detected by a dull sound. However, using a highly resonant object to strike the concrete surface may produce sounds which may make it difficult to distinguish delaminated areas from sound concrete.[1]

4.5.2 CHAIN DRAG METHOD

This method is also used for detecting delaminated concrete and requires the use of four 500 mm (20 in.) long chains, attached to a cross bar which in turn is attached to a metal rod. To perform the test, the assembly of chains is dragged over the concrete surface in a swinging motion. As with the tapping test, a distinctly different (dull) sound is generated when the chains are dragged over delaminated concrete. Currently, this method is used extensively because it has been reported to give fairly accurate results and is relatively inexpensive.[1]

4.5.3 MECHANICAL IMPEDANCE

This method is capable of detecting low density or honeycombed concrete, microcracking, and delaminations. The test involves striking the concrete surface with a small hammer containing a load-cell and monitoring the response (velocity) of the impulse with a geophone. The transducer signals are fed to a data acquisition system and processed by a PC computer. The velocity graph is divided by force to give the mechanical impedance response graph, thus providing information on dynamic stiffness, structural resonance, concrete quality and integrity.[5] The method has also been adapted to detect loss of support or voids beneath concrete pavements, floors, dam spillway linings and runways.[10] A typical impulse response method for slabs and pavements is shown in Figure 4.18.

4.5.4 IMPACT ECHO (IE) SCANNING SYSTEM

The IE method is a sonic test used for evaluating member integrity and thickness. It is a nondestructive test that only requires access to one side of the concrete member. It can be used to evaluate the integrity of slabs, walls, bridge decks, dams, tunnel linings, and parking garage decks.[10] The IE method was initially used in 1945 by Long et al.[1] and developed by Carino and Sansalone[25,26] in the early 1980s.

The IE method involves impacting the concrete at a point with a transmitter. As the pulse travels through the concrete, it is reflected by internal defects, such as honeycombing, cracks, or material of different density (Figure 4.19). A transducer "coupled" to the surface records these reflected pulses and indicates the presence of an internal defect. The transducer signals are processed through a computer and are displayed on a screen. The general shape and height of the pattern on the screen indicates the type and extent (surface area) of the defect present (Figure 4.20).[27]

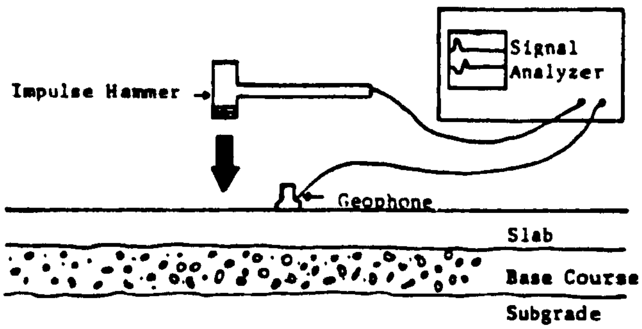


FIGURE 4.18 - IMPULSE RESPONSE METHOD FOR SLABS AND PAVEMENTS[10]

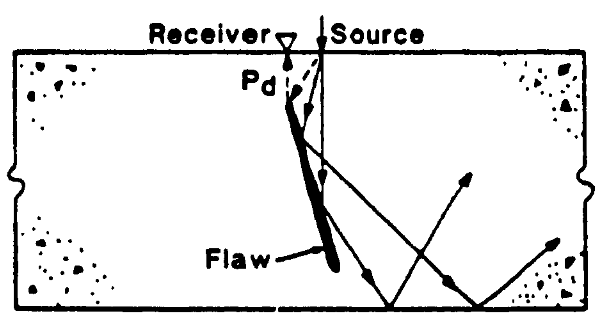


FIGURE 4.19 - SCHEMATIC DIAGRAM SHOWING HOW DIFFRACTED RAYS ARE USED TO IDENTIFY FLAWS IN CONCRETE[1]

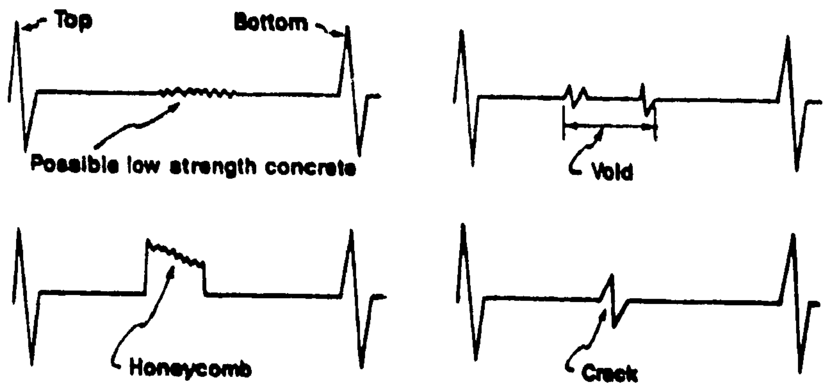


FIGURE 4.20 - THE READOUT ON THE OSCILLOSCOPE INDICATING THE TYPE OF DEFECT IN THE CONCRETE[27]

The IE scanning system is similar to the UPV scanner and uses basically the same hardware (Figure 4.21). In contrast to the UPV scanner, the IE scanner requires only a single scanner unit which incorporates both signal source and receiver. The IE scanner consists of an impulse hammer mounted on an electrically driven solenoid. The electrical impulses can be generated automatically or manually by an operator switch, which allows testing at various speeds, locations, and data densities.[9]

4.5.5 INFRARED THERMOGRAPHY

Infrared thermography uses remote sensing techniques to record the heat emission from the surface of an object. It is a diagnostic tool used extensively for assessing the condition of concrete roadways and pavements, since heat emission is affected by internal defects such as cracking and delamination. Since concrete is not a good conductor of heat, cracking and delamination will create different rates of heat transfer.[1] Delaminations are displayed as well-defined white colored areas on the infrared thermogram as opposed to a "mottled grey-white color" that is produced by sound concrete.

Heat radiation from the sun can help to produce a more noticeable contrast between sound and unsound concrete. For instance, a difference in temperature of about 1.5°C (2.7°F) will create a clear contrast between the two. Thermographic scanners currently in use can detect a temperature difference of up to 0.2°C (0.36°F). However, a greater accuracy is required to compensate for different heat emission rates due to surface finish and debris.[1]

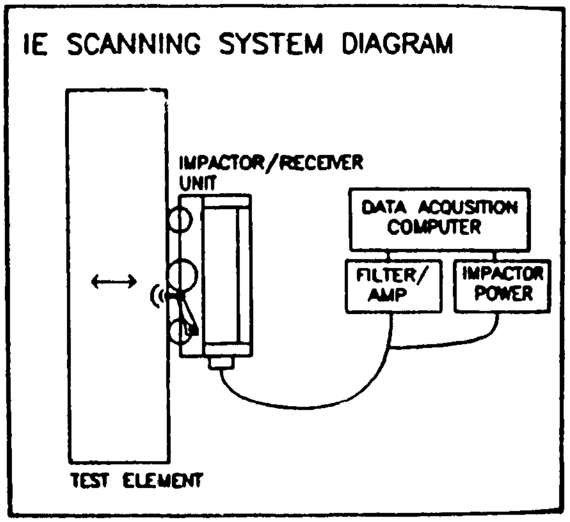


FIGURE 4.21 - IMPACT ECHO SCANNER SYSTEM[9]

4.5.6 IMPACT VIBRATION TEST

A quantitative evaluation of the structural integrity of a structure, such as a bridge pier or footing, can be obtained by determining its eigenfrequency (dynamic response) when subjected to an impact vibration test. Since the state of a structure (condition of concrete, condition of bearing stratum which supports the concrete structure, etc.) affects its eigenfrequency, the integrity of the structure can be judged by comparing the measured eigenfrequency with an established standard value.[28]

The test, which was recently developed by the Railway Technical Research Institute (RTRI) in Japan,[29] involves applying an impact load to the pier by means of a 30 kg (13 lb.) weight suspended from the girder, and its responses (displacement and acceleration) are measured

(Figure 4.22). The weight can be separated into several sections to facilitate its transportation. A data acquisition system records and processes the responses to produce the measured eigenfrequency of the structure.

A judgement of the structural integrity is made by a determination of the "index of integrity" which is obtained by dividing the measured eigenfrequency by the standard value of eigenfrequency The standard value for the eigenfrequency of the pier in the direction perpendicular to the bridge axis can be obtained by empirical formulae. For example, the standard value for the eigenfrequency of a pier on a spread footing can be obtained by the formulae shown in Table 4.8. The index obtained is then compared to the values in Table 4.9 for judgement of integrity.

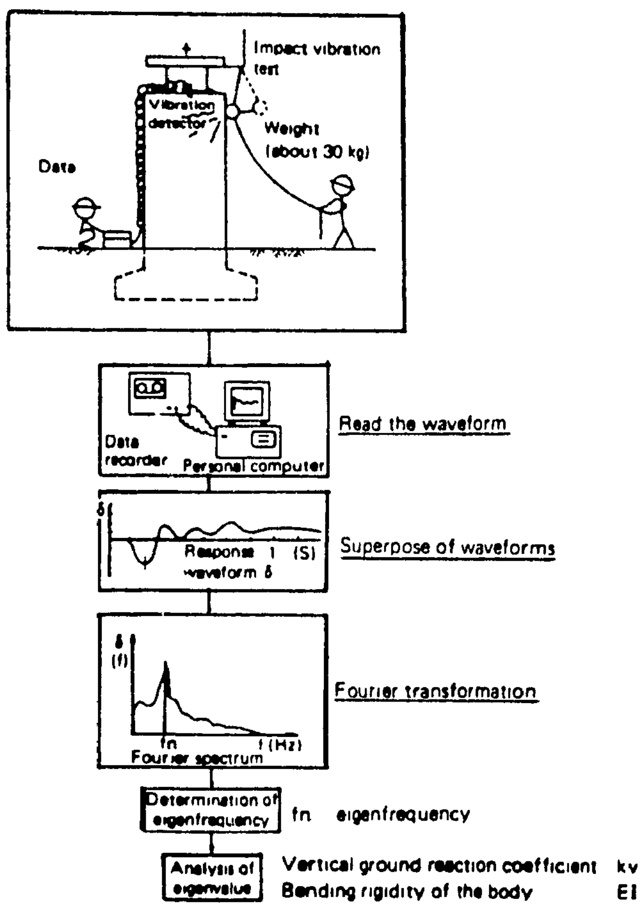


FIGURE 4.22 - METHOD OF IMPACT VIBRATION TEST[28]

TABLE 4.8 - FORMULA TO OBTAIN A STANDARD VALUE OF EIGENFREQUENCY OF PIER (AT 90° TO BRIDGE AXIS) (RTRI)[28]

Type of foundation	Classification of ground	Formula to obtain standard value (F) for eigenfrequency
Spread foundation	Analytic ground	Hd: Height of pier earth covering (m) Wh: Weight of girder (tf)
	100 or below 101 ~ 300 300 or above	$F = 25.4Hd^{-0.47} Wh^{0.11}$ [Hz] $F = 49.0Hd^{0.47} Wh^{0.24}$ [Hz] $F = 83.7Hd^{-0.71} Wh^{0.20}$ [Hz]

TABLE 4.9 - CRITERIA FOR JUDGEMENT OF PIER INTEGRITY (RTRI)[28]

Index of integrity	Judgement ranks		Measures
Below 0.70	A	(A1)	Precise inspection should be made and adequate measures should be considered
Below 0.85		(A2)	Monitoring progress of change in state such as inclination and scour
Above 0.86	B		Present state poses little problem and is considered to be satisfactory

4.5.7 STATIC LOAD TESTING (PROOF LOADING)

Static load testing is usually performed as a means for evaluating the load carrying capacity and performance of a structure, or component of the structures. Full scale load tests have been developed for a variety of structures including, beams and girders (ASTM E 529), cladding components (ASTM E 997, E 998), roofs (ASTM E 196, E 695), truss assemblies (ASTM E 73, E 1080), and piles (ASTM D 3966, D 1143). The loading is usually applied by hydraulic jacks, mechanical jacks, air pressure, or other heavy materials. Prior to conducting the test, the component or area of the structure being tested is isolated from the structure to obtain an accurate response.[30]

During static load testing, two types of responses are measured: deflection and strain. Deflections are usually measured by deflection transducers, deflecting dial gauges, and high precision levels or laser equipment. Strains are measured by the use of standard electronic strain gauges, electronic displacement transducers, accelerometers, and pressure transducers.[30] Strains are either recorded manually, with a portable strain indicator, or automatically by a data acquisition system.[1] Following each load test, a detailed examination (i e., crack survey) of the structure or component should be conducted [13] Although very expensive, these tests are very informative.

4.6 LABORATORY TESTING METHODS

There are numerous laboratory test methods for determining the cause or causes of deterioration in concrete, and many others for determining its composition. When ordering a specific laboratory test, it is important to understand the intent and purpose of the test which is being conducted and its significance to the investigation before it is performed. The relevant parameters that may cause test results to vary from in-situ conditions must be clearly understood.[30] It is not the intent of this section to describe all these tests, but rather to describe some of the types most commonly used when investigating concrete deterioration.

4.6.1 PETROGRAPHIC EXAMINATION

Petrographic analysis uses microscope techniques to determine the concrete composition, concrete quality, and the cause or causes of distress or deterioration. This test procedure, which is standardized by ASTM C 856, was originally developed to describe and classify rocks and has been adapted to include hardened concrete, mortar, grout, portland cement, and other construction materials.[44] The analysis is typically performed on 25 mm (1 in.) diameter specimens obtained from the site by core drilling.

Petrographic analysis can also be used for estimating future durability and structural safety of concrete elements. For example, some of the items that can be evaluated by a petrographic analysis include cement paste, aggregate, mineral admixture, and air content; frost and sulfate attack; alkali-aggregate reactivity; degree of hydration and carbonation; water-cement ratio; bleeding characteristics; fire damage; scaling; popouts, and several other aspects.[31]

4.6.2 X-RAY DIFFRACTION

This method is used to determine the detailed nature of the "strength-conferring agents" including the presence and distribution of "relict" cement grains, and the extent and location of carbonation in portland cement concrete.[8] The method involves x-ray examination of a paste concentrate made by breaking up some of the concrete. The mortar is removed from the aggregate and is sieved over a 150μ (Nº 100) sieve and the material passing the sieve is ground to pass the 45μ (Nº 325) sieve, placed in a holder, and scanned on a diffractometer. Useful information for interpreting x-ray charts of hydrated portland cement is found in References 32, 33 and 34.

4.6.3 CEMENT CONTENT

Cement content tests are valuable for determining the cause of strength loss or pore durability of concrete.[31] Cement content can be determined by ASTM C 85 and C 1084 standard methods or by the maleic acid or other nonstandard procedures.[35,36] The standard method is used to determine the amount of calcium oxide and soluble silica content in the concrete by performing an oxide analysis. The cement content is then computed from each component through the use of a mathematical relationship. The cement content is taken as the average of the computed values from each component, provided they are within one percent (or 25 kg/m^3) of each other. When the two computed values are not within these limits, the lower value is taken as the cement content.[1]

4.6.4 SULFATE CONTENT

Sufficient amounts of sulfates in hardened cement paste can lead to sulfate attack, causing expansion and disruption of concrete. Sulfates can penetrate the concrete from exposure to seawater or seawater spray, mix water, chloride-containing admixtures, or deicing salts. The sulfate content or sulfate attack in concrete can be identified using chemical analyses of field specimens. The concrete sample is broken up, weighed, and dispersed in a solution of water and hydrochloric acid. It is subsequently boiled and filtered, and methyl red indicator is added. The solution is then neutralized by adding ammonium hydroxide, hydrochloric acid, and barium chloride. Next, the sample is boiled again and maintained for a period of 30 minutes, after which the sample is left to stand for 12 to 24 hours. Desiccation of the sample will precipitate a mass of barium sulfate which is then weighed. The sulfate content (expressed as a percentage of cement content) is computed through the use of a mathematical expression relating sulfate content to the mass of barium sulfate produced. If the sulfate content is in excess of three percent, chemical attack is likely to occur.[1]

4.6.5 AIR CONTENT

The amount of air voids in hardened concrete significantly affects its permeability, hence its resistance to various deleterious attack mechanisms, such as freeze-thaw action, sulfate attack, and penetration of deicing salts. Accordingly, ACI 318-92[37] has set forth minimum air content values for various environmental exposure conditions. The various types of air voids that exist in hardened concrete include: micropores or gel pores, capillary pores, and macropores (Figure 4.23). The capillary pores and macropores are those most relevant to concrete durability.[38] The micropores are those formed within the interparticle spaces in the calcium silicate hydrate and its total volume is considered to be too small to have an adverse effect on the durability of concrete.[1]

The air content and air-void system characteristics of hardened concrete can be determined by ASTM C 457 test procedures. There are three standard test methods which are normally employed to determine air void content: Linear Traverse (Rosiwal) Method, Point Count Method and Modified Point Count Method.[1] The tests typically involve microscopic examination of concrete samples removed from the structure. The information obtained from these tests also include the volume of entrained air, its specific surface, paste content, and spacing factor.[31]

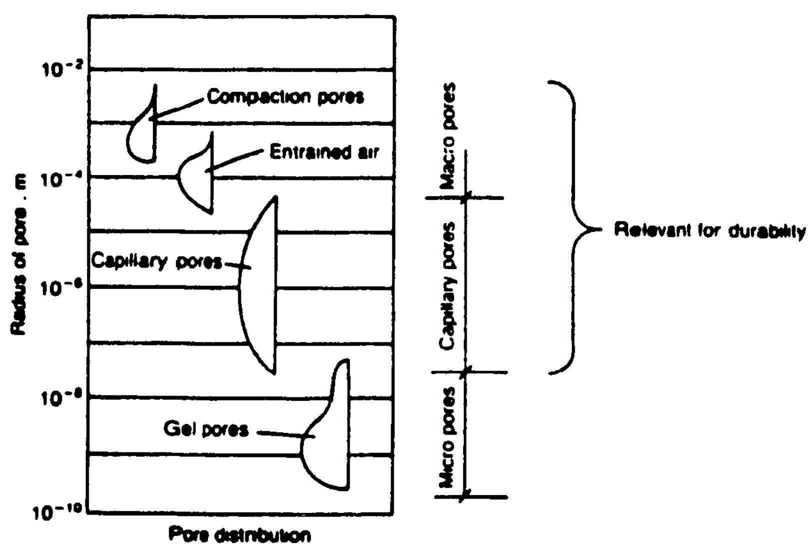


FIGURE 4.23 - PORE SIZE DISTRIBUTION IN CONCRETE[38]

4.7 PASSIVE MONITORING AND INSTRUMENTATION

For any repair solution to be effective, the factor or factors causing distress or deterioration in the concrete must be clearly understood. This often includes knowing if any cracks in the structure are still moving as load or temperature changes. This information can most often be obtained by periodic visual inspection or by employing passive monitoring methods. These include devices such as crack monitoring devices and other movement measurement instruments. These are discussed below and are mainly adapted from a review of Reference 5.

4.7.1 TELL-TALE PLATES

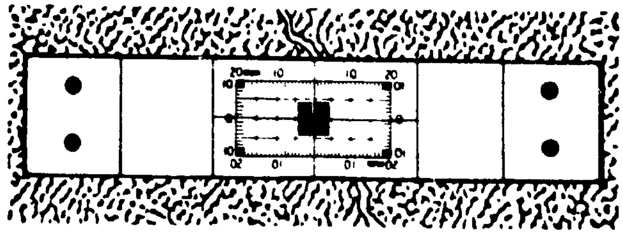
This device is the simplest of all the crack monitors and consists of thin glass plates which are glued across the crack. Any subsequent crack movement will break the glass.[5] Although, this method is easy to use, it does not provide an indication of the extent and direction of crack movement. A more sophisticated instrument is the Avongard crack monitor which gives a direct reading of crack displacement and rotation (Figure 4.24).[39] The unit consists of two acrylic plates; one is etched with a fine grid pattern and the other is marked with a cross-hair. The plates are glued on both sides of the crack, with the cross-hairs overlapping and the grid centered on the crack. Any movement of the crack can be measured from the position of the cross-hairs on the grid.[5]

4.7.2 MECHANICAL CRACK INDICATOR

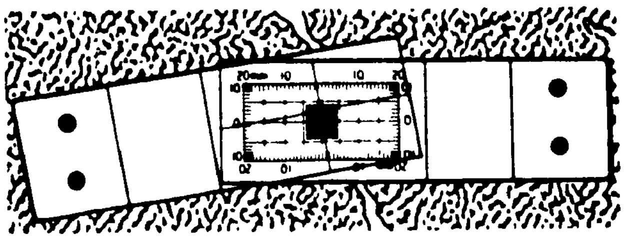
Crack movement can also be monitored and amplified (50 times) by the use of a mechanical crack indicator, shown in Figure 4.25. This device has the advantage of indicating the maximum range of movement occurring during the monitoring period.[39]

4.7.3 STRAIN GAGES

Strain gages can be either electrical or mechanical, and are used to monitor slow movements of cracks caused by load and temperature changes.[5] To obtain a more detailed time history of the crack movement, a wide range of transducers (linear and rotary potentiometers, and LVDTs) and data acquisition systems (strip chart recorders or computer based) are available.[30,39]



Newly Mounted Monitor



Monitor After Crack Movement

FIGURE 4.24 - CRACK MONITOR[39]

4.7.4 ACOUSTIC EMISSION

Acoustic emission is used to monitor and detect very small movements near cracks or voids within a concrete structure. These movements produce acoustic sound which can be monitored and recorded by sensors attached to the concrete surface. Once a change occurs in the structure, such as an increase in crack length, a corresponding change in stress will create a different acoustic sound.[5]

4.7.5 SETTLEMENT MONITORING

If it is suspected that settlement of the structure may have caused cracking, more sophisticated devices are available for monitoring movement of earth near or below the foundation. Devices, such as extensometers or inclinometers, are used to monitor the movement of earth over time to determine if settlement is still occurring. If this problem is not corrected, further settlement will continue to cause cracking in the structure, which in turn will make any repair ineffective.

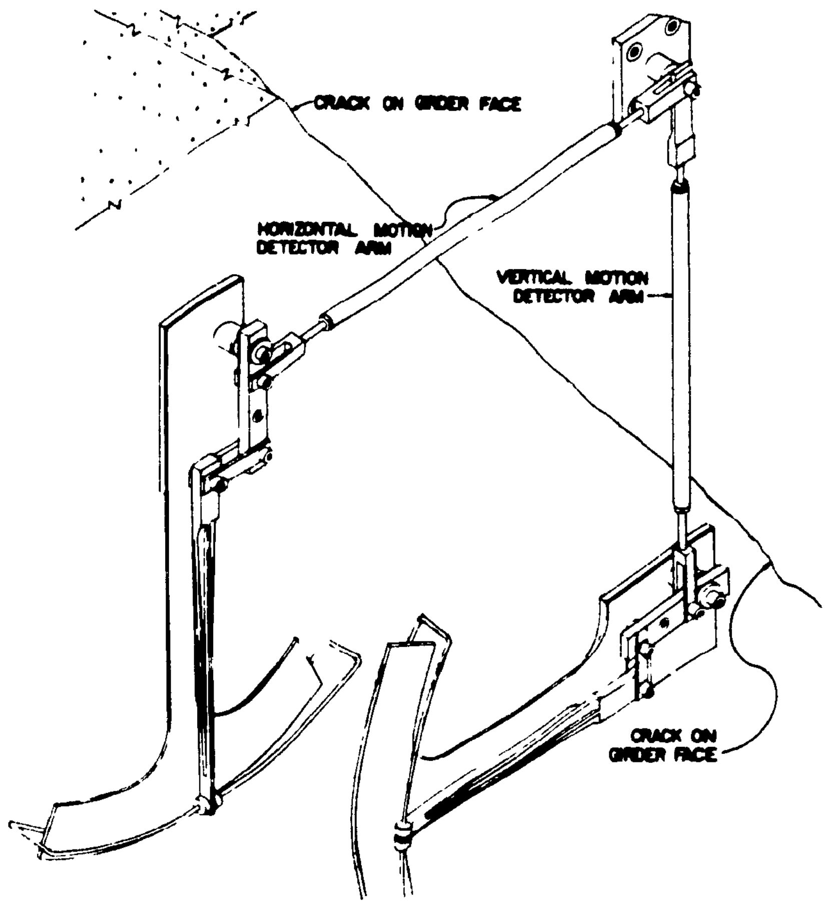


FIGURE 4.25 - MECHANICAL CRACK MOVEMENT INDICATOR[39]

4.8 UNDERWATER ACOUSTIC INSPECTION METHODS

The integrity of concrete structures in seawater depends mainly on the presence of surface cracks, which in many cases, lead to corrosion of the reinforcement and serious deterioration to the concrete. Therefore, one of the concerns of underwater inspection is to detect cracks. Traditional inspection methods can only detect damage when the corrosion process has developed extensive cracking which can be visually detected by divers.

Acoustic inspection techniques do not require divers, can be used in low-visibility conditions, and can perform inspections through layers of sediment or soft marine growth. There are two modes of acoustic inspection techniques: sonar or echo sounding from the surface or by a towed underwater vehicle, or ultrasonics, "a local high-resolution underwater acoustic system."[40] The side-scan sonar is a good method for mapping the general conditions of large areas, such as stilling basin floors.[41] Ultrasonics is useful for determining concrete deterioration when cracking, spalling and pitting has occurred. Conventional ultrasonic methods use bulk sound waves which are scattered by the aggregates in the concrete. Therefore, ultrasonic techniques that do not penetrate deeply into the concrete are more effective. Two such techniques are the leaky Rayleigh wave and acoustic microscopy methods.[40,42,43] To apply these methods remotely, a hydraulic manipulator has been developed which can be mounted on ROVs. The basic principles governing the operation of these two techniques are summarized below.

4.8.1 LEAKY RAYLEIGH WAVE METHOD

This method employs Rayleigh waves for detecting and measuring crack depth. By directing an ultrasonic beam to the concrete surface at a specific angle, a Rayleigh wave is generated and propagates along the surface of the concrete. The presence of a surface crack will affect the Rayleigh wave propagation and split it into various components: a reflected Rayleigh wave, a longitudinal body wave, and a Rayleigh wave which travels down along the crack wall (Figure 4.26). When the wave reaches the crack root, diffraction occurs and a Rayleigh wave travels upward along the far side of the crack and out towards the receiver. A wave is also reflected directly to the receiving transducer. These waves do not penetrate deeply into the structure and will only approximate the location and depth of the crack.

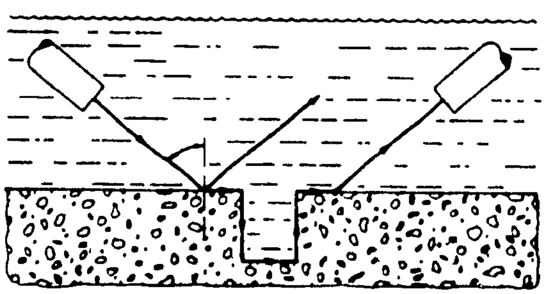


FIGURE 4.26 - SCHEMATIC REPRESENTATION OF A LEAKY RAYLEIGH WAVE MEASURING SET-UP[43]

4.8.2 ACOUSTIC MICROSCOPY

Acoustic microscopy is used in conjunction with the leaky Rayleigh wave method to determine the width of the crack and its trajectory. This involves scanning a concrete surface with a transducer which emits a highly focussed ultrasound beam (Figure 4.27). The reflected image which is produced is similar to a photograph, except it is less affected by turbid water, soft marine growth, and loose debris in the crack.

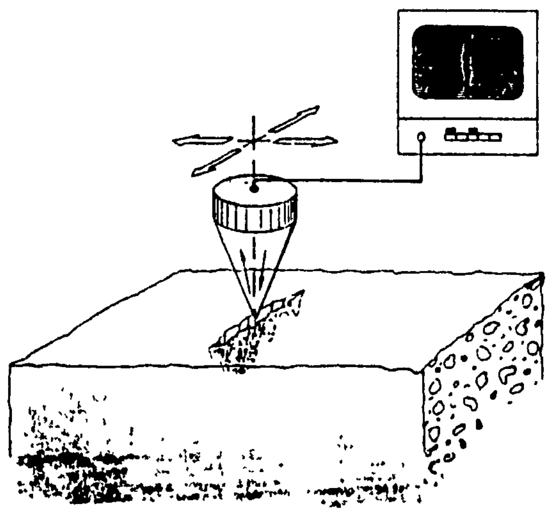


FIGURE 4.27 - SCHEMATIC REPRESENTATION OF AN ACOUSTIC MICROSCOPY MEASUREMENT[43]

In scanning the surface, the beam focal point is located on the concrete surface and reflected back again to the same transducer. A short pulse signal is emitted immediately after transmitting the beam so that it can use the transducer as a receiver. By scanning the surface along closely spaced parallel lines, an image of the concrete surface can be constructed. The width and length of the focal point depend on the transducer diameter, curvature (focal distance), and frequency of the ultrasound used.

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CHAPTER 5
UNDERWATER REPAIR TECHNIQUES

5.1 INTRODUCTION

The best quality repair of underwater concrete structures can be performed in dry conditions after dewatering the structure. This can be usually accomplished by pumping the water from a steel sheet pile enclosure or cofferdam built around the structure, or portion of the structure being repaired. However, in some cases, dewatering is impossible, expensive and often politically sensitive. The U.S. Army Corps of Engineers reports that dewatering costs associated with the repair of the underwater portions of concrete hydraulic structures average approximately 40 percent of the total repair costs.[1] In these cases, it becomes necessary to repair or place concrete in submerged conditions.

Many of the techniques available for above-water repairs can be easily adapted for use under water. However, materials used for dry repairs often cannot be used under water.[2] As a result, there has been considerable effort in the past 15 years by government agencies and specialist contractors toward developing effective and affordable techniques for placing concrete under water.[3]

The major factors which must be considered when developing an underwater repair scheme are summarized below:[2]

- The cost of carrying out underwater repairs is much greater than for similar repairs performed in dry conditions. The work carried out at the site should be as simple as possible.
- Surface preparation of the damaged concrete requires special techniques to ensure that the repair surface is not contaminated before placing the repair concrete.
- The repair material used must be able to cure under water.
- Special formwork and placement techniques must be considered to prevent or minimize mixing between the repair concrete and water.

The cause and extent of deterioration established during the condition evaluation of the structure, site logistics, and cost will dictate the method of repair. The repair technique selected must be designed to suit the specific site conditions and meet the client's needs and budget constraints. In some cases, it is necessary to perform laboratory and site tests on both repair methods and materials to identify potential problem areas. No one technique will be most efficient and cost effective for all underwater repair jobs. The material in this chapter has been adapted from a review of different available references, especially 2, 12 and 38.

5.2 PREPARATION OF DAMAGED AREAS

Before a repair operation is performed, the damaged area of the structure must be cleaned to allow a detailed inspection by divers or ROVs. This is necessary so that an accurate assessment of the damage can be made and an effective repair program can be prepared. The first step in the repair will be to remove all loose and unsound concrete, and severely deteriorated or distorted reinforcing steel.

Removing concrete and cutting reinforcing steel underwater is more complex than performing the same tasks above water. The nature of the underwater work will often dictate the selection of cutting equipment. For example, the thermic lance (Section 5.2.2.5) is capable of cutting reinforcing steel and concrete at the same time, while high-pressure water jets can be used to remove only concrete.[2] The following sections outline the various techniques and equipment most commonly used for preparing underwater concrete surfaces for repair and are adapted from References 2, 4 and 5.

5.2.1 SURFACE CLEANING

Underwater cleaning is often necessary to remove marine growth to facilitate the inspection and to be able to define the extent of damage. It is also required to ensure a good bond between the substrate and the repair concrete. Typically, the repair concrete, type and amount of marine growth, and accessibility of the concrete surface serve as a guide for selecting the proper cleaning equipment.[6] Hand-held or mechanical wire brushes, needle guns or scabbling tools are good for cleaning small areas while for large areas, a high-pressure water jet will be more effective. If the marine growth present is hard or the concrete surface is contaminated with oil, adding an abrasive slurry or detergent to the water jet will improve the cutting ability of the tool.[2]

5.2.2 REMOVAL OF DAMAGED CONCRETE

Once the area and extent of damage has been defined, the cracked and deteriorated concrete can then be removed. The method selected must ensure that the remaining concrete and reinforcing steel is not damaged. The following is a summary of techniques which can be used to remove concrete and reinforcing steel.

5.2.2.1 HIGH-PRESSURE WATER JET

This method is used extensively for performing underwater work. The high-pressure water tool uses a thin jet of water driven at high velocities to remove the hardened cement paste mortar. The system operates in the same way as that used for cleaning concrete surfaces, except that water is delivered at much higher pressures (typically between 5000 and 30,000 psi).[2] The diameter of the nozzle orifice, and the water pressure at the nozzle determine the flow rate of the jet. The depth of the cut mainly depends on the number of times the water jet is passed over the surface of the concrete.[6] The water jet can either be frame-mounted and automated, or portable. If properly used, the water jet can be used to cut irregular shapes with minimal damage to the remaining concrete and steel reinforcement.[4] However, a high-pressure water jet is potentially dangerous as it will easily remove bone and muscle.[7,8] Therefore, it should be used only by experienced divers or operators.

5.2.2.2 CONCRETE SPLITTER

The concrete splitter is a pneumatic or hydraulic expansive device that is used to break concrete into sections. Expanding cylinders are inserted into drilled boreholes along a predetermined plane and pressurized until splitting occurs.[2,4] The device, shown in Figure 5.1, consists of a hydraulic system and a splitter. It contains a plug, feathers, cylinders, a piston, a commanding valve, and a control lever. Several splitters can be used simultaneously with one hydraulic system.

The pattern, spacing, and depth of the holes the orientation of the feathers and the number of splitters will determine the direction and extent of crack planes that develop. The spacing of the holes is determined on the basis of the percentage of steel reinforcement present in the concrete. The diameter of the holes range from 30 mm (1-3/16 in.) to 45 mm (1-¾ in.) and minimum hole depths range from 300 mm (12 in.) to 660 mm (26 in.), depending on the type of splitter used.[4]

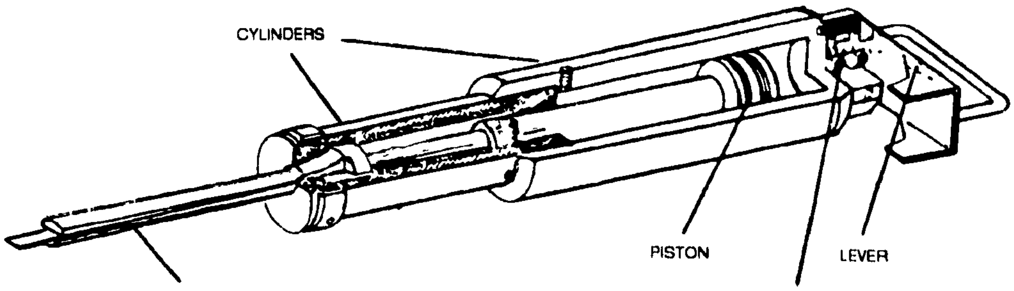


FIGURE 5.1 - CUTAWAY VIEW OF A TYPICAL HYDRAULIC SPLITTER[4]

The advantage of using the concrete splitter is that it can be used to pre-split large sections of concrete for removal. The splitter is safe to use and limited skill is required by the operator or diver. The main disadvantage is that the depth to which it can remove concrete from mass structures is limited. Also, secondary methods of removal are often required to complete the work[4]

5.2.2.3 EXPANSIVE AGENT

Recent developments have demonstrated that removing concrete with expansive agents can be less costly and as effective as using the concrete splitter. The agent (or cement) is mixed to a slurry form with water and poured into plastic bags. The bags are then placed into pre-drilled boreholes within the concrete by divers. As the slurry solidifies and expands, it produces tensile stresses (as much as 30 MPa) that generally exceed the tensile strength of concrete. Cracking of the concrete will begin to propagate out from the hole over the next 12 to 24 hour period and may continue for a couple of days before stopping. Secondary means of breaking the concrete are also usually required with this method to complete the removal.

Expansive cements are relatively safe to use and require limited installation skills. When the boreholes are located in areas exposed to ambient temperatures, the agent may freeze (during cold weather) or overheat (during summer months), causing it to loose its effectiveness. Its main disadvantage is it could take a couple of days before pre-splitting becomes optimum.[4]

5.2.2.4 HIGH-PRESSURE CARBON DIOXIDE BLASTER (CARDOX SYSTEM)

The carbon dioxide blaster is a blasting device which uses pressurized carbon dioxide gas to breakup large masses of material. It was first used in 1930 for breaking coal, and has since been reportedly used for breaking concrete, rock, and stone. The blasting device consists of a reusable cartridge, switch, electrical cable and power supply. The cartridge, shown in Figure 5.2, is a hollow steel tube containing pressurized carbon dioxide and is fitted with a firing head screwed on at one end and a discharge head screwed on at the other end. The cartridges are placed and caulked firmly into pre-drilled holes at predetermined spacings. The cartridges are then electrically detonated, producing a mild explosion which breaks the concrete apart. The explosion causes minor damage to the remaining concrete.[2,4]

The diameter of the boreholes range between 55 mm (2-¼ in.) and 75 mm (3 in.), and are drilled approximately 3 mm (⅛ in.) larger than the size of the cartridge being used. Only one cartridge per hole is recommended. Since the technique is potentially hazardous, highly skilled personnel are required for blast design and execution of blast design.[4]

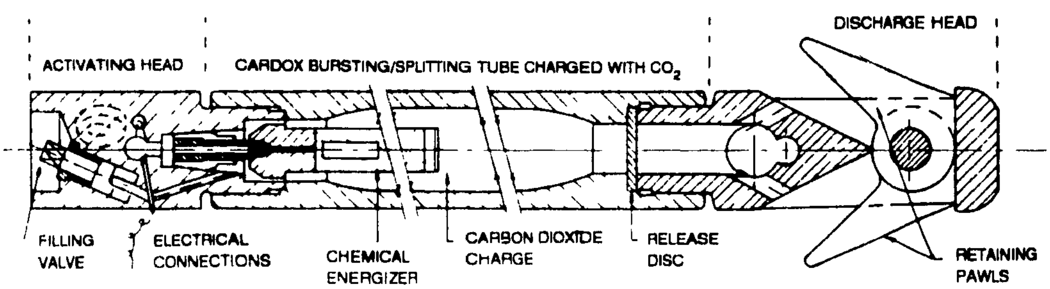


FIGURE 5.2 - CUTAWAY VIEW OF THE CARDOX SYSTEM[2]

5.2.2.5 THERMAL LANCE

The thermal lance is the simplest and most commonly used method of cutting. It uses intense heat generated by the reaction between oxygen and mild steel rods to cut through concrete. The system consists of a long steel tube filled with mild steel rods, a flexible pressure hose, an oxygen supply, and an acetylene or propane supply tank. Oxygen is forced through the tube and ignited by the oxygen-acetylene or propane flame. The resulting reaction creates temperatures up to 3500°C, which allows the tip of the lance to melt the concrete or reinforcing steel. The resulting cut is approximately 40 to 50 mm (1-½ to 2 in.) wide and advances at an approximate rate of 215 mm^2/sec (0.33 in^2/sec). The higher the percentage of steel reinforcement in the concrete, the faster the cutting rate.[24] Underwater cutting rates decrease sharply with increasing water depth and are usually limited to relatively shallow depths (60 m maximum) [2]

5.2.2.6 MECHANICAL CUTTING

Mechanical cutting tools for underwater work have been used extensively for many years. Of the many underwater mechanical cutting tools available, the ones most often used are the hydraulically powered diamond tipped rotary abrasive saw and chipping hammer. The rotary abrasive saw is considered the most useful because it can cut concrete and reinforcing steel simultaneously. However, this type of cutting tool is relatively slow and can only be used to cut to a limited depth, depending on the saw blade diameter.[2] The chipping hammer is very useful for removing deteriorated concrete cover to expose steel reinforcement that is to be reused in the repair. Other underwater cutting tools include hydraulically-operated drills used for drilling small diameter holes or taking concrete cores. Conventional pneumatic breakers and saws are limited to water depths of about 6 to 9 m (20 to 30 ft.). However, recent developments using a "closed hydraulic system" have been successfully employed to greater depths. Diver-operated chipping hammers using an oil compression system are not depth sensitive and can be used to a depth of about 6100 m (20,000 ft.).[5]

5.2.3 CUTTING OF REINFORCEMENT

Reinforcing steel which is badly corroded or damaged must be removed and replaced before applying the repair concrete. This will require the use of underwater steel cutting techniques. Many underwater steel cutting methods have been used in the past and are similar to those used on land. Selection of the actual method used depends on how deep the work is, and on the available equipment and fuel (or power).[5] All of the known thermal cutting processes and their advantages are listed in Tables 5.1 and 5.2, respectively.

The methods most commonly used for cutting steel reinforcement under water are oxy-fuel (acetylene or hydrogen), oxy-arc, or mechanical cutting. Oxy-arc is the most widely used underwater cutting technique with oxy-fuel cutting being used only in special applications. New techniques have also been used, either on an experimental basis or for performing actual underwater cutting work. Underwater oxygen cutting requires prior removal of heavy marine growth, scale and surface rust.[5]

TABLE 5.1 - THERMAL PROCESSES USED FOR UNDERWATER CUTTING[5]

Oxy-Fuel Gas Cutting Processes
Oxy-acetylene
Oxy-gasoline
Oxy-natural gas
Oxy-naptha
Oxygen-MAPP[a] gas
Electric Cutting Processes
Air-carbon arc
Bare-metal arc
Carbon arc
Gas-metal arc
Metal arc
Oxygen arc
Plasma arc
Shielded-metal arc
Other Cutting Processes
Liquid oxidizer-liquid fuel (chlorine trifluoride-hydrazine)[b]
Oxygen lance
Pyrotechnic torch

[a]Trade designation of Dow Chemical Company
[b]Experimental use only

5.2.3.1 OXY-FUEL GAS CUTTING

These underwater oxygen cutting techniques use intense heat to melt the steel being cut by a process known as "burning".[2] The steel is preheated to its melting temperature and then a high-velocity stream of oxygen is directed at the preheated metal to produce the cut. The cut is produced as a result of a chemical reaction between iron and oxygen. The molten metal is blown away by the oxygen stream.

The gases used for underwater cutting with oxy-fuel techniques are the same as those used in air (acetylene and hydrogen). However, due to the instability of acetylene at pressures over about

103 kPa (15 psi), it is not used at depths greater than approximately 10 m (32.8 ft.). Therefore, only hydrogen is generally used for underwater cutting. Stabilized methyl-acetylene propadiene (MAPP), has been used to a limited extent. Propane and natural gas have also been used for underwater cutting, but are not as effective. Oxygen-fuel gas techniques are generally used when electric currents, produced by the oxy-arc system, can cause electrolysis, spark formation, or electrocution.[5] A typical gas cutting torch tip is shown in Figure 5.3.

TABLE 5.2 - ADVANTAGES OF VARIOUS CUTTING PROCESSES[5]

Oxygen-Arc Process, Tubular Steel Cutting Electrodes
- Preheating is not required
- Flame adjustments are unnecessary
- Applicable to all metal thicknesses
- Overlapped plates can be cut
- Only one gas (oxygen) is needed
- Torches are lightweight
- Less training and skill are required
- Higher cutting rates on thin metal

Oxygen-Arc Process, Ceramic Cutting Electrodes
- Low burnoff rate, long life
- Short length provides easier access in confined spaces
- Light weight improves transportability

Shielded Metal-Arc Process
- Preheating is not required
- Cuts ferrous and nonferrous metals
- Fuel gases and oxygen are not required
- Standard electrode holders can be used in an emergency if properly adapted

Oxy-Hydrogen Process
- Electricity is not required for cutting
- Nonmetallic materials can be severed
- Insulated diving equipment is unnecessary
- Power generators are not required
- There are no ground connections
- Higher cutting rates on thick metal

Oxy-acetylene
- High-flame temperature
- Electricity is not required for cutting
- Insulated diving equipment is unnecessary
- Power generators are not required
- Nonmetallic materials can be severed

Plasma-Arc
- Potentially high cutting rates
- Fuel gases and oxygen are not required
- Cuts ferrous and nonferrous materials

TABLE 5.2 - ADVANTAGES OF VARIOUS CUTTING PROCESSES[5] (Continued)

Pyrotechnics	
	High cutting rate
	Cuts ferrous and nonferrous metals
	Fuel gases and oxygen are not required
Explosives	
	Multiple cuts can be made simultaneously
	High cutting rates
	Fuel gases and oxygen are not required
	Electricity is not required

5.2.3.2 OXY-ARC CUTTING

Oxygen-arc cutting is similar to oxygen-fuel gas cutting except that an electric arc is used to preheat the steel instead of oxy-fuel gas flames. A high velocity stream of oxygen is forced through an electrode to jet away the molten metal. Underwater oxy-arc cutting can cut steel thicknesses ranging from sheet gages to about 75 mm (3 in.).[5]

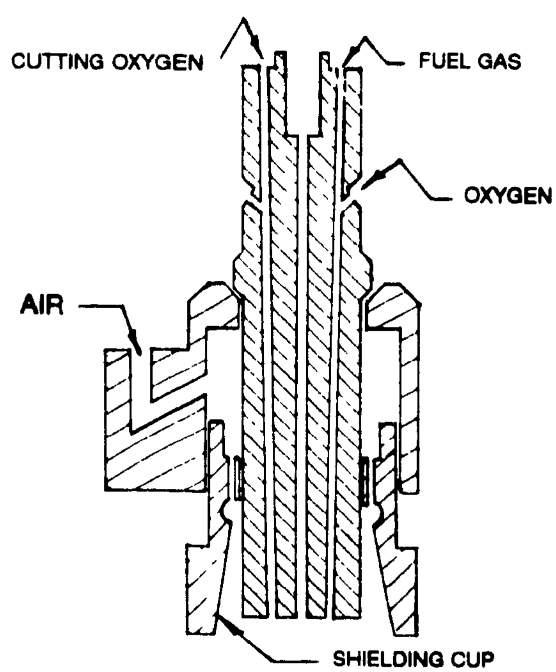


FIGURE 5.3 - TYPICAL UNDERWATER GAS CUTTING TORCH TIP[5]

Steel tubular electrodes, ceramic tubular electrodes, and carbon-graphite electrodes have all been used for underwater cutting. The steel tubular electrode was specifically developed for underwater cutting and is the most commonly used electrode. The main disadvantages of using steel electrodes are its short life and narrow "kerf". The narrow kerf makes it difficult for the diver to look for incomplete cuts. These limitations can be overcome by using ceramic tubular electrodes, but these are brittle and expensive. Carbon-graphite electrodes are also brittle.[5] A typical thermic cutting torch and steel tubular electrode are shown in Figure 5.4.

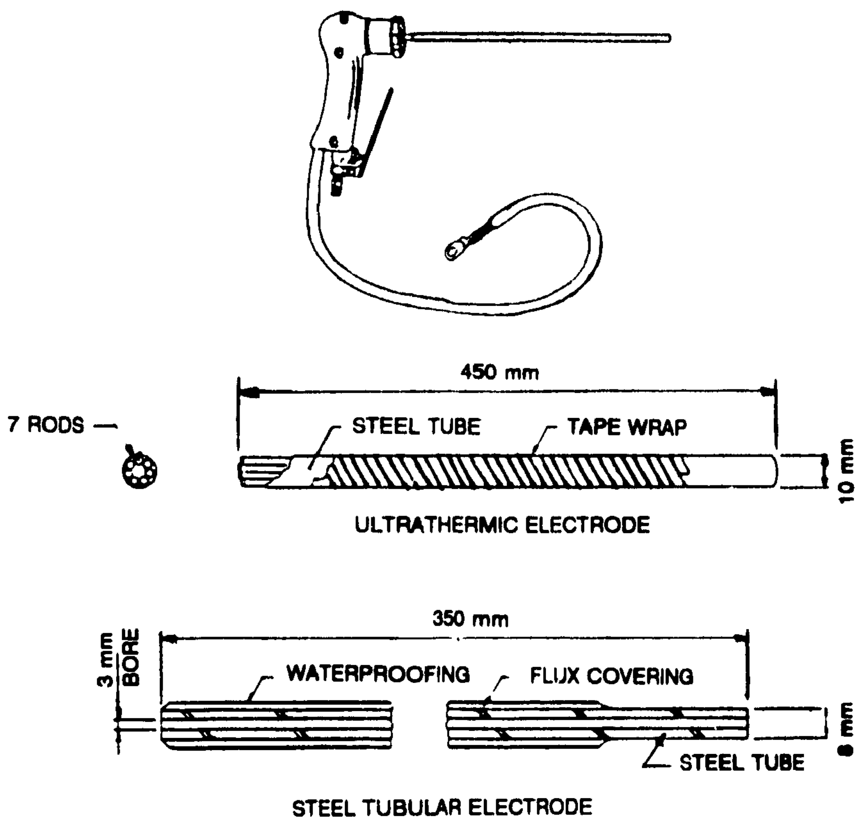


FIGURE 5.4 - UNDERWATER THERMIC CUTTING TORCH AND OXY-ARC CUTTING ELECTRODES[9]

5.2.3.3 SHIELDED METAL-ARC CUTTING

Shielded metal-arc cutting is similar to oxy-arc cutting and can be done with virtually any kind of mild steel welding electrode provided it is properly waterproofed. However, cutting rates with the shielded metal-arc system are much lower than those attainable with oxy-arc cutting. The shielded metal-arc system is especially effective for cutting cast iron and nonferrous materials.[5]

5.2.3.4 MECHANICAL CUTTING

Mechanical cutting methods are usually employed when only a limited number of small diameter reinforcing bars need to be cut. The most commonly used machines are usually hydraulically-operated diamond-tipped rotary saws. Hand-operated tools, such as bolt croppers, have also been used.[2]

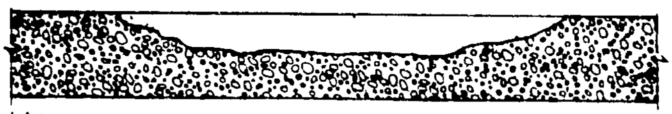
5.3 PATCH REPAIR

Before carrying out repairs, the causes of the damage or deterioration must be clearly identified. In the case of spalled concrete it is important to distinguish between damage caused by scouring or impact, and that caused by corrosion of embedded reinforcement. Each type of damage will require a different type of repair procedure. Therefore, once the cause of damage has been determined, the appropriate repair method can be chosen.[2]

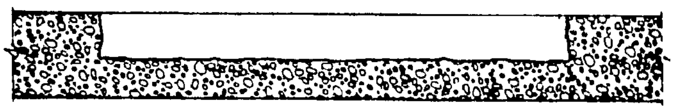
The repair can be achieved by the use of either portland cement or resin-based materials. Their selection depends on the intended purpose of the repair, since they protect concrete in different ways. For instance, cement-based materials provide an alkaline environment for the reinforcing steel which prevents or delays corrosion, while resin-based materials prevent the ingress of oxygen and moisture. The selected material should closely match the mechanical properties of the substrate. Although this implies that, using cement-based materials may be more appropriate, resins are more suitable for underwater repairs.[2] The following sections describe the surface preparation requirements and the general characteristics of these two types of repairs.

5.3.1 SURFACE PREPARATION

Preparing the substrate surface is probably the single most important factor for a successful repair. Applying a sound patch to an unsound surface will lead to failure of the repair, because the patch will spall away by removing some of the unsound material. Therefore, the first step must be to thoroughly remove the unsound or contaminated concrete. The perimeter of the deteriorated concrete area should be saw-cut to a depth ranging from about 6 to 25 mm (¼ to 1 in.) to provide a neat edge. The cut should be normal to the surface or slightly undercut, for a depth of a least 10 mm (⅜ in.) as shown in Figure 5.5. Feathered edges are not desirable and should be avoided as much as possible. Depending upon the depth of deterioration, it is usually preferable to expose the full perimeter of the reinforcing steel because it provides a good mechanical anchorage for the patch repair.[9]



(a) Incorrectly installed patch The feathered edges will break down under traffic or will weather off



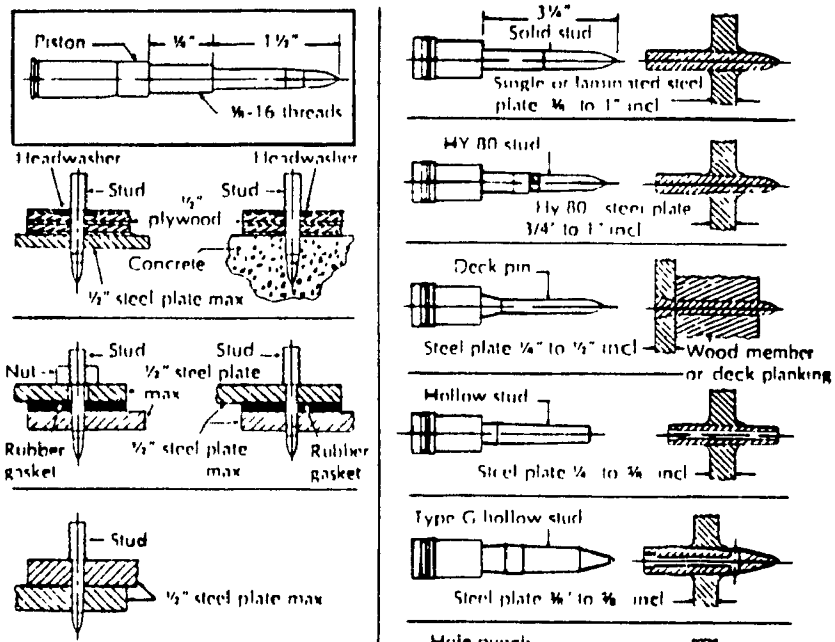
(b) Correctly installed patch The chipped area should be at least 3/4 in deep with the edges at right angles or undercut to the surface

FIGURE 5.5 - PATCH INSTALLATION[34]

Once all the unsound concrete is removed, the surface must be given a final treatment prior to performing the repair. Any reinforcing which is removed must be replaced with new pieces, either spliced with couplers or lapped with existing bars. If reinforcing bars are not available for anchorage, it is often desirable to install dowels drilled and grouted into the surrounding concrete.[2] Alternatively, metal fixings can be fired into the concrete with a velocity-powered underwater stud driver[5] (Figure 5.6). As a final step, the concrete surface and reinforcing steel should be flushed with clean water to remove any dirt, grease, rust or marine growth which may reduce the bond strength of the patch.

5.3.2 CEMENT-BASED MORTARS

Conventional cementitious mortars are susceptible to washing out of fines when they are immersed in water. To prevent this from occurring, special admixtures have been developed which improve the cohesiveness of the mortar and resist cement washout. Several proprietary grouts have also been developed, based on "special cementitious cements and sands with thixotropic and adhesive additives",[2] which resist washout when they are poured through water. The mixes are formulated to be self-levelling and can normally be used in thicknesses of 19 mm (¾ in.) to 150 mm (6 in.). For vertical surfaces, the mortar is poured through water or pumped to fill formwork as shown in Figure 5.7.



Strength of Driven Solid Studs in Structural Steel Plate

Plate thickness	Average pull out strength
¼"	3 000 lbs
5/16"	3 500 lbs
½"	4.000 lbs

Strength of Driven Solid Studs in Structural Steel Plate

Plate thickness	Average pull out strength
⅜	8 000 lbs
½"	14 000 lbs
⅝"	16 000 lbs
¾"	19 000 lbs
⅞"	22 000 lbs
1"	26 000 lbs
1⅛'	29,000 lbs

FIGURE 5.6 - VELOCITY-POWERED TOOL PROJECTILES[5]

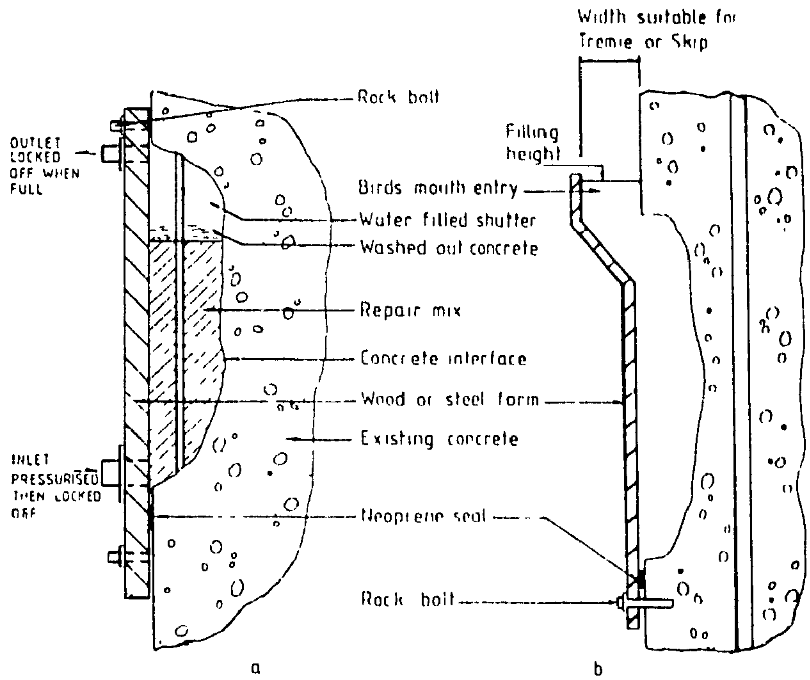


FIGURE 5.7 -TYPICAL FORMWORK DETAIL FOR UNDERWATER REPAIR: (a) PUMPING SHUTTER; (b) BIRD'S MOUTH SHUTTER[2]

Quick-setting (hydraulic) cements are also suitable for repairing small vertical spalls or voids.[7] A hydraulic cement is a single-component material that can cure under water because of the interaction of water and the constituents in the cement. Admixtures can be added to the hydraulic cement for obtaining specific performance requirements (i.e., slow or quick-setting).[9] Quick-setting cement is prepared in small quantities and either hand placed or tool smeared by the diver. Quick-setting cement can attain a compressive strength of up to 41 MPa (6000 psi) and bond reasonably well to the existing concrete provided the substrate has been properly prepared.[7]

5.3.3 RESIN-BASED MORTARS

Conventional epoxy or polyester resin mortars are unsuitable for underwater use. However, with the use of special curing agents, repair mortars have been developed that are insensitive to water and are capable of curing under water.[10] Epoxy compounds are 100 percent reactive, two or three component thermosetting polymers, generally formed by mixing an epoxy resin and a hardening agent (sometimes referred to as a catalyst). Sometimes an oven-dried aggregate is added to the mixture to alter the performance characteristics.[9] In some cases, heavy aggregates, such as barytes are added to the resin so that water can be displaced more effectively when the mortar is poured into the formwork.[2]

Their good adhesion to concrete, bond durability, and variable cure characteristics over a wide temperature range make them very versatile and ideal for patching small spalls or voids in concrete under water. Epoxies can be chemically formulated to suit the specific construction requirements in terms of performance and environmental conditions,[11] but must meet the requirements of ASTM C 881.

The versatility of epoxy formulation is described by Mendis[10] and is shown by the wide range of properties which can be attained:

▸ Physical properties:	Low to high modulus
▸ Rate of cure:	Instantaneous to very long or moderately long cure times.
▸ Temperature cure:	Cure varies from very low to high temperatures.
▸ Water insensitivity:	Ability to cure under moist conditions or under water.
▸ Chemical resistance:	Is resistant to solvents, alcohols, ketone, alkalies, bases, organic acids, and inorganic acids.
▸ Handling versatility:	Low to high viscosities or of gel consistency.

Epoxy mortar compounds must be mixed immediately prior to use. Correct proportioning and thorough mixing is essential for a good performance repair. Most epoxy mortar repair failures are due to incorrect proportioning or inadequate mixing.[2] Thorough mixing can be usually accomplished by the use of mechanical mixing devices, such as drill motor paddle mixers. If the epoxy mortar constituents are of different colors, streaking in the mixture will indicate that mixing is not complete, and should continue until a uniform color is achieved.[12]

Epoxies cure by chemical reaction which begins immediately after the constituents are mixed. The rate of curing or pot life of the mixture depends on temperature and time. Pot life is the amount of time after mixing for which the epoxy mortar can be used before it begins to set. In general, the pot life decreases with increasing ambient temperature. The normal operating temperature range for most commercially available compounds is between 4°C (40°F) and 32°C (90°F). When the concrete or ambient temperature is outside this range, it may be difficult to apply and cure the mixture. In these cases, the constituents can be preheated or cooled to a suitable temperature to ensure effective and adequate curing of the epoxy.[12] The water temperature in which the repair will be made must also be considered because it is usually much colder than the ambient temperature and may affect the curing process

Surface preparation and the patch work should be performed in accordance with the applicable requirements of ACI Committee 503[13] and the manufacturer's recommendations The proper safety procedures for the use of epoxies should be followed and should be in accordance with the requirements of the Federation of Resin Formulators and Applicators [14]

5.4 CRACK INJECTION

Laboratory[15,16] and field studies[12,17] have demonstrated that pressure injection is a viable and cost effective method for restoring the structural integrity of cracked concrete, provided that the crack is dormant (non-moving) and properly cleaned. Pressure injection has been successfully used for repairing cracks in bridge substructures, dams,[18,19] locks,[20] wharves,[21] piles,[22] and other types of concrete structures. Injection, whose use dates back several centuries,[23] involves injecting a sealant liquid that eventually hardens in the crack. The materials currently used for crack injection are either cement-based or epoxies, depending upon the width of the crack and their intended function once hardened.[24]

A general range of crack widths that can be treated by epoxy injection is between 0.05 mm (0.002 in.) to 6 mm (¼ in.). Narrow cracks (0.05 mm/0.002 in to 1 25 mm/0.05 in.) require a low-viscosity epoxy with a rapid cure time. A higher viscosity epoxy can be used to repair wider cracks but it should have a longer gel time to avoid excessive buildup of heat. Too much heat can cause excessive expansion, resulting in cracking when the epoxy cools.[12] Cementitious grouts are suitable for cracks wider than 6 mm (¼ in.). However, due to the risk of washout of cement, epoxy is usually preferred. In these cases, a fine aggregate is added to the epoxy to provide a more substantial filler material and to reduce material cost.[7]

Experimental work has shown that penetration of epoxy adhesive into cracks is affected by crack geometry,[24] temperature, viscosity, pot life, and to a lesser degree, injection pressure.[25] For instance, low viscosities, in the range of 100 to 1000 cp, and a pot life greater than four hours, are typical of high penetration epoxies. Viscosity must be such that back pressure is less than about 700 kPa (102 psi) to prevent the concrete from cracking. Gel time must be long enough so that it does not affect the viscosity drastically. A rapid increase in viscosity may cause difficulty during injection.[11]

Epoxies with a high modulus (high bonding strength and low elongation characteristics) are generally suitable for injecting cracks which are stable. Low modulus, stress-relieving epoxies (lower bond strength and high elongation characteristics) are used for injecting moving cracks. In both cases, the material prevents water from penetrating into the crack. A typical range of desirable epoxy resin properties are given by Mendis[10] and Bean[12] and are included in Appendix K.

5.4.1 INJECTION TECHNIQUE

The injection procedure generally consists of drilling holes at close intervals along the crack, installing entry ports, sealing the crack between the ports, and injecting the epoxy or grout under pressure. The injection usually begins by pumping the epoxy resin into the lowest port of vertical or inclined cracks, and the port at one of the ends of a horizontal crack. The pumping continues until a good flow of epoxy emerges from the next higher or adjacent port. The first port is then plugged, usually with wooden dowels, and injection continues into the adjacent port and so on. In some cases, where draining the water from the crack is not necessary, injection can be started at the port in the widest crack because it is easier to fill a narrow crack from a wider portion of the crack rather than vice versa.[26] A typical crack injection system is shown in Figure 5.8.

5.4.2 INJECTION PORT INSTALLATION

The most common method of installing entry ports involves inserting fittings into drilled holes. The fittings are usually surface mounted, however, in some cases can be socket mounted (recessed).[26] The holes are either drilled directly into the crack or at an angle to intercept the crack. The injection ports can be bonded into the holes with quick-setting epoxy resin to prevent them from being ejected during pressure injection. The ports should be strong enough to allow the epoxy to be injected into the cracks, such as one-way pipe nipples, tire valve stems, and copper tubing[12] or patented packers.[27] Ports fabricated from cutting nylon tubing have also been used extensively for underwater injection work. Other methods frequently used to provide entry ports include bonding a fitting (with a hat-like cross section and a hole in the top) flush with the concrete face over the crack, or omitting the epoxy seal from a small length of the crack. The latter can be used with special gasket devices that cover the unsealed portion of the crack and allow epoxy injection directly into the crack.[28]

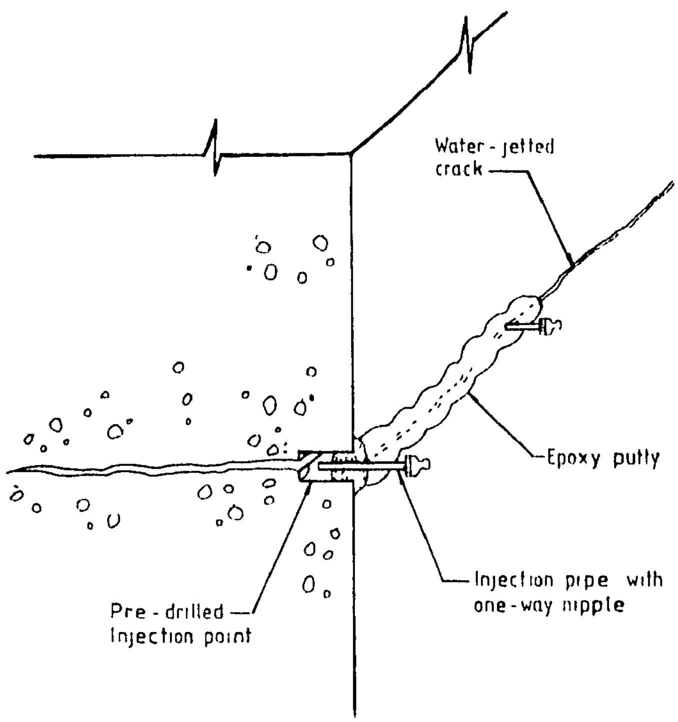


FIGURE 5.8 - TYPICAL CRACK INJECTION SYSTEM[2]

5.4.3 PORT HOLE DIAMETER AND SPACING

Injection holes for most jobs are 13 mm (½ in.) or 16 mm (⅝ in.) in diameter. For massive structures, 22 mm (⅞ in.) and 25 mm (1 in.) diameter holes are drilled to intercept the crack at several locations.[28] The depth of the hole into or at the intersection of the crack can vary from a minimum of 50 mm (2 in.) to 300 mm (12 in.) for thicker concrete sections.[27]

Drill hole spacing depends on crack width and depth.[12] Injection holes are normally spaced from

100 mm (4 in.) to 300 mm (12 in.) apart.[2] In some cases, holes can be spaced as much as 1.5 m (5 ft.) apart. In general, if cracks are less than 0.125 mm (0.005 in.) wide, injection port spacing should not be more than 150 mm (6 in.). For cracks in members less than 610 mm (2 ft.) in thickness, ports should not be spaced more than the thickness of the member. For cracks that are greater than 610 mm (2 ft.) in depth, intermediate ports should be installed to monitor grout flow,[12] and to ensure full depth penetration as shown in Figure 5.9.

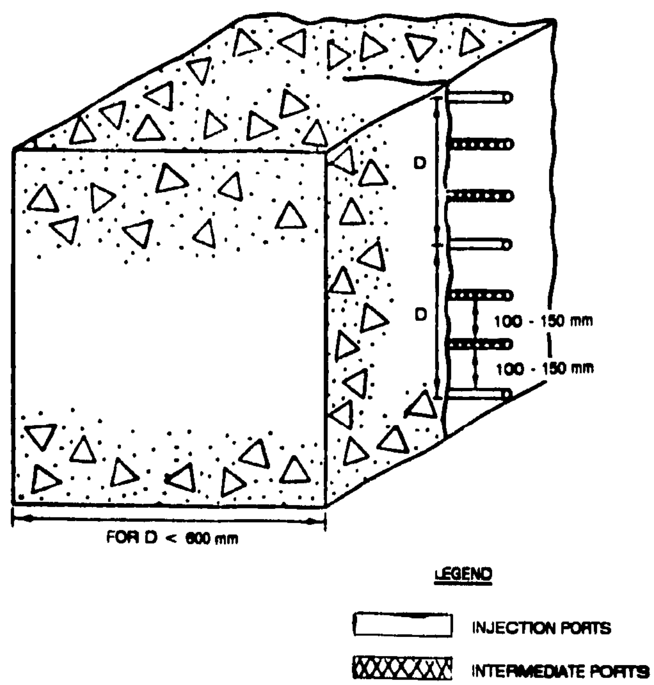


FIGURE 5.9 - INJECTION AND INTERMEDIATE PORTS FOR MONITORING GROUT FLOW[12]

5.4.4 CRACK SEALING AND CLEANING

After all the injection ports have been installed, the crack lengths between the entry points should be sealed to prevent the epoxy from running out of the crack during injection. The material often used for sealing cracks in underwater concrete structures is a thixotropic epoxy paste. The epoxy should have adequate bond strength to withstand injection pressures. A U.S. Engineer Army Waterways Experiment Station (WES) laboratory report[12] recommends that a sealant be capable of containing the epoxy resin at an injection pressure of about 690 kPa (100 psi) for up to 10 minutes.

Prior to injecting the epoxy resin, thorough cleaning of the crack is essential. The method of cleaning is dependent on the size of the crack and the nature of the contaminants.[29] In most cases, the crack is flushed with a high-pressure straight-nozzle water jet to remove internal contaminants (such as grease or marine growth) which can prevent epoxy penetration or inhibit the bonding of the faces of the crack. If necessary, water blasting can be combined with wire brushing, routing, or the use of picks or similar tools.[12] The bonding characteristics of the substrate can also be improved by mixing a bio-degradable alkaline based detergent or specially formulated chemicals with the blast water.[20]

5.4.5 INJECTION EQUIPMENT

There are three types of equipment used for epoxy injection of cracks: a hand caulking gun, a pressure pot, and a dispensing machine.[12] With the hand caulking gun and the pressure pot the epoxy resin and hardener components must be mixed manually, whereas the dispensing machine mixes the components in the system immediately prior to injection. Although the resin components can be mixed manually with graduated beakers, mixing paddles, and power drills, the best method of mixing is done with dispensing machines. All of these injection systems are designed for low-pressure injection applications. A good review of these three methods is provided in Reference 12 with the salient points summarized below.

5.4.5.1 HAND CAULKING GUNS

Caulking guns are usually employed for small jobs involving low-pressure grouting operations. The standard caulking gun consists of a 325 ml (1/12-gal.) caulking tube with a 75 mm (3 in.) tapered plastic nozzle. The epoxy compound mixture is poured into the caulking tube, the cap is placed into the tube, and the cartridge is then inserted into the gun. About 3 mm (⅛ in.) to 6 mm (¼ in.) of the tip of the plastic nozzle is cut off and the aluminum seal is pierced. The epoxy resin should be pushed to the tip of the nozzle to force out the air at the top of the cartridge.

The grouting operation should be started as soon as possible to prevent the epoxy resin from gelling in the cartridge. The tip of the plastic nozzle is inserted into the entry port and the epoxy is injected by squeezing the trigger. Flow of the epoxy can be monitored by watching for the movement of air bubbles in the clear plastic nozzle. If the epoxy-resin mixture in the cartridge starts to generate heat, the pot life is about to be reached and grouting should stop until a new cartridge is prepared. When the grouting stops, the caulking gun should be cleaned with solvent, if necessary.

The injection operation can be facilitated and expedited by using pneumatic-powered hand caulking guns. The injection procedure is identical to that with a hand-powered caulking gun except that hydraulics are used to deliver the epoxy into the cracks instead of the hand trigger.

5.4.5.2 PRESSURE POT

The pressure pot apparatus is similar to equipment used for spraying paint, and uses an 8 liter (2-gal.) pressure pot as the reservoir for the freshly mixed epoxy-resin, which is poured into a 2 liter (½-gal.) plastic container, which is then placed in the pressure tank. A flexible rubber feed line is attached to the inside of the outlet port on the lid of the tank extending to the bottom of the reservoir and the lid is secured and pressurized. Once the tank is pressurized, the epoxy injection hose can be used to grout the cracks.

The pot uses either compressed air or an inert gas to provide the operating pressure (690 kPa/100 psi minimum). To minimize pressure losses in the system, the injection hose is usually not very long (less than 3.3 m/10 ft.), and therefore it must be placed near the injection ports. For this reason, the pressure pot has seen limited use for underwater applications. The pressure pot should be flushed at the end of each days' work, or any time the injection work is stopped longer than the pot life of the mixture. Flushing could be done with methylethyl ketone, toluene, or any other recommended solvent.

5.4.5.3 DISPENSING MACHINES

Using epoxy dispensing machines is the quickest and easiest method of injecting cracks. With this method, the epoxy compound is mixed as it is needed, thus, eliminating any concern about pot life. Several types of proprietary dispensing machines are available which pump the proper proportions of epoxy resin and hardener to a special intermixing nozzle near the injection port.

Pneumatically-operated, variable ratio dispensers are most widely used for crack injection operations. For this system, the epoxy resin and hardening agent are placed in separate canisters and the desired pumping ratio is set. Each component is then pumped by the proportioning pump to the mixing nozzle by a remote control switch which is attached to the feed lines. This allows a diver to operate the pump as needed while injecting the epoxy into the ports. Epoxy injection continues until one or both canisters are empty. When this occurs, fresh materials are added to the proper canisters and injection can proceed. If injection is stopped for any period longer than the pot life of the material, the complete system must be flushed with a solvent. The system should be also flushed out with compressed air to ensure that any remaining solvent is removed.

To ensure the dispensing machine is delivering the correct mix volumes, two control devices are provided: the ratio check device and the pressure check device. The ratio check device is connected to the dispensing machine and both adhesives are pumped simultaneously through the device during the same time interval into separate calibrated containers. The amounts pumped are compared to determine if the volume ratio is correct. Adjustments should be made if the amounts pumped vary more than two percent. The pressure check device ensures that the proportions are not changing due to leakage or seepage. The device is connected to the mixing head and the pressure drop is monitored once the pump is stopped. If the pressure drops more than 140 kPa (20 psi) in three minutes, grouting should stop until the problem is corrected.

5.5 LARGE VOLUME REPAIRS

Occasionally, large volumes of concrete are required to be placed under water, for example, to repair erosion damage to dam stilling basins, navigation lock floors, spalled seawalls, or simply to protect foundations against scour damage. Underwater concrete placement is often carried out under conditions which adversely affect the characteristics of the fresh mix. The quality and resulting durability (compressive strength, bond, permeability, etc.) of the concrete will depend on the composition of the mix and the method by which it is deposited.[30]

The major concern in placing conventional concrete under water is the washing out of cement fines and sands as the fresh mix moves through the surrounding water, resulting in a higher water-cement ratio.[31] Therefore it is, therefore, essential to produce a mix which is cohesive enough not to segregate, but adequately workable so that it can consolidate under its own weight without the need for compaction.[32]

The resultant demand for higher quality underwater repairs, due mainly to the high cost and technical difficulty usually associated with dewatering, prompted considerable research into developing concrete mixtures and techniques suitable for underwater repair work. The following sections provide a summary of the existing and recently developed techniques for repairing concrete structures under water.

5.5.1 UNDERWATER CONCRETE MIX DESIGN

In many respects, the mix design for underwater repair concrete is normally designed using the same rules and recommendations as would be used for repairing concrete in dry conditions.[2,33] However, depending on the nature of the repair work and the available resources, certain modifications may be required, as described below.

5.5.1.1 CEMENT

Different types of portland cement are manufactured to meet various physical and chemical requirements of specific environments to which it will be exposed.[34] American Society of Testing and Materials (ASTM C150) lists eight types of portland cement, of which Type II (Type 20: moderate sulfate resistance) is usually recommended for underwater concrete. In cases where sulfate exposure is more severe, Type V (Type 50) cement is usually more suitable.

To reduce the effects of washout and maintain a sufficiently low water-cement ratio, a relatively high cement content is needed. A review of the literature[7,32,35,36,37] indicates that a cement content of approximately 350 to 415 kg/m^3 (590 to 700 lb/yd^3) of concrete will be suitable for most underwater concreting applications. For such rich mixtures, water-reducing admixtures are required to produce a highly flowable concrete while maintaining a low water-cement ratio. For large repairs, a portion of the cement content (up to 15 percent) is sometimes replaced by pozzolans, such as fly ash or silica fume, to reduce the heat of hydration usually associated with rich mixtures. Addition rates in excess of 15 percent can significantly reduce the workability of the concrete and decrease the strength gain.[38] Lean mixes of less than 330 kg/m^3 (556 lb/yd^3) are highly susceptible to cement washout and will probably not be suitable for underwater applications.[2]

The most suitable cement content used in a mix design should be determined by trial mixes performed at the site, and will largely depend upon the particular application (large volume or thin lift) and the method (tremie or pump) used for depositing the concrete.[30] For large reinforced concrete repairs, where concrete is placed by bottom skip or toggle bag (Section 5.5.2), mixes are not usually designed. Instead, a common approach is to use existing mix proportions, known to give the desired compressive strength at the workability normally used in the dry, and slightly oversanding the mix and increasing the cement content by approximately 25 percent. This results in a cohesiveness and workable mix which does not require compaction and resists loss of cement by washout.[33]

5.5.1.2 AGGREGATES

To produce a flowable, self-levelling concrete, while maintaining a low water-cement ratio, the use of well-graded rounded aggregate is generally recommended. Properly washed marine dredged aggregates and round river gravels will be most suitable.[2] The maximum aggregate size is particularly dependent on the method of placing the concrete. Pumped concrete, for instance, will require a finer particle size than tremie-poured concrete.[30] For large unreinforced repairs a 40 mm (1-½ in.) aggregate size is usually recommended, while for reinforced placements a maximum aggregate size of 19 mm (¾ in.) should be used.[32]

Research by Gerwick[39] has shown that a high sand/gravel ratio is beneficial to the concrete mix with regard to segregation and washout. Accordingly, Gerwick recommends the use of 42 to 45 percent (by weight) of sand to the total quantity of aggregate. To obtain a cohesive mix, a sand gradation without the finest particle size should not be used. If such a sand is used, the addition of fine material, such as fly ash, should be used.[30]

5.5.1.3 MIXING WATER

Fresh, potable water is always an essential ingredient for producing good quality concrete. This is especially important in reinforced concrete where certain contaminants in the water (i.e., chlorides, sulfates, etc.) not only can affect concrete strength, but also cause corrosion of the reinforcing steel. However, some waters that are not fit for drinking may also be suitable for concrete.[34] Acceptable criteria for water to be used in concrete is provided in ASTM C 94 specifications.

In many parts of the world, however, this restriction can present major practical problems, and often a financial burden. For instance, many parts of the world, such as the United States

(California and Florida), the United Kingdom, and France, depend heavily on sea-dredged sands and gravels, which even after double washing, contain harmful levels of chlorides. In developing countries, construction is often plagued by shortages of fresh water, which may need to be imported at a high cost. A recently developed "seawater concrete process"[40] may prove to be a viable solution to both practical and financial difficulties imposed by water requirements. The new process makes it possible to mix unwashed beach sands and sea-dredged aggregates with seawater. The process uses a chemically modified portland cement, mixed with complex mineral constituents in ratios that depend on project specifications and the available materials at the site. The process allows hydration with water containing up to 100 grams per liter of salts, which is considerably greater than the 32 grams per liter content of normal seawater. Contractors must be licensed to use the process. The following advantages of the process have been reported:[40]

- Provides a protective inorganic polymer coating on the reinforcement
- Reduces setting times
- Improves compressive strength
- Reduces permeability, shrinkage, and cracking
- Increases modulus of elasticity

5.5.1.4 ADMIXTURES

Recent research[41,42] has shown that certain concrete admixtures has made it possible to place higher quality concrete under water. Test results show that the incorporation of antiwashout admixtures (AWA) and water-reducing agents produces cohesive, flowable, and abrasion-resistant concrete which resists cement washout, and reduces segregation and bleeding. Well proportioned concrete containing AWAs can decrease the mass loss of the fresh mixture when dropped through water by three times as compared to conventional tremie mixes with an equivalent slump.[43]

AWAs are natural or synthetic water-soluble polymers which physically bind the mixing water in the concrete, thus increasing the viscosity of the mixture. A majority of AWAs consist of microbial polysaccharides, such as welan gum or polysaccharide derivatives, such as hydroxypropyl methylcellulose and hydroxyethyl cellulose.[37] Optimum dosage rates of AWAs are small and decrease with a decreasing water-cement ratio. Too much AWA can significantly reduce the workability of concrete. Studies show that mixes with water-cement ratios from 0.32 to 0.40 require only approximately one-tenth of the amount of AWA recommended by the manufacturer, because their dosage rates are based on water-cement ratios ranging from 0.45 to 0.65.[38] The five categories of AWAs, as classified by Ramachandran,[44] along with dosage ranges are summarized in Table 5.3.

TABLE 5.3 - CLASSIFICATION OF ANTIWASHOUT ADMIXTURES (RAMACHANDRAN)[41]

Class A	Water-soluble synthetic and natural organic polymers, which increase the viscosity of the mixing water. Examples include cellulose ethers, pregelatinized starches, polyethylene oxides, alignates, carrageenans, polyacrylamides, carboxyvinyl polymers, and polyvinyl alcohol. The dosage range used is 0.2 to 0.5 percent solid by mass of cement.
Class B	Organic water-soluble flocculants, which are absorbed on the cement particles and increase viscosity by promoting interparticle attraction. Examples include styrene copolymers with carboxyl groups, synthetic polyelectrolytes, and natural gums. The dosage range used is 0.01 to 0.10 percent solid by mass of cement.
Class C	Emulsions of various organic materials, which increase interparticle attraction and also supply additional superfine particles in the cement paste. Examples include paraffin-wax emulsions that are unstable in the aqueous cement phase, acrylic emulsions, and aqueous clay dispersions. The dosage range used is 0.10 to 1.50 percent solid by mass of cement.
Class D	Inorganic materials of high surface area, which increase the water-retaining capacity of the mix. Examples include bentonites, pyrogenic silicas, silica fume, milled asbestos, and other fibrous materials. The dosage range used is 1 to 25 percent solid by mass of cement.
Class E	Inorganic materials that supply additional fine particles to the mortar pastes. Examples include fly ash, hydrated lime, kaolin, diatomaceous earth, other raw or calcined pozzolanic materials, and various rock dusts. The dosage range used is 1 to 25 percent solid by mass of cement.

Since AWAs increase the water demand of concrete mixtures, especially those with a high cement content and a low water-cement ratio, high-range water reducers (HRWRs) are needed to maintain a flowable concrete without reducing its strength or durability.[42] The type of HRWR used affects the washout characteristics of the mixture. For instance, mixtures containing melamine - and lignosulfonate - based HRWRs have proven to be more resistant to washout than mixtures containing naphthalene or synthetic polymers. However, mixtures containing naphthalene improved the abrasion-erosion resistance of concrete more than other HRWRs.[41] Some HRWRs and AWAs are incompatible. Cellulose-derivative AWAs are compatible only with melamine-based HRWRs. Many proprietary products are sold with the HRWR and AWA in the admixtures.[38]

It is reported that combining AWAs and HRWRs may entrap up to 15 percent air by volume, resulting in reduced concrete strength.[38] Natural gum AWAs, such as welan gum, do not entrap air and can be used with either naphthalene or melamine-based HRWRs.[37] Alternatively, air-detraining admixtures, such as tributyl phosphate or octy alcohol have been used to reduce the air content.[38]

The use of pozzolans, such as silica fume and fly ash, are frequently used in concrete to enhance durability, strength, and adhesion, and are sometimes added to improve washout resistance of underwater concrete.[37,38,42] A portion of the cement is sometimes replaced with pozzolans of high fineness to minimize expansion due to alkali-silica reaction and sulfate attack.[45] Pumping-aid admixtures, are sometimes used to produce flowable mixtures, however, they do not adequately prevent washing out of fines and cement.[38] The proper type and dosage of admixtures should be determined by trial batches at the site prior to the beginning of any concrete placement. A list of admixtures for use in concrete along with the applicable ASTM specification under which they are standardized is provided in Appendix L.

5.5.1.5 WORKABILITY

As underwater concrete must be able to compact under its own weight, the fresh mix must have a high workability (slump) and possess good flow characteristics. To achieve this, a slump of 150 to 200 mm (6 to 9 in.) is commonly used. For heavily reinforced repairs or when concrete must flow over long horizontal distances, a slightly higher slump may be required.[32] The degree of workability of the concrete also depends on the method chosen for placing and finishing.[38]

Research conducted by Heaton[46] concluded that for concrete with slumps from 150 to 200 mm (6 to 9 in.), there is virtually no difference between the compressive strength of compacted and uncompacted concrete. The research revealed that uncompacted concrete placed under water can produce compressive strengths of about 35 to 50 MPa (5000 to 7200 psi). The results of the compressive strength of concrete as a function of slump and degree of compaction are shown in Figure 5.10.

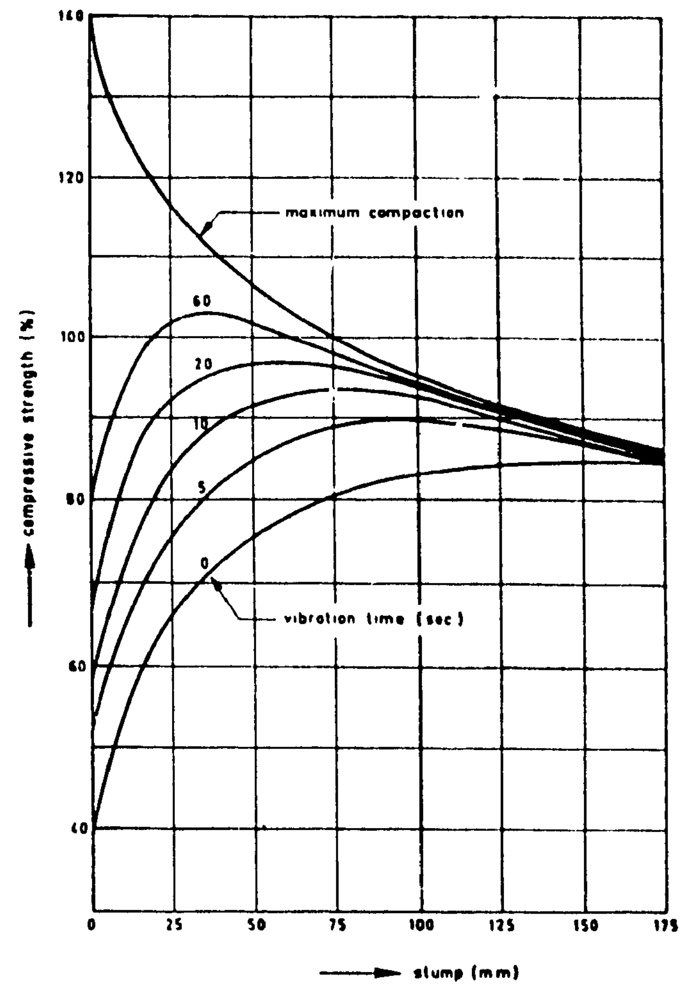


FIGURE 5.10 - COMPRESSIVE STRENGTH OF CONCRETE AS A FUNCTION OF PLASTICITY AND DEGREE OF COMPACTION[30]

5.5.2 UNDERWATER CONCRETE PLACEMENT METHODS

A review of the literature[2,30,33,41] reveals that there are several techniques used for placing concrete under water, some of which have existed since the turn of the century. In the earliest applications, massive volumes of concrete, where high compressive strengths were not required, were successfully placed under water using the well known tremie method. Variations of the tremie method, such as the pumping and hydrovalve methods, were later developed and used extensively in Europe. Although these methods were designed to prevent cement washout, they did not reach their full potential until the relatively recent development of suitable admixtures (AWAs) to minimize this problem. For many applications in Europe and Japan, pumped concrete has become the preferred method over the traditional tremie pipe.[41]

Other underwater placement methods, such as the skip box and the recently developed tilting pallet, allow the concrete to free fall through the water. These methods rely on the use of AWAs to prevent cement washout. The tremie method and pumped concrete "are designed to protect the concrete from exposure to the water".[41] The method chosen for placing concrete underwater must not create tublulence so that the contact between the concrete and the water is minimized.[2] The following sections describe the possible methods for placing concrete under water and are summarized in Table 5.4.

TABLE 5.4 - UNDERWATER CONCRETE PLACEMENT METHODS[30]

place of making the concrete	manner of making the concrete	method of placing	cement washout risk	compaction of the concrete	quality control
above water	- element with final shape (precast and hardened)	- pouring - stacking - assembling		yes	applied to fresh and to hardened concrete
	- plastic element (fresh concrete in bags)	- depositing - stacking	no		
	- grout	- injecting between fabric			applied to fresh concrete or grout before placing or injecting
	- concrete (freshly mixed)	- tremie method - pump method - hydrovalve method - skip method	yes	no	
under water	- grout injected into coarse aggregate mass	- prepakt method - colcrete method			

5.5.2.1 TREMIE METHOD

For many years concrete has been successfully placed under water using the tremie method. It is best suited for placing large volumes of highly flowable concrete. This method allows concrete to be placed from the surface to the exact underwater location by the use of a tremie pipe. The pipe is connected to a hopper into which the concrete is deposited by skips, belt conveyor, or by pumping. The lower end of the tremie pipe is kept immersed in the freshly placed concrete to prevent the concrete which flows out of the pipe from intermixing with the water.[30,32] However, with the use of AWAs, this requirement may not be as critical for ensuring a successful underwater repair. The following important factors must be considered when placing concrete by the tremie method:

(a) Tremie Equipment. There are three possible configurations for the tremie pipe.[9] constant length pipe which is raised as concreting proceeds; pipe which is made up of a number of sections (with flanged and gasketed joints) which are dismantled during concreting, and telescopic pipe (attached to the hopper is a pipe of smaller diameter than the actual concrete placing pipe).[30] The pipe and pipe joints must be strong and watertight. Typically, a steel tremie pipe is used, but a rigid rubber hose[9] or a flexible pipe[2] could be used instead An aluminum alloy tremie pipe should not be used because it can produce an adverse chemical reaction with the concrete.[9] The tremie pipe should have a smooth inside surface and be of adequate cross-section for the size of aggregate to be used.

Tremie pipe diameter usually ranges from 200 to 300 mm (8 to 12 in.), though diameters as small as 150 mm (6 in.) and up to 450 mm (18 in.) have been occasionally used. A tremie pipe diameter of 150 mm (6 in.) is commonly considered as the minimum for 19 mm (¾ in.) aggregates and 200 mm (8 in.) as the lower limit for 40 mm (1-½ in.) aggregates,[33] and should be at least eight times the maximum coarse aggregate size.[9] Smaller diameters may cause pipe blockages, however, 100 mm (4 in.) diameter tremie pipes have been used for small repairs.[32]

The hopper is used to provide a steady flow of concrete down into the pipe, and should be large enough to enable the level of the concrete in the hopper to remain constant.[33] The pipe and hopper assembly is usually supported by a crane which can control vertical and horizontal movement of the tremie pipe. A typical tremie pipe arrangement is shown in Figure 5.11.

(b) Tremie Seal. The tremie pipe is positioned over the area to be repaired with the lower end of the pipe resting on the bottom. Various methods have been used to prevent intermixing of concrete with water in the pipe. Steel plates or wooden plugs are fitted to the end of the pipe, when the "dry pipe" method is used for starting the tremie pour. As the pipe is lowered to the bottom, the hydrostatic water pressure seals the gasket and keeps the interior of the pipe dry. Once the tremie pipe is filled with concrete, it is raised slightly (usually no more than 150 mm/6 in.), allowing the end seal to break. Concrete flows out and accumulates up around the mouth of the pipe, creating a seal.[32]

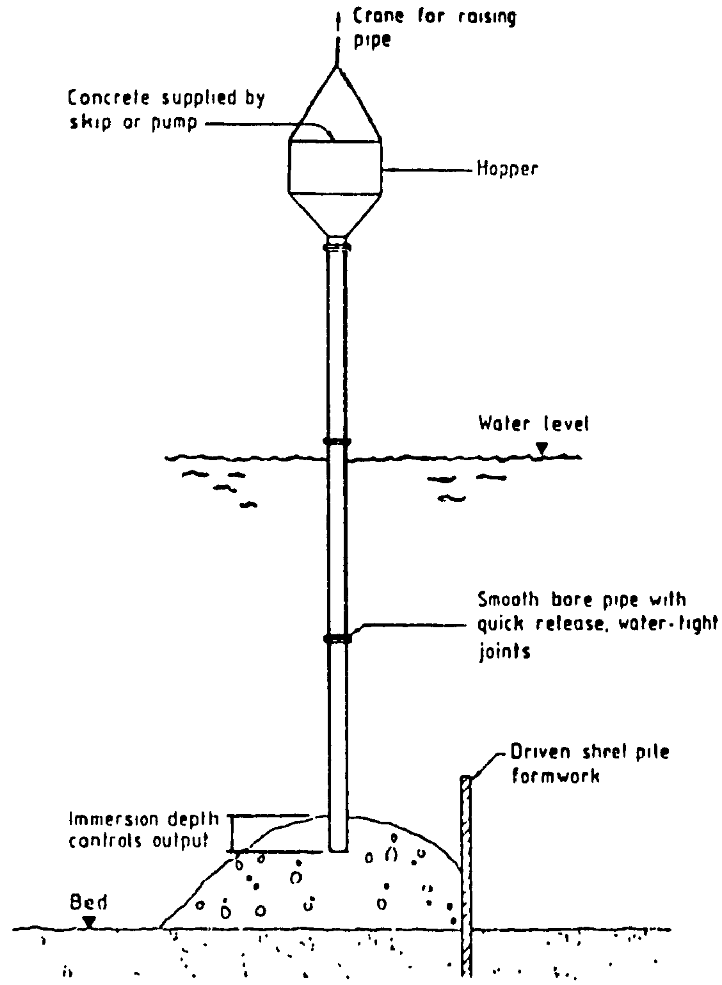


FIGURE 5.11 - TYPICAL TREMIE PIPE ARRANGEMENT[2]

For deep water applications, the buoyancy of the empty pipe may be a problem during positioning. For this reason, the "wet pipe" technique is more commonly employed.[32] In this method, a travelling plug is inserted at the top to act as a barrier between the concrete and the water. The water in the pipe is then pushed out as the weight of the concrete forces the plug to the bottom. Once the plug reaches the bottom of the pipe, it usually floats back to the surface once the tremie is lifted. Foam plastic (or rubber) and inflated rubber balls have been frequently used as a travelling plug.[33] However, an inflated rubber ball may collapse at depths greater than 7.6 m (25 ft.) and may not be effective as a seal.[32] To resolve this problem, wooden spheres, made from low density wood, such as pine,[47] or a wad of burlap have been used.[9]

(c) Placing the Concrete. Once concrete placement has started, the mouth of the pipe should

remain buried about 1 to 1.5 m (3 to 5 ft.) deep in the fresh concrete. Concrete placement should be as continuous as possible making sure that the level of the concrete in the hopper is kept at a constant height to ensure a smooth continuous flow.[32] The concrete flow rate in the pipe is controlled by raising and lowering the tremie.[2] All vertical movements of the tremie pipe must be slowly and carefully done to prevent loss of seal. The volume of concrete being placed during the tremie operation should be continuously monitored to detect a loss in seal Underruns are indicative of a loss in seal because washed and segregated aggregates occupy a larger volume. A noticeable increase in flow rate of concrete in the pipe will also indicate loss of seal.[32]

During concrete placement, the tremie pipe must remain fixed horizontally to avoid damaging the concrete surface in place, which could lead to additional laitance (weak mortar) and loss of seal. Tremie pipes should be closely spaced so that concrete does not have to flow over long distances. Otherwise, too much concrete surface area will be exposed to water, causing segregation and formation of laitance. A pipe spacing of two or three times the depth of concrete being poured has been suggested.[32] Distributing the concrete horizontally is either accomplished by flow of the concrete itself or by stopping and repeating the process, including reestablishing the tremie seal.

When depositing large volumes of concrete, two methods are used to spread the concrete horizontally: the layer method and the advancing slope method. In the layer method, the entire area is concreted at the same time using several tremie pipes, keeping a level surface as the concrete rises. With the advancing slope method, the area is concreted one section at a time by moving the tremie pipe to adjacent areas.[32] A single tremie pipe can usually concrete an area of about 30 m^2 (300 $ft.^2$).[33]

(d) Flow Pattern. It was traditionally believed that during a tremie pour, concrete flows under and is protected by previously placed concrete. However, a recent study has shown that tremie concrete may produce different flow patterns which may expose more concrete to water than was originally perceived. The study also concluded that the flow pattern is affected by the shear characteristics of the fresh concrete. Two different flow patterns were observed: layered and bulging flow.[48]

A layered flow pattern was associated with concrete having a high internal shear resistance. The new concrete flowed up and around the pipe and then outward over the previously placed concrete, producing very steep slopes. This created a significant amount of laitance at the far end of the pour. The bulging flow pattern produced a more uniform displacement of the concrete, resulting in much flatter slopes and less formation of laitance. This preferred flow pattern was reportedly made possible by reducing the internal shear resistance of the fresh concrete.[46]

Since the flow pattern seems to be related to the shear properties of the fresh mix, it is important to produce a highly flowable concrete. For instance, conventional tremie concrete mixes did not perform the best in the studies. The studies found that replacing up to 50 percent of the cement with fly ash improved the performance of the tremie concrete. Also air-entraining agents and some water-reducers decreased the shear resistance of fresh concrete, producing the preferred type of flow pattern.[46] A sample of a good mix proportion and a typical aggregate gradation are shown in Tables 5.5 and 5.6, respectively.

TABLE 5.5 - EXAMPLE OF A GOOD CONCRETE MIX PROPORTION FOR USE WITH THE TREMIE METHOD[38]

	kg/m^3	lb/yd^3
Cement with 10% fly ash	360	605
Silica fume	36	60
Coarse aggregate, 8-20 mm	858	1445
Coarse sand, 0-12 mm	860	1450
Fine sand, 0-8 mm	146	245
Water	146	245
Water-reducing admixture	7	7
Superplasticizer	7	7
w/c = 0.47, slump = 23 cm, air content = 2 percent		

5.5.2.2 PUMPED CONCRETE

Pumping concrete is an extension of the tremie method. Recent improvements in the design of concrete pumps[33] and development of AWAs has made pumping the preferred method of placing concrete under water.[37] It provides the most expeditious means of placing concrete under water in areas of limited or difficult access, such as beneath piers.

TABLE 5.6 - TYPICAL GRADATION OF AGGREGATES FOR TREMIE PIPE CONCRETE[9]

Aggregate	U.S. Standard Sieve	Percent Passing (by weight)
Gravel	1 in.	100
	3/4 in.	90 to 100
	3/8 in.	20 to 55
	№ 4	0 to 10
	№ 8	0 to 5
Sand	3/8 in.	100
	№ 4	95 to 100
	№ 8	80 to 100
	№ 16	50 to 85
	№ 30	25 to 60
	№ 50	10 to 30
	№ 100	2 to 10

Pumping concrete offers several advantages over the tremie method:

- Concrete can be deposited directly from the mixer into the formwork.[37,38]
- Concrete can be pumped to the bottom of the formwork to displace the water through a vent at the top[2], or by inserting the end of the pipe or hose into the form from the top,[7] avoiding free-fall of concrete through the water.
- Concrete is delivered under pressure rather than fed by gravity, so blockages in the pump line can be easily corrected.[37,38]
- Use of a crane boom affords more precise positioning of the concrete during discharge.[37,38]
- Depositing the concrete under pressure reduces the need to constantly lift and free the tremie pipe. This reduces the risk of segregation within the concrete.[33,38]

For small-volume pours, small-diameter (50 to 100 mm/2 to 4 in.) pump lines can be easily controlled by divers.[37] When using small diameter pump lines, the concrete must be flowable and cohesive enough to pass through the pump without blockage. This usually requires a lower slump concrete than that used for tremie mixtures. Slumps from 100 to 125 mm (4 to 5 in.) have been used successfully.[30,33] However, mixtures which contain too much water tend to segregate and cause blockage in the hose or pump line. Higher slump concrete (200 to 250 mm/8 to 10 in.) has also been used with the aid of AWAs to provide a high degree of cohesion, needed to prevent washout of fines. For instance, the underwater concrete that was pumped to repair the end sill at Red Rock Dam[36] in south central Iowa had a slump of 230 mm (9 in.) and contained an AWA. The concrete was delivered using a 100 mm (4 in.) diameter pump and the mix proportions used are shown in Table 5.7.

TABLE 5.7 - CONCRETE MIX PROPORTIONS USED FOR PUMPING REPAIR CONCRETE AT RED ROCK DAM[36]

	lb/yd^3	kg/m^3
Portland cement (Type I)	700	415
Fine aggregate (natural)	1,299	771
Coarse aggregate (3/4 in. [19.0 mm] crushed limestone)	1,594	946
Water	275	163
Antiwashout admixture	5	3
	fl oz./yd^3	ml/m^3
Water-reducing admixture	42	1,600
Air-entraining admixture	2	77

Small, rounded coarse aggregates are preferred over crushed stone. If crushed rock is used, the coarse aggregate should have a maximum size of less than one-third the smallest inside diameter of the hose or pipe being used. Porous aggregates, such as expanded clay, foamed slag, pumice, and many coralline materials, should not be used, since they tend to absorb water and stiffen the fresh mix.[9] If these aggregates must be used, they should be presoaked as described in ACI Committee Report 304.2R.[49] The properties of fine aggregates (sand) are more important than those of coarse aggregates. The sand should have a relatively high fraction of the finer sizes.[9] A typical gradation of aggregates suitable for use with a 50 mm (2 in.) diameter pump is shown in Table 5.8.

Careful planning of pump location and hose routing is essential before starting an underwater repair operation. The pump line should be placed horizontally or vertically to prevent the buildup of bleed water in the pump line. The hose or pump line should be lubricated with a lean cement slurry before pumping commences to prevent segregation and blockage.[9] Segregation of the concrete can also be prevented by forcing a sponge plug into the top of the line to physically support the mix and prevent free-fall between pump strokes. For very deep water applications, this may need to be supplemented with bends in the pipe or hose to break the fall of the concrete.

TABLE 5.8 - TYPICAL GRADATION OF AGGREGATES FOR PUMPED CONCRETE[9]

Aggregate	U.S. Standard Sieve	Percent Passing (by weight)
Gravel	1/2 in.	100
	3/8 in.	85 to 100
	Nº 4	10 to 30
	Nº 8	0 to 10
	Nº 16	0 to 5
Sand	3/8 in.	100
	Nº 4	95 to 100
	Nº 8	80 to 100
	Nº 16	50 to 85
	Nº 30	25 to 60
	Nº 50	15 to 30
	Nº 100	5 to 10

At the end of the pour, the pump can be continued to flush out weak concrete which may have developed as a result of segregation or washout. Also by closing the inlet and the outlet valves, the concrete within the formwork can be pressurized to minimize bleeding of the mix.[2] Concreting should proceed reasonably fast and should continue as long as possible without long delays. Care must be taken not to pump fresh concrete under concrete which has already begun to set.[33] A typical pump line arrangement for underwater concreting is shown in Figure 5.12.

5.5.2.3 HYDROVALVE METHOD

The hydrovalve method, which was developed and first used by the Dutch in 1969,[30] is a variation of the tremie method. This method uses a flexible (nylon) hose which is compressed by the hydrostatic water pressure to deliver the concrete. As the concrete is placed in the upper part of the hose its weight will eventually overcome the combined hydrostatic pressure and friction within the hose, allowing it to move slowly down the hose. The "slow and contained movement" of the concrete down the hose helps to prevent segregation.[38] The bottom part of the placing hose is enclosed within a rigid tubular section which is placed at the desired level of concrete surface.[30] The thickness of the concrete is built in successive layers and can be placed with a tolerance of ± 100 mm (± 4 in.).[38] A typical hydrovalve apparatus is shown in Figure 5.13.

An advantage of this method is that it can place stiff mixtures (having a slump less than 140 mm/5-½ in.) as well as higher slump mixes usually employed with the tremie method. It is also relatively simple and inexpensive. Further details on this method is provided in Reference 47.

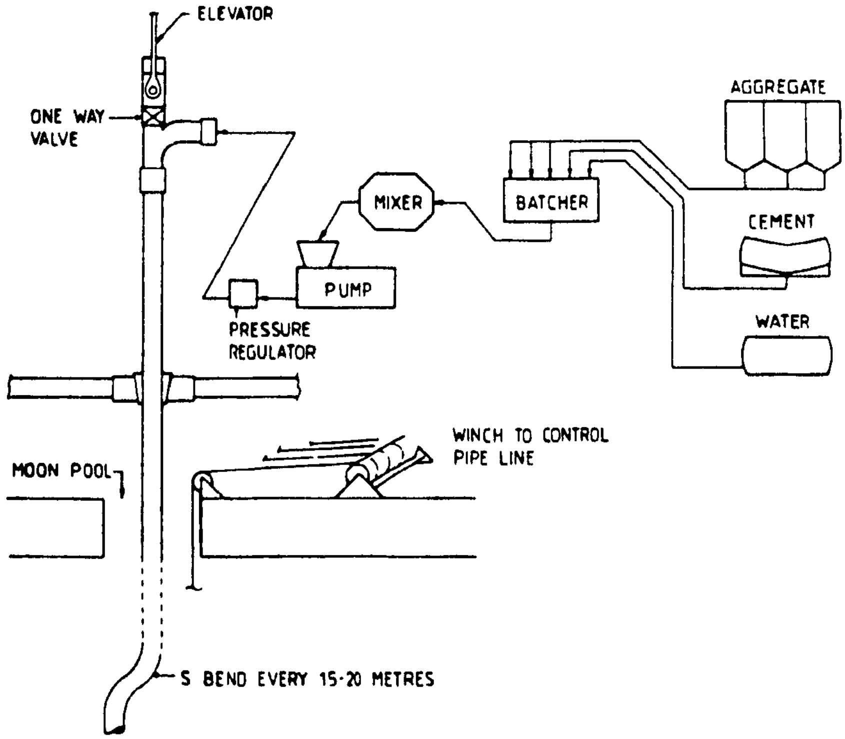


FIGURE 5.12 - TYPICAL PUMP LINE ARRANGEMENT FOR UNDERWATER CONCRETING[2]

The Kajima's Double Tube (KDT) tremie method, which was developed in Japan, is very similar to the hydrovalve method. The KDT method also uses a collapsible tube, but it is encased in a steel tube that has several vertical slits. The slits allow horizontal movement of the tube, often done when resetting the KDT. Field tests show that this method is reliable and inexpensive.[38] Further information on the KDT tremie method is given in Reference 50, and the typical procedure used is shown in Figure 5.14.

5.5.2.4 PNEUMATIC VALVES

The Abetong-Sabema[51] and the Shimizu[52] pneumatic valves are attached to the end of a concrete pump line (Figure 5.15). The valves are used to control the flow rate and amount of concrete that

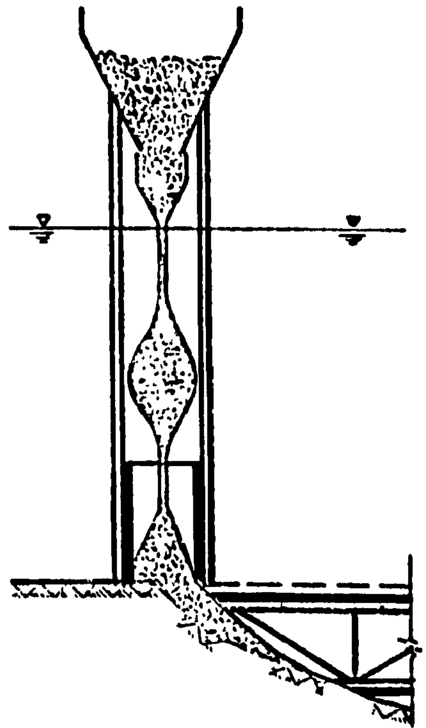
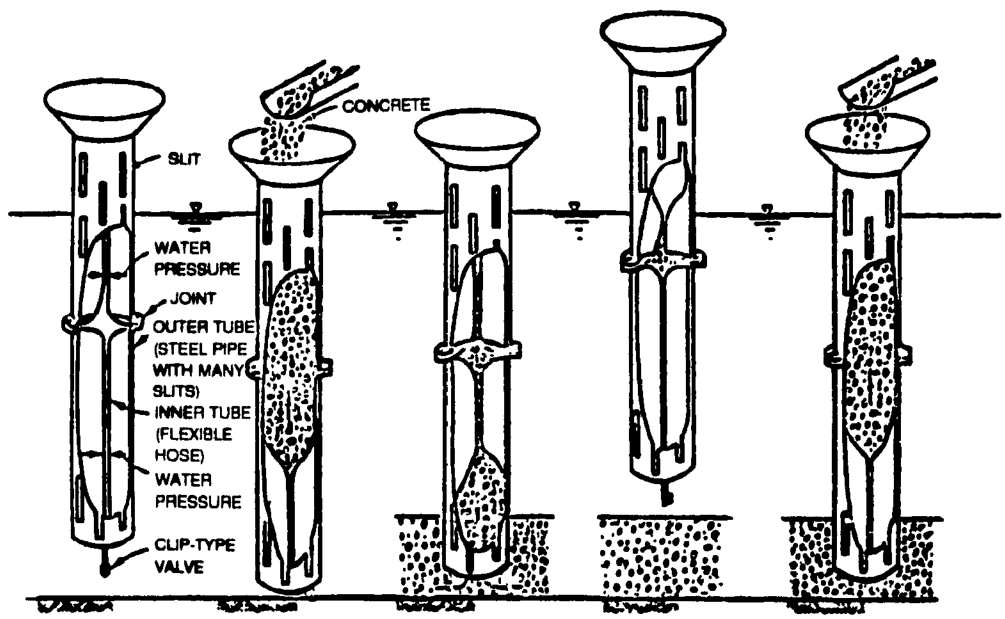


FIGURE 5.13 - TYPICAL HYDROVALVE APPARATUS[38]

is placed by, "permitting, restricting, or terminating" the flow of concrete through the pump line. When the pumping boom is moved, the valve is closed to protect the concrete in the line.[41]

The Shimizu pneumatic valve is similar to the Abetong-Sabema valve, but has a level detector attached to the valve unit. When the level detector senses that the concrete has reached a specified thickness, the valve closes and allows the tube to be repositioned. This method is currently considered to be one of the best methods for underwater repair.[38]

There is another type of check valve which is available for use in pumping underwater concrete. The valve, which fits a 125 mm (5 in.) diameter pump line, is 450 mm (18 in.) long and has only one moving part. The valve is constructed on a "gum rubber reinforced with nylon fabric plies"[38] and can operate in up to 52.7 m (173 ft.) of water with a maximum line pressure of 690 kPa (100 psi). With this valve, concrete can be placed without immersing the end of the hose in the freshly placed concrete.[38]



SET OF KDT TREMIE PIPE	FEED OF CONCRETE	INTERRUPTION AT CONCRETE FEED	WITHDRAWING OF THE PIPE	RESETTING OF THE PIPE AND SUCCESSIVE FEED OF THE CONCRETE
Inner tube is pressed flat by water pressure	Inner tube is distended by concrete which is poured	Inner tube is pressed flat again Last feed concrete is squeezed out.	Surrounding water does not enter in the inner tube	The water which presses the inner tube is turned out through the slits.

FIGURE 5.14 - TYPICAL KDT PROCEDURE USED FOR UNDERWATER CONCRETING[38]

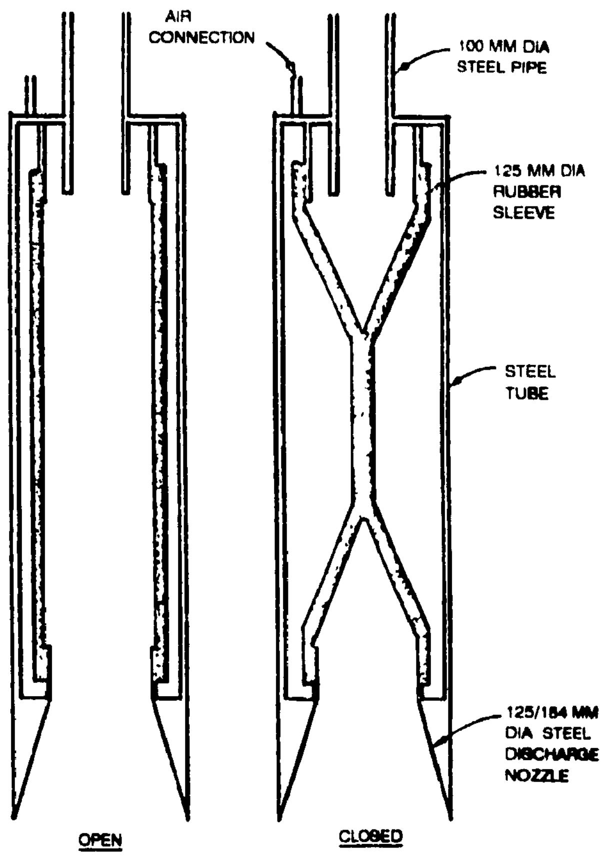


FIGURE 5.15 - ABETONG - SABEMA PNEUMATIC VALVE[38]

5.5.2.5 THE SKIP METHOD

Underwater concrete can be placed with the aid of bottom-opening skips (buckets). This method involves filling a bucket with concrete above water and slowly lowering it down through the water and discharging it at the repair area. To minimize washout of cement, the skip must be equipped with two overlapping canvas flaps, which are pressed against the top surface of the concrete by the water pressure. This prevents turbulence as the bucket is lowered through the water. After the bucket is lowered and is penetrated a small distance into the already placed concrete, the skip must be lifted slowly so that the discharged concrete does not intermix with the surrounding water as the bottom opens. The bucket should have bottom-opening double doors which can be operated automatically or manually.[33] As additional protection against washout, skirts fitted at the bottom may be used to confine the concrete while it is being placed (Figure 5.16).[2]

An advantage of this method is that very stiff, dense concrete can be placed if used in combination with vibratory or pressure compaction methods.[38] A slump value between 100 mm (4 in.) and 140 mm (5-½ in.) is commonly used.[30] Since the nature of the skip work subjects the concrete to a greater risk of washout, AWAs should be used. A recent laboratory study conducted in India concluded that when concrete is placed under water using the skip method, the use of superplasticizers results in segregation due to the air-entraining effect of the admixture.[35] Therefore, the use of HRWRs without the incorporation of AWAs and air-detraining agents should be avoided when using the skip method for underwater concrete placement. Although washout cannot be entirely eliminated, it can be minimized by applying the following additional precautions:[30]

- The skips should be completely filled.
- The skips should be raised and lowered slowly.
- The "advancing front" of the concrete should be built from the bottom upwards.
- A continuous supply of concrete should be provided to prevent layering or washout while waiting for the next batch (delays should not be more than 10 minutes).[33]

The main disadvantages with the skip method are its slow rate of operation and the small volumes of concrete they carry. Also, it may be difficult to place the concrete in formwork with small openings. In this case, divers are needed to control the placing of the skips. However, the skip method is used best where small volumes of concrete are needed at different locations or where mass concrete is required to stabilize the foundation of a structure.[2]

5.5.2.6 TILTING PALLET BARGE

The tilting pallet barge was recently developed by the Sibo group in Osnabruck, Germany.[38,41] This method is used to place thin layers of concrete in shallow water. The concrete is evenly spread on tilting pallets constructed along the deck of the barge and then dropped into the water in a free-fall. The method, which requires AWAs, can be adopted for use in deeper water by lowering a skip with tilting pallets to the repair area.

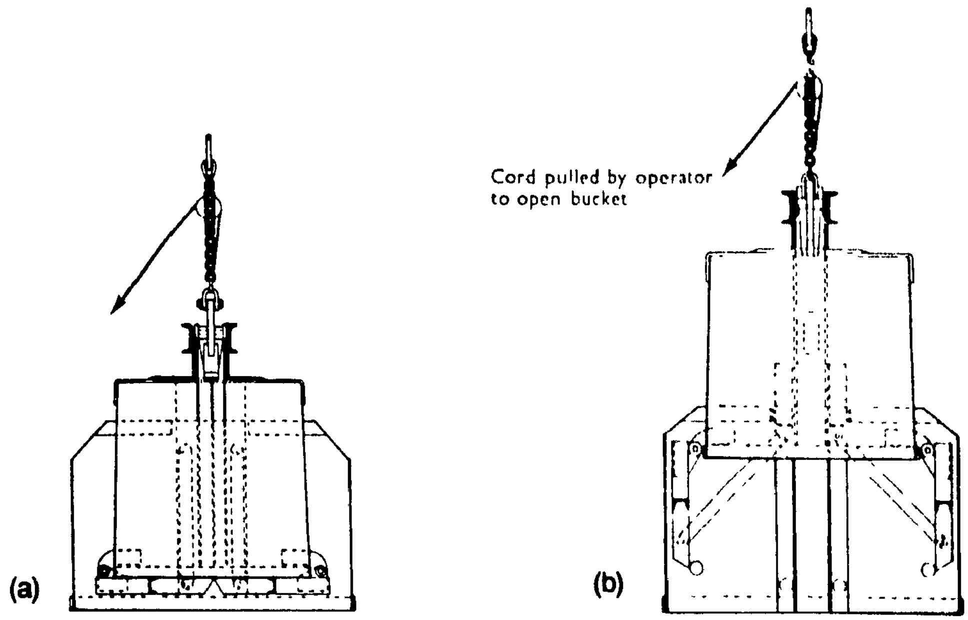


FIGURE 5.16 - BOTTOM-OPENING SKIP WITH SKIRT: (a) CLOSED; (b) OPEN[33]

5.5.2.7 PREPLACED AGGREGATE CONCRETE

Preplaced aggregate (PA) concrete is an effective way of repairing concrete structures under water, especially in areas where placement of conventional concrete would be either difficult or impossible.[2,7,9,38] In this technique, coarse aggregate is placed in formwork and a cementitious grout is slowly injected under low pressure from the bottom up, displacing water and filling the voids between the aggregate. The resulting high aggregate-cement ratio and point contact between aggregate particles produces a significantly lower shrinkage strain, typically 50 to 70 percent that of conventional concrete. Bonding strengths of PA concrete to existing concrete surfaces are between 70 to 100 percent of that attainable in conventional concrete.

The grout is injected at the bottom of the formwork to prevent the formation of air or water pockets. For this reason, the grout pipes are usually installed before the aggregate is placed and extend to the bottom of the formwork.[30] During injection, they are gradually withdrawn as the level of grout rises (Figure 5.17). Grout pipes may range from 19 mm (¾ in.)[2] to 35 mm (1-⅜ in.)[30] in diameter and are usually spaced no more than 1.5 m (5 ft.) apart.[7] Sounding tubes are often placed alongside the grout pipes so that the grout level can be monitored during placement. Alternatively, translucent panels can be provided in the formwork so that grout flow can be monitored.[2]

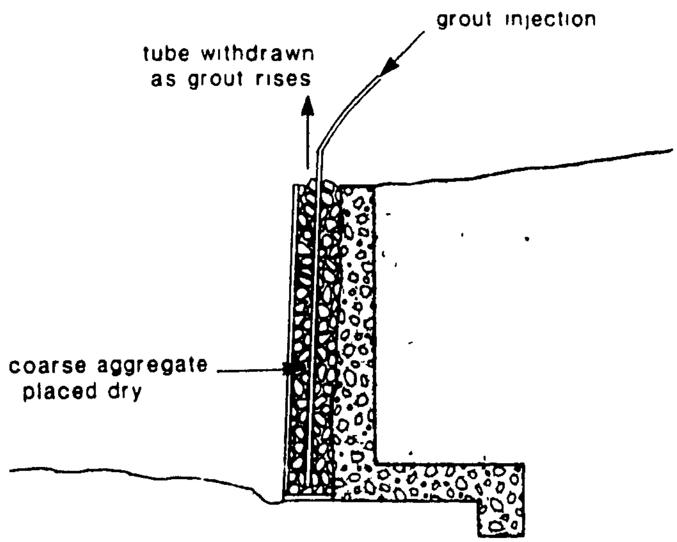


FIGURE 5.17 - PREPLACED AGGREGATE CONCRETE REPAIR TECHNIQUE[2]

If grouting from the bottom requires too great an injection pressure, injection tubes may be built into the formwork at several levels. In this case, the grout would be injected at the lowest inlet first and proceed upwards in a similar manner as with epoxy the injection method. For small repairs, injection can be done through an inlet pipe at the bottom of the form.[2] When grouting is completed, a pressure of about 70 kPa (10 psi) is held for several minutes to allow any remaining air and water to escape through a vent at the top of the formwork.[9]

For this method to be successful the formwork must be watertight and must be able to withstand the full hydrostatic pressure of the grout.[2] Vents must be provided at the top of the formwork to allow water to escape as the grout fills the form. If the forms are not sufficiently vented, back pressures will create voids in the concrete fill.[7] To prevent the loss of fines and cement at the top of the grout, the formwork usually completely encloses the aggregate. The top venting forms are usually made of a permeable fabric next to the concrete face, and backed with a steel grillage or wire mesh. This backing is attached to a stronger backing which is made of plywood and perforated steel to allow air to escape. The formwork is usually anchored with dowels to resist the uplift pressure generated by the grouting operation.[38]

Selection and proportioning of materials for PA concrete must be done very carefully. Aggregate must be graded so that grout can flow easily between the particle spaces. Coarse aggregate maximum size should not be larger than one-third the minimum thickness of the repair concrete,[53] but not smaller than 19 mm (¾ in.).[2] If aggregates smaller than 19 mm are used, the grout should not contain any sand to prevent "bridging" between voids, although this may cause bleed lens to form beneath the aggregate particles (Figure 5.18). The void content of the aggregate should be between 35 and 50 percent, which can be achieved by using uniformly graded aggregate. Each cubic yard of PA contains about 27 cubic feet (bulk volume) of coarse aggregate, compared with 18 to 20 cubic feet in conventional concrete.[53] A typical PA gradation is shown in Table 5.9.

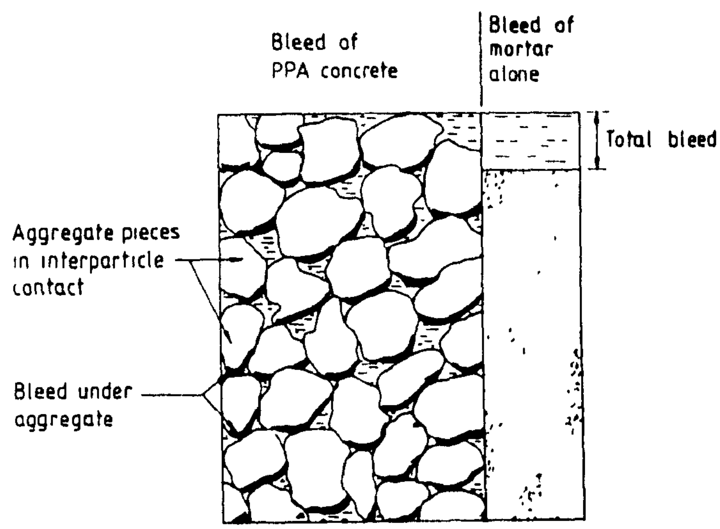


FIGURE 5.18 - SCHEMATIC REPRESENTATION OF BLEED LENS UNDER AGGREGATE[2]

TABLE 5.9 - GRADATION OF AGGREGATES FOR PREPLACED AGGREGATE CONCRETE[9]

Aggregate	U.S. Standard Sieve	Percent Passing (by weight)
Gravel	2 in.	100
	1-1/2 in.	90 to 100
	1 in.	20 to 55
	3/4 in.	0 to 15
	3/8 in.	0
Sand	№ 4	100
	№ 8	80 to 100
	№ 16	50 to 85
	№ 30	25 to 60
	№ 50	10 to 30
	№ 100	2 to 10

Several proprietary grouts for PA concrete are available and normally consist of portland cement, a pozzolan such as fly ash, fine aggregate, and a grout "fluidifier".[53] To minimize bleeding, fine aggregate (well-graded zone M sand)[2] is generally graded to a fineness modulus of 1.3 to 2.1, with most particles passing a № 16 sieve. The fly ash makes the grout pumpable and retards setting time. The grout fluidifier also retards setting time and keeps a low water-cement ratio, usually between 0.42 and 0.50. The fluidifier is effective because it produces an expansive gas which prevents the buildup of bleed water under the coarse aggregate. The retarder found in most fluidifiers provides about two percent entrained air, which improves the durability of the hardened concrete. Fluidifier is normally added at a rate of one percent by weight of the total cementitious material in the grout.[53] Also, adding AWAs to the grout avoids the construction of expensive formwork.[38]

Preplaced aggregate concrete can also be made by using epoxy resin instead of cementitious grout. Although it is significantly more expensive, epoxy resin is advantageous for several reasons: it has a very small particle size (typically less than $100\mu m$), variable viscosity, and variable pot life. This results in a material which is more versatile and can be used with much smaller coarse aggregate sizes.[2]

5.5.2.8 TOGGLE BAGS

This method consists of lowering small volumes of concrete in bottom-opening canvas bags. The bags are reusable and are sealed at the top with a chain or rope and secured with a toggle.

When the bag is located over the repair area, the bottom is released to discharge the concrete. Placing concrete with toggle bags involves the same procedures as for using bottom-opening skips with regards to preventing cement washout.[2,33]

5.5.2.9 BAGGED CONCRETE

In this method, fresh concrete is placed into bags and then placed under water by divers. This method is normally used to repair scour, renew ballast,[2] or as a temporary repair measure to seal holes and construct expendable formwork.[33] The bags are made of a strong fabric (usually hessian) and are usually of 10 to 20 liters (2-½ to 5 gal.) capacity when the work is performed by divers. The concrete used has a slump of between 19 and 50 mm (¾ and 2 in.) and the maximum aggregate size is approximately 40 mm (1-½ in.). If smaller bags are used (5 to 7 liters/1.28 to 1.8 gal.), maximum aggregate size should not exceed about 10 mm (⅜ in.).[30]

The bags are usually placed in a brick bond fashion and are half-filled to ensure good interlocking to form a solid structure. The cement paste which seeps out from between the weave of the fabric provides adhesion between the individual bags.[33]

Grout bags were recently used to change the downstream slope of a small dam to eliminate a dangerous undertow.[54] The bags were made out of polyester and measured 1.8 m (6 ft.) wide by 0.6 m (2 ft.) thick. The length of the bags varied between 2.1 m (7 ft.) and 7.3 m (24 ft.). To interlock the bags together 0.76 m (2-½ ft.) long epoxy coated bars were forced through them at about 1.8 m (6 ft.) spacings.

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CHAPTER 6

WHARF AND BRIDGE SUBSTRUCTURE REPAIR

6.1 INTRODUCTION

The maintenance or repair of the submerged portion of a wharf or bridge substructure requires a clear understanding of what caused or is causing distress or deterioration. As described in Chapter 2, the causes are many and the sequence in which the various processes occurred are difficult to determine. Frequently, underwater damage or deterioration originates from design errors and poor construction practices. Reinforced and prestressed concrete piles supporting marine facilities or offshore structures are often subjected to severe loadings from winds, waves, currents, ice, chemical and biological attack, and ship impact. Bridge piers and abutments are often undermined due to scour caused by floodwaters or changed channel flow conditions.

Evaluating these structures requires consideration of the natural phenomena which are present in a marine environment and the effects these have on the substructure.[1] Distress or deterioration which continues may require an on-going investigation process in order to obtain a complete understanding of the type, cause, extent, and rate of deterioration that is occurring. This may require a series of inspection and testing techniques at regular intervals which can provide guidance to the engineer for determining the optimum repair solution.[2]

There are not as many underwater repair procedures as there are for work above the water. Cleaning and surface preparation, which is essential for a successful repair, is more difficult under water. Underwater construction work is more difficult, slower, and its quality is less certain than for work above water. Deciding on a repair alternative requires expert engineering knowledge, experience and judgement. Some of the procedures available for underwater repair and maintenance of wharf and pier substructures are shown in Table 6.1. The basic steps in any underwater concrete repair are:[2]

(a) Cleaning the deteriorated concrete surface of all marine growth
(b) Removal of all deteriorated concrete and badly corroded reinforcing steel
(c) Replacing the removed reinforcing steel with new rebars
(d) Sealing any cracks by epoxy injection
(e) Replacing the concrete section with new concrete

(f) Applying protective surface coatings to the concrete

The following sections provide a summary of some of the repair methods listed in Table 6.1. The material in this chapter has been adapted from different available references, especially 3, 6 and 9.

TABLE 6.1 - REPAIRS AND PREVENTIVE MEASURES FOR UNDERWATER PIERS AND PILES*

Type of Repairs (Underwater and in Splash Zone	Nature of Problem				
	Scour	Deterioration	Damage (Structural)	Structural Failure	Foundation Distress
Replacement of Material	x				
Sheet Piling	x				
Training Works	x				
Modification of the Structure	x			x	x
Epoxy Injections		x	x		
Quick Setting Cement		x	x		
Epoxy Mortar		x	x		
Underwater Bucket		x	x	x	
Tremie Concrete		x	x	x	x
Prepacked Concrete		x	x	x	x
Pumped Concrete		x	x	x	x
Bagged Concrete		x	x	x	x
Cathodic Protection		x			
Pile Jackets		x	x	x	
Flexible and Rigid Barriers		x			
Oil Drum Method			x		

*Adapted from Reference 2

6.2 CONCRETE PILE REPAIR

There are several methods available for pile maintenance and repair and many of them are proprietary in nature and are variations of the same principle. These repair and maintenance methods can be grouped into six general categories: epoxy patching/injection, protective coatings, encapsulation (or wrapping), reinforced concrete jacketing, partial replacement, and cathodic protection. The epoxy patching/injection techniques presented in Chapter 5 are applicable to concrete pile repair, so they will not be discussed here.

6.2.1 PROTECTIVE COATINGS

Surface coatings are usually applied to concrete surfaces to act as a barrier against further deterioration. They prevent the ingress of corrosive chemicals but are also useful for resisting abrasion and freeze-thaw damage. For a coating system to be effective, proper surface preparation is important and the coating should have the following minimum characteristics:[3]

- It should have an adhesive strength greater than the tensile strength of the concrete
- The coefficient of expansion should closely match that of the concrete
- It should be fairly elastic and resist creep
- It should have a long durability

There are a great number of surface treatments available today for use in rehabilitating concrete surfaces. Most of these treatments can be categorized as penetrating sealers, coatings, or membranes.[4] It should be noted that some of these coating systems must be applied in dry conditions. In these cases, cofferdams may be constructed around the structure being repaired, so that the water can be pumped out to maintain dry working conditions. Table 6.2, which was developed by Bruner,[4] provides a selection guide to concrete surface treatments along with their performance characteristics. The following is a summary of the most commonly used surface coatings for protecting concrete piles in a marine environment.

6.2.1.1 ASPHALT AND TAR COATINGS

Asphalt coatings are highly resistant to acids and oxidants and can be applied cold with the use of a solvent. Tar is often used for repairing concrete in a marine environment, but it is not very resistant to acids and bases. It also does not offer a high degree of protection against abrasive action. Both coatings are applied in two coats; the second coat containing a silica filler compound for added stiffness.[3] Rubberized asphalt tape has also been used for protecting concrete piles.[1]

6.2.1.2 EPOXY COATINGS

Epoxy coatings have gained widespread use in the past decade primarily due to their good adhesive properties, low shrinkage, and high compressive strength. The epoxy is a two or three component system with 100 percent solids, consisting of a hardener and a base resin. Epoxy coatings are generally chemically inert, impervious to water vapor, and moisture resistant.[4] Epoxies are relatively easy to apply (usually with a brush or roller), but require strict quality control during mixing.

TABLE 6.2 - CONCRETE SURFACE TREATMENT SELECTION GUIDE[4]

Material / Property	Boiled Linseed Oil	Silane	Siloxane	Sodium Silicate	Penetrating Epoxy	Cementitious Coating	Epoxy Coating	Urethane Membrane
Ability to Penetrate	A	G	G	G	G	N/A	N/A	N/A
Ability to Bridge Cracks	N/A	N/A	N/A	N/A	N/A	P	VP	G
Ability to Bond to Concrete	N/A	N/A	N/A	N/A	N/A	G	G	G
Ability to Reduce Permeability	A	G	A	G	G	G	G	VG
Allow Water Vapor Transmission	A	G	G	G	P	A	VP	VP
Improve Aesthetics	VP	VP	VP	VP	A	VG	VG	VG

VG - Very good performance in meeting required property
G - Good performance in meeting required property
A - Average performance in meeting required property
P - Poor performance in meeting required property
VP - Very poor performance in meeting required property
N/A - Not applicable, not appropriate to address property

Epoxy compounds are temperature-sensitive and, once mixed, must be applied immediately due to their rapid setting characteristics. Epoxy coatings have an inherent tendency to creep, are impact sensitive, and have a strain incompatibility with the underlying concrete surface. Since epoxies are impermeable, any moisture trapped beneath the coating may cause it to blister and peel off if the coating is exposed to direct sunlight or freeze-thaw cycles. Accordingly, epoxies should not be used unless the concrete is capable of withstanding freeze-thaw cycles on its own.[3]

6.2.1.3 ACRYLIC RUBBER COATINGS

This type of coating has been used successfully for protecting concrete surfaces against reinforcement corrosion by preventing chloride ion penetration and carbonation of the concrete. The coating is a highly elastic rubber which reportedly performs two functions in chloride contaminated concrete: it acts as a barrier against further deterioration, and it allows chloride ions to move freely within the concrete pore matrix to prevent peak concentrations from occurring at the surface of the rebar. Further details of its configuration, behavior, and method of application have been reported by Swami and Tanikawa.[5]

6.2.2 ENCAPSULATION

Encapsulation or wrapping techniques are often used for repairing concrete piles that have minor surface deterioration and no significant loss of structural capacity. They act primarily as a protective barrier against further deterioration and isolate the pile from the various aggressive agents causing the deterioration. These repairs are generally performed as a maintenance item to extend the service life of the structure.[1] There are several proprietary repair systems available for encapsulating piles which can be grouped into two basic categories: impermeable plastic surface wrapping using polyvinylchloride (PVC) sheets, and molded glass fiber-reinforced plastic (FRP) jackets with epoxy grout. These are described below.

6.2.2.1 POLYVINYLCHLORIDE (PVC) WRAPS

This type of protection technique is widely used for timber pile repair but can be easily adapted for repairing or protecting concrete piles. Two commonly used proprietary systems, one consisting of a single-unit and the other a two-unit barrier wrap are illustrated in Figures 6.1 and 6.2, respectively. Commercially available PVC pile wraps can be purchased in prefabricated sizes to fit many pile sizes and lengths. Both systems require cleaning of the pile to remove all marine growth and soft surface concrete.

The two-unit system consists of an upper intertidal unit which starts at least one foot above mean high water (MHW) level and extends to at least 1 m (3 ft.) below mean low water (MLW). The bottom unit overlaps the intertidal unit a minimum of 300 mm (12 in.) and extends to below the mud line. The closure seam of the lower unit is rotated 90° from the upper unit seam. Polyurethane foam seals are installed at each end of the intertidal unit to prevent the ingress of water and air.[9] The basic installation procedure is as follows:

(a) The PVC wrapper is placed around the pile and tightened by rolling the ends of the vertical seam using poles and a ratchet wrench

(b) The wrapper is then fastened with regularly spaced aluminum alloy bands along the pile

(c) After the PVC barrier is installed, the area around the base of the pile is backfilled with bagged concrete. When the pile is surrounded by stones, the base can be backfilled with hydraulic cement.

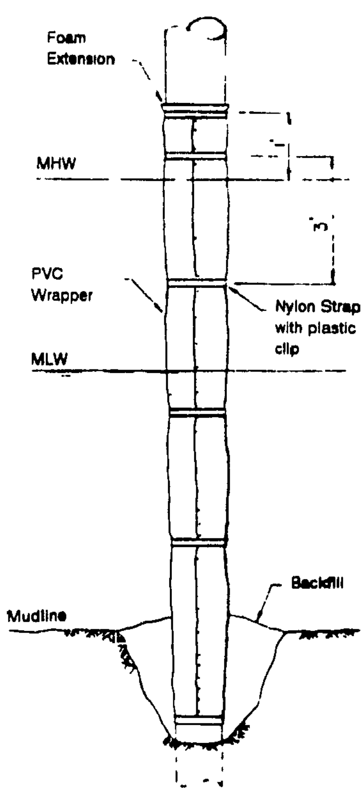


FIGURE 6.1 - SINGLE-UNIT PILE WRAP[9]

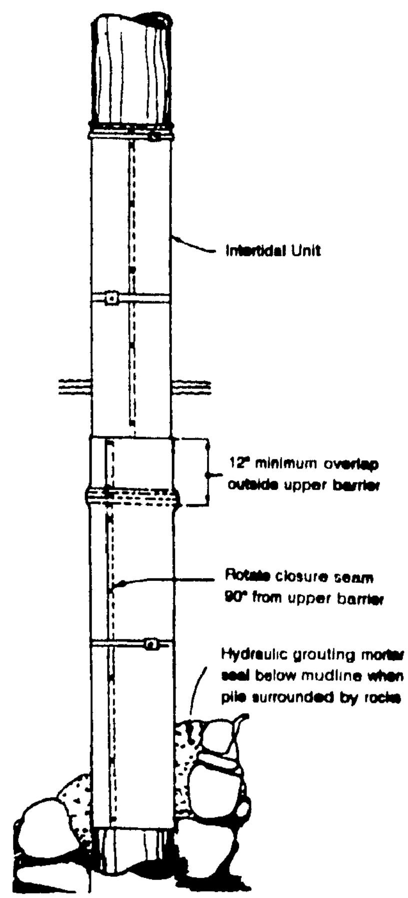


FIGURE 6.2 - TWO-UNIT PILE WRAP[9]

With the single-unit system, the full length of the pile to be protected is wrapped with a single jacket. Polyurethane foam seals are wrapped around the pile at the top and bottom end of the jacket. The installation procedure for installing the single unit system is similar to that of the two-unit system and is as follows:[9]

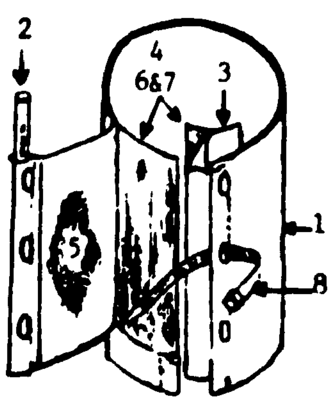
(a) The jacket is placed around the pile, and the closing zipper is started from the top.

(b) After installing the polyurethane seal at the top, the top nylon strap is installed and tightened to secure the jacket in place.

(c) The zipper is then closed continuously to the bottom, with nylon straps being installed at every 1 m (3 ft.) on center.

(d) If the base around the pile has been previously excavated, it can be backfilled using the same methods as for the two-unit system.

Another proprietary system (RETROWRAPS) developed by Cathodic Systems Inc., makes use of geo-synthetic technology and is shown in Figure 6.3. The system is specifically developed to protect piles in the splash zone and can be used on piles of any geometrical configuration. It is reportedly designed to withstand the deteriorating effects of ultraviolet radiation, environmental, ozone, and temperature variations for the design life of the system. The system can also be provided with an outer coating to prevent buildup of marine growth. The system is modular and is capable of encapsulating any length of pile. The butt joints of multiple units are sealed with a cummerbund unit to provide continuous encapsulation. The units can be removed for the purpose of inspecting the substrate and reinstalled without damage to the system itself.



1 Outer geo-membrane
2 Pultruded stiffeners
3 Inner sealing flap
4 Inner geo-textile membrane (bonded to 1 & 3)
5 Bonding agent
6 Thixotropic gel (containing 7)
7 Specified additive (corrosion inhibitor, biocides, conductive gels)
8 Cable ties

FIGURE 6.3 - SCHEMATIC DIAGRAM OF A PILE WRAPPING UNIT (RETROWRAPS, CATHODIC SYSTEMS, INC.)

Each unit is comprised of an outer geo-membrane bonded to an inner layer of geotextile fabric. The outer membrane is constructed of a nylon fabric encapsulated in polyurethane or polyether. The inner fabric is impregnated with a thixotropic gel that can be used to carry corrosion inhibitors, biocides or cathodic protection anodes. Once tensioned, the elastic properties of the membrane generate sufficient forces to push out oxygen and water from the substrate interface allowing the gel to form a homogeneous contact with the pile surface. These forces permit the system to "self heal", if punctured, by forcing the impregnated gel into the damaged area.

The units are installed in a similar manner as other types of wraps. At the leading edges of the fabric are pockets for inserting stiffeners used for sealing of the unit. A semi-rigid polypropylene inner sealing flap ensures a 360° seal at the loading edges. To facilitate proper installation, tensioning calipers (Figure 6.4) can be obtained. Typical specifications for the RETROWRAP pile encapsulation system are included in Appendix M.

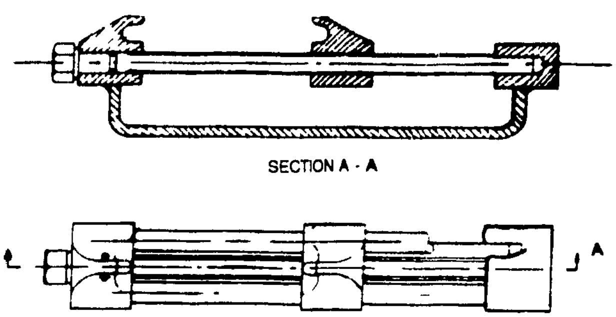


FIGURE 6.4 - TENSIONING CALIPERS (RETROWRAPS - CATHODIC SYSTEMS, INC.)

6.2.2.2 FIBER-REINFORCED POLYMER (FRP) JACKETS

Polymer pile encapsulation is the state-of-the-art method for protecting and resurfacing concrete piles. The system consists of pumping epoxy grout into rigid encapsulation jackets that are custom fabricated to precisely fit the pile for each job. The encapsulation is highly corrosion resistant, has a very low permeability, and possesses high compressive, tensile, and impact strengths. They are relatively easy to install and, if properly installed, they can provide a long service life.[6]

A typical polymer pile encapsulation system consists of two symmetrical fiber glass reinforced polyester or vinylester jacket units (Figure 6.5) each with a minimum thickness of 3 mm (⅛ in.). The units can be either rectangular or circular in shape and are sized slightly larger in dimension than the pile being repaired. The two units are joined together around the pile to form a grout space between the pile and the jacket. Each individual unit is fitted with polymer 'stand-offs' bonded to the interior of the jackets to provide a uniform annulus between the pile and the jacket. The stand-offs are cone-shaped to minimize pile contact area.

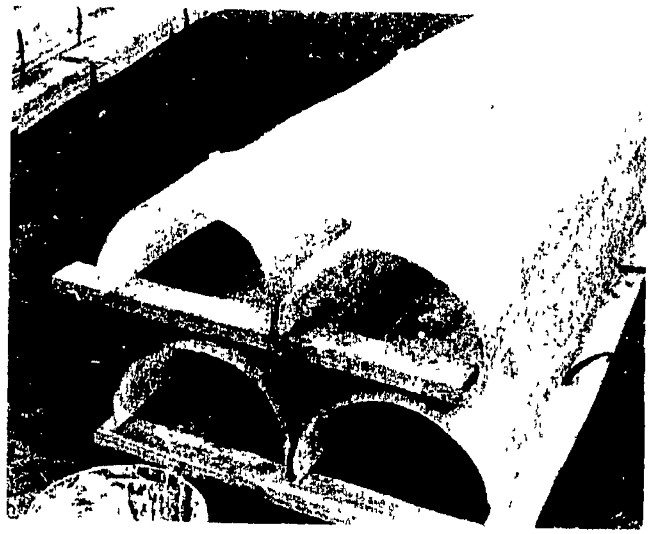


FIGURE 6.5 - TRANSLUCENT FRP JACKETS[6]

Once installed, the bottom of the jacket is sealed and the epoxy grout is pumped through an injection port located at the bottom of the jacket. The principal components and the various phases of a successful pile encapsulation system are shown in Figures 6.6 and 6.7, respectively. A standard treatment of the remaining portion of the pile above the FRP jacket, which is often used by the Florida State Department of Transportation, is illustrated in Figure 6.8.

Polymer pile encapsulation was successfully used to repair rock borer damage to 1.37 and 1.68 m (4.5 ft. and 5.5 ft.) outside diameter precast, post tensioned concrete cylinder piles of a trestle located in the Arabian Gulf.[7] Several alternative repair schemes were evaluated and it was determined that FRP encapsulation (Figure 6.9) was the most viable option and was chosen due to the following reasons: they are suitable for installation in open sea, they are impermeable to

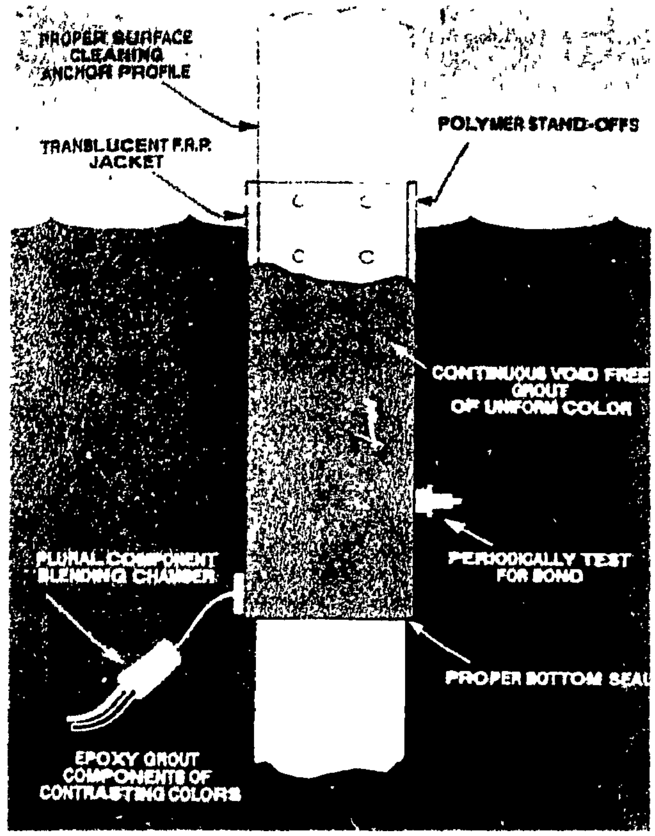


FIGURE 6.6 - PRINCIPAL COMPONENTS OF A PILE ENCAPSULATION SYSTEM
(A-P-E, MASTER BUILDERS, INC.)[6]

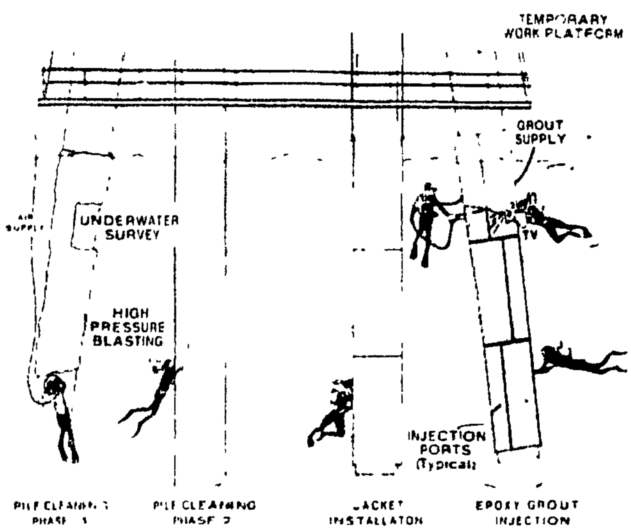


FIGURE 6.7 - VARIOUS PHASES OF A POLYMER ENCAPSULATION SYSTEM ARE SHOWN
FROM LEFT TO RIGHT (A-P-E, MASTER BUILDERS, INC.)[6]

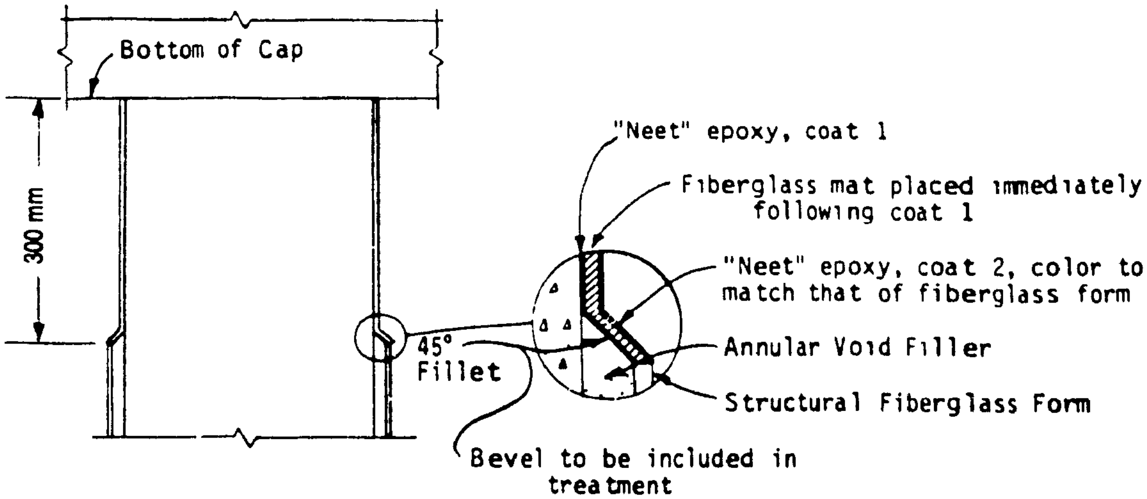


FIGURE 6.8 - DETAIL OF STANDARD TREATMENT OF TOP 300 MM OF CONCRETE PILING (FLORIDA STATE DEPT. OF TRANSPORTATION)[2]

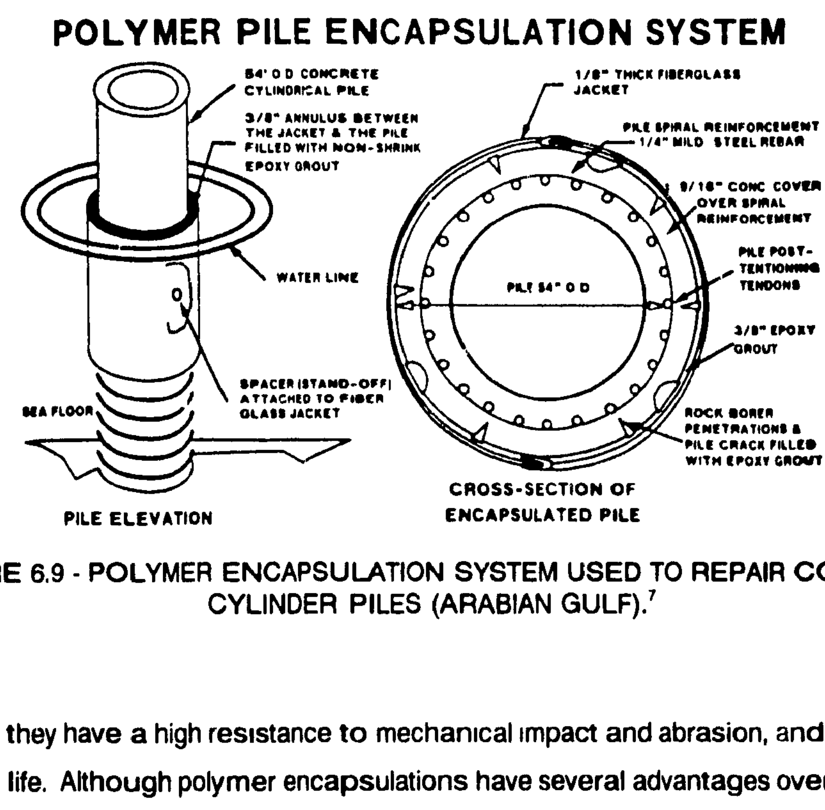


FIGURE 6.9 - POLYMER ENCAPSULATION SYSTEM USED TO REPAIR CONCRETE CYLINDER PILES (ARABIAN GULF).[7]

rock borers, they have a high resistance to mechanical impact and abrasion, and their potentially long service life. Although polymer encapsulations have several advantages over other systems, a wide range of problems have been encountered on a number of projects. Based on field observations of a large number of marine concrete structures, combined with various field and laboratory tests, Snow[6] has developed a list of causes influencing performance and has provided possible remedies for minimizing the problems. Below is a brief summary of the findings.

(1) DISCONTINUITY OF POLYMER GROUT

Discontinuity of grout was observed at or near the original water elevation at the time of construction. In some instances, voids developed in areas at lower elevations. The phenomenon was attributed primarily to entrapped air as the grout was poured into the jacket from the top. Another principal cause of grout discontinuity is not providing an adequate number of stand-offs to maintain the proper spacing between the jacket and the pile, thereby restricting grout flow. These deficiencies can be easily corrected by:

(a) Pumping the epoxy grout into the jacket from the bottom.

(b) Using translucent jackets so that grout flow can be monitored and the necessary corrections can be made before the grout sets.

(2) LACK OF BOND BETWEEN POLYMER GROUT AND SUBSTRATE

The most common cause for bond failure between the grout and the substrate was attributed to improper surface preparation, and the presence of biofilms on the submerged surfaces which begin to develop immediately after cleaning. As a result, the following precautionary measures are suggested:

(a) Use of proper surface preparation techniques including a suitable pile "anchor profile". This is usually accomplished by using sand blasting or abrasive rotary tools.

(b) Preparation of substrate, installation of jackets, and pumping of grout into the jackets within the shortest time-frame possible, preferably less than 36 hours.

(c) Pumping the epoxy grout into the translucent jacket from the bottom up through injection ports provided in the jacket.

(3) LACK OF BOND BETWEEN POLYMER GROUT AND FRP JACKET

Bond failure between the grout and the FRP jacket is similar to the lack of bond between the grout and the pile. The most probable causes are reported to be improper preparation of the inside surface of the jacket, the presence of mold release agent residue on the jackets, marine biofilms, or inadequate compaction of the grout during placement. In addition to implementing preventive measures (b) and (c) in Section (2), the following are also suggested.

(a) The inside of the jackets should be roughened at the site immediately prior to installation, preferably by light grit blasting.

(b) Use of jackets with protective liners. Some jacket manufactures provide jackets with a liner that leaves a rough finish on the inside surface of the jacket when it is peeled off

(4) IMPROPER MIXING AND/OR CURING OF POLYMER GROUT

The field observations showed that several of the encapsulations failed as a result of improper mixing of the grout components. This resulted in soft spots of uncured materials which easily peeled off the substrate. To address this problem, the following solutions were suggested:

(a) Selection of different color (i.e., black and white) epoxy components. If the components are properly mixed, they will produce a uniform different color (grey) without any streaks of the original colors.

(b) The grout should be mixed and pumped by the "plural component method" using commercially available dispensing machines.

(5) THERMAL INCOMPATIBILITY OF THE POLYMER ENCAPSULATION WITH THE SUBSTRATE

Polymer encapsulation materials have a much higher coefficient of expansion than the concrete substrate. As a result, the materials become partially debonded and in some cases, the FRP jacket is removed by wave action. However, when polymer encapsulations are properly installed, debonding caused by thermal incompatibility is apparently eliminated. Therefore, to minimize distress caused by temperature fluctuations, the following procedures should be adopted:

(a) Implementing all the procedures previously outlined.

(b) Performing in-situ tests on the completed encapsulation to ensure that good bond has been achieved.

(c) Installing encapsulations during the summer season so that the polymer materials will be placed in a more expanded state and their subsequent cooling will create a tightly bonded system. If the encapsulation is installed during colder periods, the grout should be heated and warm water should be pumped into the installed jacket prior to grout injection. This will simulate installation during summer months.

(6) ULTRAVIOLET (UV) DETERIORATION OF FRP JACKET

Most of the jackets that failed as a result of UV deterioration appeared to be fabricated with glass fibers. However, in more recent applications, jackets fabricated using a combination of woven roving, mat fibers, and an outside gel coat, have shown increased resistance to UV deterioration. Translucent jackets 3 mm (⅛ in.) thick, consisting of one woven roving and two mats plus a gel coat, have passed 500-hour accelerated weathering tests without any significant damage. The following suggestions will help to minimize the effects of UV deterioration:

(a) Using "hand laid up or pultruded jackets" with adequate resin cover over the glass fibers.

(b) Add UV screening agents to the jacket resins at the time of fabrication

(c) For severe UV exposure, coat the completed encapsulation with a compatible polyurethane paint to block the UV rays.

(7) IN-SITU BOND TESTING

As previously stated, the most prevalent cause of distress in polymer encapsulations is debonding of the materials. To ensure that good bond is achieved, periodic in-situ bond tests, both above and below the water line can be conducted using the modified Elcometer Bond Strength Tester (Figure 6.10). It is a field test device that determines direct tensile bond on an isolated section of a completed encapsulation. The tester applies a calibrated tensile load on a 80 mm (3-⅛ in.) diameter test "dolly" that is glued to the outside of the FRP jacket. Prior to applying the load, the section being tested is isolated from the rest of the encapsulation by cutting a circular groove around the dolly down to the substrate surface.

Results of several hundred tests performed with the device indicate that the bond between polymer materials is usually greater than the bond between the grout and the pile. Hence, a well bonded system is achieved when failure occurs at or below the grout-to-substrate interface.

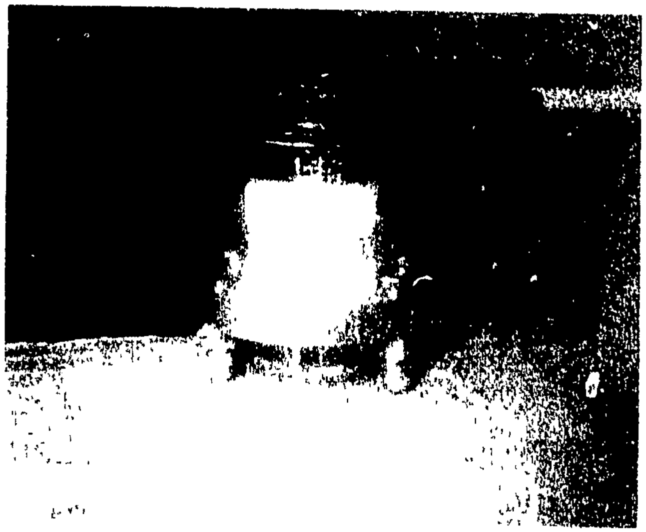


FIGURE 6.10 - THE MODIFIED ELCOMETER BOND STRENGTH TESTER[11]

6.2.3 REINFORCED CONCRETE JACKET

This repair method is used for piles that have undergone significant loss in cross section and can no longer support the design service load. Reinforced concrete jackets are used to prevent further deterioration and restore the service load capacity of the piles.[1] Depending on the degree of deterioration, the steel reinforcement used in the repair can be wire mesh, standard deformed bars (epoxy coated or uncoated), or a combination. The basic technique requires cleaning the pile and removing all deteriorated concrete, installing steel reinforcement around the damaged areas, placing a jacket around the pile, and filling the annular space between the jacket and the pile with concrete.

Several proprietary systems for repairing submerged piles have been developed but all basically involve the technique outlined above. Most commercially available systems use flexible fabric jackets, while rigid forms have also been used extensively.[9] For each system, casting underwater concrete around the piles and reinforcing requires special techniques and proper material proportioning and selection. Submerged piles can also be repaired using the preplaced aggregate and bagged concrete methods.[8]

6.2.3.1 FLEXIBLE FORMS

An example of a commonly used proprietary system (SeaForms) using flexible fabric jackets is illustrated in Figure 6.11. After the pile is cleaned, a wire reinforcing mesh is placed around the pile using 75 mm (3 in.) PVC spacers to provide a grout space between the pile and the form as shown in Figure 6.12. The fabric is then placed around the pile, the zipper is closed, and the form is secured to the pile at the top and bottom with mechanical fasteners so that it does not slide down during concrete placement. The concrete is pumped into the form from the top through openings (seacocks) supplied in the fabric using a suitable hose which is extended down to the lowest point in the jacket During concrete placement, the form should be jostled to make sure the concrete settles uniformly in the form. When the form is full, the pump hose is removed and the seacocks are sealed.[9] Figure 6.13 illustrates the various steps involved in the procedure.

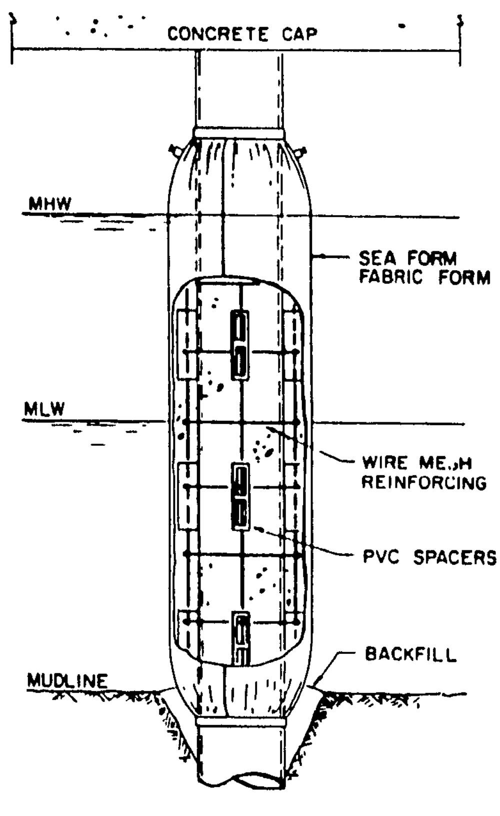


FIGURE 6.11 - FLEXIBLE FABRIC JACKETS[9] (SEAFORMS)

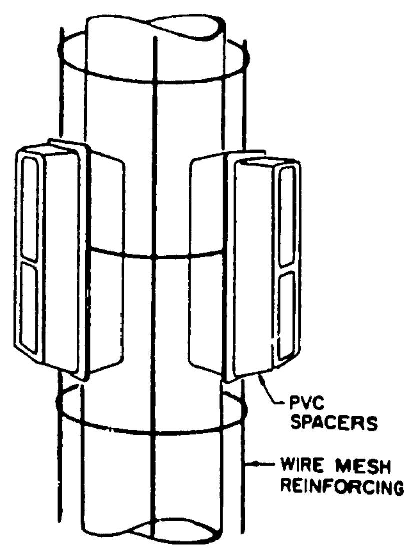
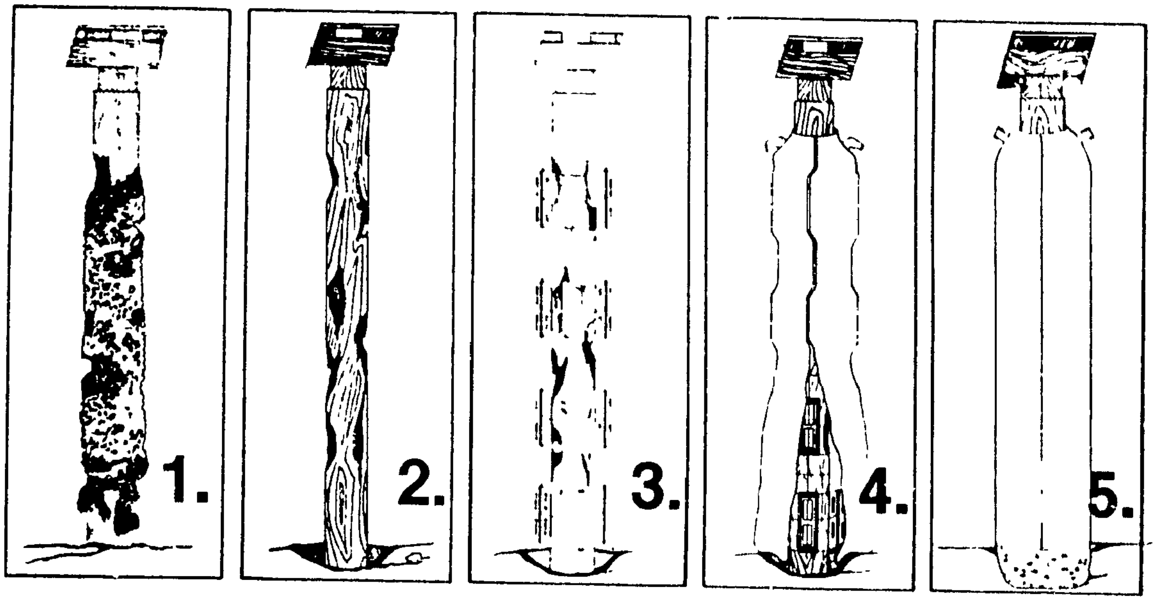


FIGURE 6.12 - FABRIC FORM SPACERS[9]



(1) A typical piling heavily encrusted with marine life. (2) Encrusted and deteriorated pilings are cleaned of all sea life. Wooden pilings are scraped. Steel H-beam, and sometimes, concrete pilings are cleaned with a high pressure hydro blaster. (3) The clean pilings are surrounded with reinforcing Standoffs maintain the spacing between the piling, the reinforcing, and the outer section of the form. (4) A ballistic nylon form is zippered closed over the structure, and attached to the piling at the top and bottom. Each reinforcing structure and form is pre-fabricated, pre-numbered and measured for each piling (5) A special concrete mix is tremie pumped into the form through the seacocks located at the top

FIGURE 6.13 - THE VARIOUS PHASES OF THE SEAFORMS PILE ENCAPSULATION SYSTEM (AQUATIC MARINE SYSTEMS, INC.)

6.2.3.2 RIGID FORMS

Many types of rigid form systems have been used for repairing submerged piles. The most common type used by marine contractors is the split fiberglass-reinforced polyester jacket,[9] illustrated in Figure 6.14. The jackets are installed around the pile and locked with a "z-bead" closure. A minimum spacing of 38 mm (1-½ in.) is maintained between the pile and the wire mesh reinforcing, and between the reinforcing and the jacket. Reinforcing bands installed at regular spacings along the length of the pile stabilize the jacket during concrete placement. If the repair area extends below the mud line, a water jet or airlift (Chapter 7) can be used to excavate the required cavity for installing a base seal. The concrete can either be placed from the top by the tremie method or pumped through a valve at the base of the jacket. The grout fill is topped off with an epoxy cap troweled at a 45° angle. The area around the base of the pile should be backfilled [9]

A rigid form system using split fiberboard can also be used. This system is very similar to the fiberglass system described above, and is illustrated in Figure 6.15. The forms can be fitted with a closure at the lower end and suspended from the top, or the end closure may be installed first and suspended from above to support the forms.[9]

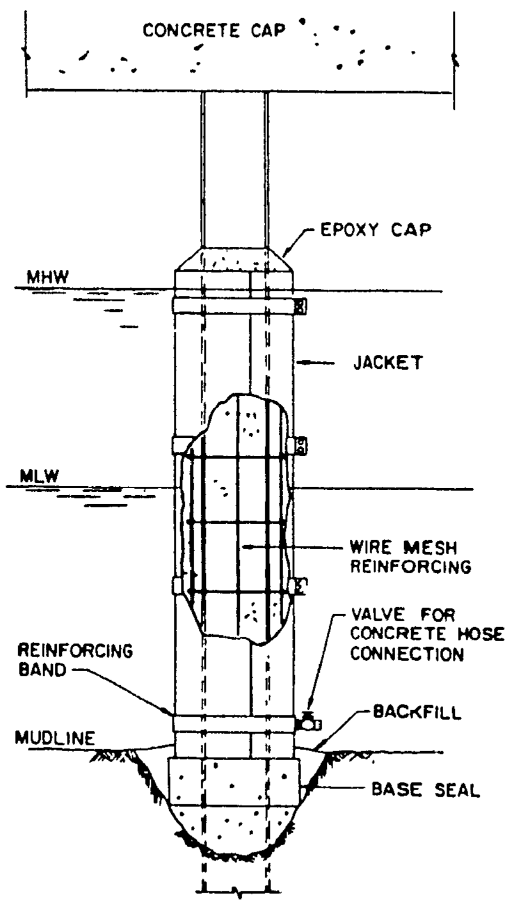


FIGURE 6.14 - FIBERGLASS RIGID FORM[9]

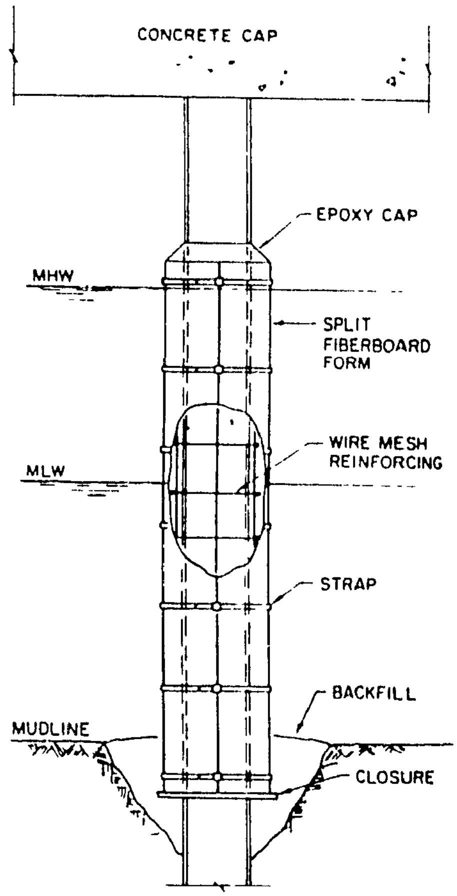
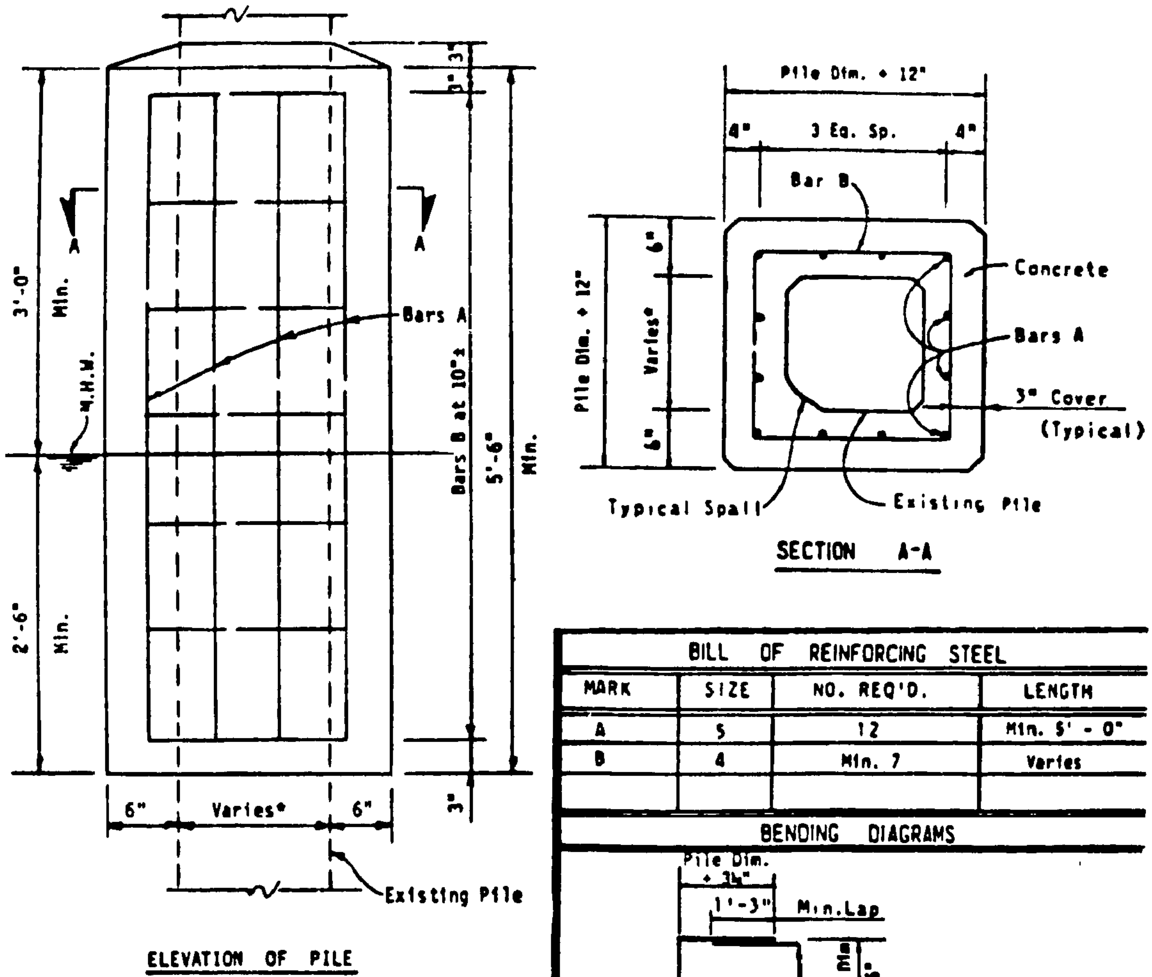


FIGURE 6.15 - FIBERBOARD RIGID FORM[9]

For piles which have deteriorated to the point where the structural integrity is in question, the jacket is reinforced with standard deformed reinforcing bars. The Florida State Department of Transportation has used concrete jackets reinforced with epoxy coated rebars. The jackets are cast by the conventional method using fabricated plywood forms[2] A typical reinforcing design is illustrated in Figure 6 16.

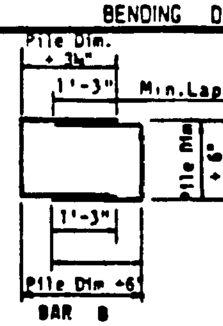
In a recent pilot test program at the Port of Oakland in California, a semi-rigid fiberglass tubular jacket with one longitudinal seam was used to repair test piles.[8] The reinforcing steel consisted of 150 mm x 150 mm (6 in x 6 in.) welded wire fabric and 20 M (15 mm) longitudinal rebars at approximately 230 mm (9 in.) spacing wrapped around the repair area. The semi-rigid jacket was fitted around the pile, and positioned with top and bottom centering devices. A bottom seal was installed and the jacket was tightened with steel bolts placed in closely spaced holes along the longitudinal seam. The concrete was placed from the top using a 50 mm (2 in.) diameter PVC tremie pipe extending down to the bottom of the repair area. The added advantage in using this system is that the jackets can be easily removed and repositioned for reuse, permitting repairs to be done in multiple lifts.



BILL OF REINFORCING STEEL

MARK	SIZE	NO. REQ'D.	LENGTH
A	5	12	Min. 5' - 0"
B	4	Min. 7	Varies

BENDING DIAGRAMS



NOTE: All bar dimensions are out-to-out.

FIGURE 6.16 - CONCRETE PILE JACKET (FLORIDA STATE DEPARTMENT OF TRANSPORTATION)[2]

6.2.3.3 PRECAST SHOTCRETE

Precast concrete half-cylinders with a wall thickness of 75 mm (3 in.) and reinforcing mesh projecting from the sides and ends of each modular unit have been used successfully for protection of piles.[2] Although it was used for protecting timber piles, the method can be easily adapted for protecting concrete piles. The concrete cylinder halves are placed around the pile above the water and the projecting reinforcing mesh is twisted together to make a complete unit. The end and side joints are then sprayed with concrete (gunited) and the complete unit is lowered into the water. A second unit is made in the same manner and placed on the first one. This procedure is continued until the concrete jacket is jetted to the desired depth below the mud line. The annulus which is formed between the jacket and the pile is then filled with grout.

6.2.3.4 OIL-DRUM METHOD

This method was successfully used for many years by the Port of Oakland as a standard method of repair for timber piles, but can also be used to repair concrete piles.[2] The procedure involves replacing a major portion of the length of the pile with new concrete. A 190 liter (50 gal.) steel oil drum with a hole the size of the pile is cut in the bottom and is fitted around the pile and filled with concrete. Reinforcing can be installed, if required, and the oil drum is filled with tremie concrete. Polyethylene sheets are usually wrapped around the pile before placing the concrete to obtain a tight, oxygen-free seal adjacent to the pile surface.

6.2.4 PARTIAL REPLACEMENT

This repair alternative is needed when the piles have deteriorated to the point where they can no longer support any load. In this type of repair, the deteriorated portion of the pile would be removed and replaced with a new load transferring mechanism to restore its full service load capacity. A common technique used by contractors involves installation of steel pipe jacks between sound portions of the pile and encasing the member with a reinforced concrete jacket,[1] as shown in Figure 6.17.

In rare cases, replacing the deteriorated pile with a new one may be more economical. One method which is useful for concrete deck structures consists of cutting a hole in the deck between existing pile locations and adjacent to the deteriorated pile. A new concrete pile is driven through the hole and cut off below the top of the deck, and a concrete cap is poured under the deck around the new pile to ensure adequate load transfer. This method is illustrated in Figure 6.18.

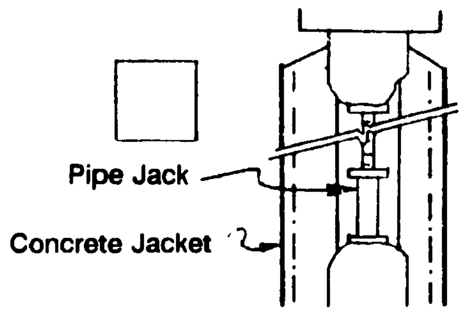


FIGURE 6.17 - PARTIAL REPLACEMENT OF PILE WITH PIPE JACK AND CONCRETE JACKET[1]

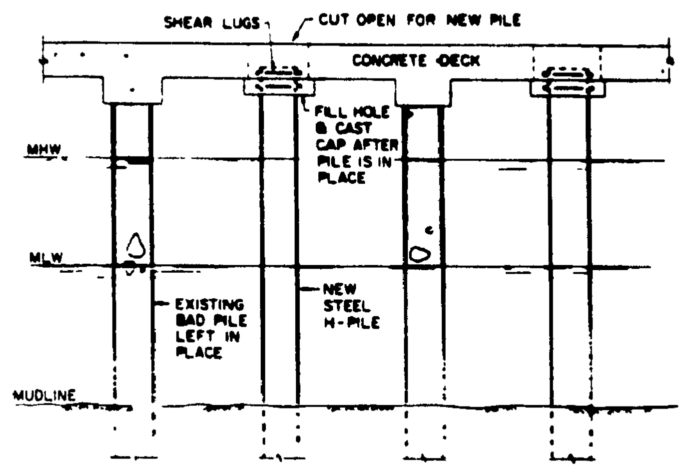


FIGURE 6.18 - ADDITION OF NEW PILE TO EXISTING STRUCTURE[9]

6.2.5 CATHODIC PROTECTION

Cathodic protection (CP) is an electrochemical method used to stop or decrease the rate of steel corrosion. It is frequently used to protect concrete located in seawater by making the embedded reinforcing steel cathodic with respect to the concrete. A direct current is applied between the reinforcement, which acts as the cathode, and a permanent anode mounted into or on the concrete surface, eliminating electric potential differences along the steel surface. Cathodic protection in a marine environment can be applied in two basic ways: the galvanic (sacrificial) anode system, and the impressed current (inert anode) system.[2] These systems are shown schematically in Figure 6.19, and a brief description of each follows. Further background to these systems is available in Reference 10.

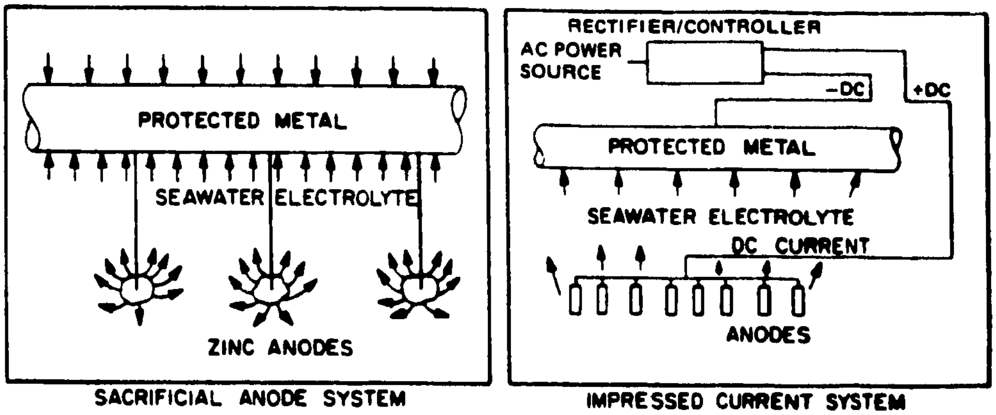


FIGURE 6.19 - SCHEMATIC REPRESENTATION OF CATHODIC PROTECTION SYSTEM[9]

6.2.5.1 GALVANIC ANODE SYSTEMS

A galvanic anode system consists of electrically connecting a sacrificial anode to the reinforcing steel and immersing it in an electrolyte (in this case seawater). The potential difference which is created between the anode and the structure cathode, consumes the anode to produce the electric current which keeps the structure in a cathodic state. Metals with high potentials such as zinc, magnesium, and aluminum have all been used effectively as sacrificial anodes.[10]

6.2.5.2 IMPRESSED CURRENT SYSTEMS

An impressed current system is similar to a battery in which the anodes, made of high-silicon cast iron or graphite, are connected to an external DC power source to produce the electric current.

The anodes are installed in the electrolyte and connected to the positive terminal of the DC source, while the structure being protected is always connected to the negative terminal.[10] This method is also termed the "rectifier type".[2] The resulting impressed current slowly consumes the anodes and is often accompanied by gaseous reaction products.[10]

6.2.5.3 GALVANIC ANODE SYSTEM VS. IMPRESSED CURRENT

The selection of the specific cathodic protection system to be used is often a difficult process which requires careful economic consideration. Besides economic consideration, other determining factors include cathodic interference, lack of power, or current requirements. For instance, in seawater, the current requirement is usually 54 to 108 mA/m^2 (5 to 10 $mA/ft.^2$) of exposed area while that of freshwater is in the range of 11 to 32 mA/m^2 (1 to 3 $mA/ft.^2$). Because seawater is a very low resistivity environment, the voltage required to produce the current is also low.[2]

The galvanic anode system is usually used in lower resistivity environments, whereas the impressed current system can be used in almost any resistivity environment. Due to concrete characteristics, the impressed current system is usually preferred for cathodic protection of reinforced concrete structures. However, when maintenance or access is difficult, or when DC power is unavailable, the use of sacrificial anodes may be the best solution.[10] Before a decision can be made on which type of CP system to use, a complete engineering and economic analysis must be made. The following is a list of some of the advantages and disadvantages of the two systems as reported by Lamberton et al.:[2]

(a) Galvanic Anode System

Advantages:

- It does not require an external power source
- Adjustment is not required after the proper current drain is determined
- It is easy to install
- Cathodic interference is minimal
- It requires very little maintenance during the life of the anode
- The current can be delivered uniformly over a long structure
- Overprotection at drainage points is minimized
- It is easy to estimate the cost

Disadvantages:

- It has a limited current output
- It is not economical in high resistivity environments
- It requires numerous anodes to protect a large structure

(b) Impressed Current System

Advantages:

- The system can be designed for a wide range of applied voltage.
- The system can be designed for a wide range of current requirements.
- A simple installation can protect a large area.
- The applied voltage and the current output can be varied.
- The current drain can be easily monitored at the rectifier.

Disadvantages:

- There is a risk of cathodic interference current from other structures.
- Its operation is affected by power failures.
- Electrical inspection and maintenance are required.
- It is very difficult to estimate cost because of the numerous possible design variations.

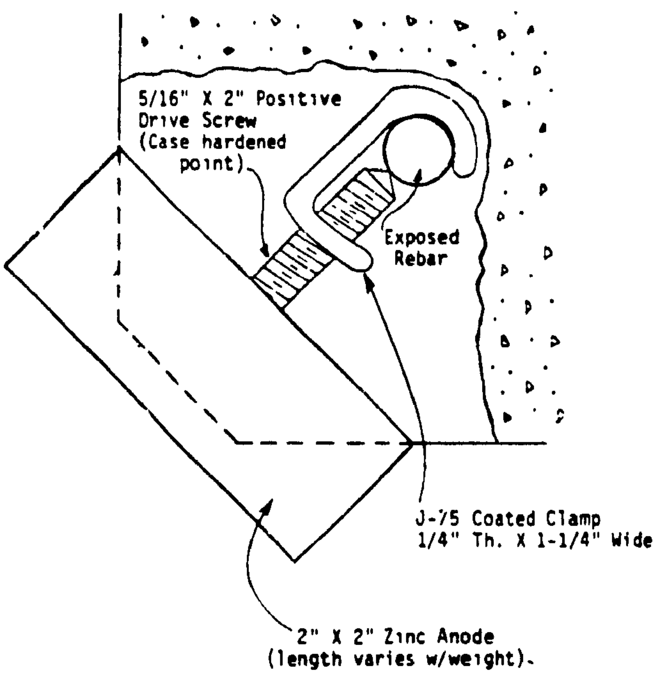
6.2.5.4 CATHODIC PROTECTION OF PILES

Cathodic protection of concrete piles damaged by reinforcement corrosion has been used extensively by the Florida State Department of Transportation. Galvanic zinc anodes have been successfully used to protect reinforced concrete piles in salt or brackish water. The method requires cleaning a large enough area on the exposed reinforcing bar to accommodate the zinc anode assembly (Figure 6.20). One anode is clamped to the exposed rebar for each 2 m (6 ft.) of pile in contact with water (Figure 6.21). The spacing will vary with the weight of the zinc anode chosen. The normal application consists of using 3.2 kg (7 lb.) anodes.[2]

6.2.5.5 SELECTION OF ANODE TYPE

Aspects such as the service environment, installation constraints, and any long-term maintenance concerns are all factors which must be considered in selecting the type of anode to be used.

Cathodic protection by sacrificial anodes using magnesium, zinc, or aluminum alloys have been commonly used. However, a recent study by de Rincón et al.[11] concluded that magnesium and zinc sacrificial anodes are not suitable for embedment in concrete. The study found that magnesium produces a large volume of oxidation products which crack the concrete in a short period of time, and that zinc does not adequately polarize the steel. Aluminum anodes produce a much smaller volume of oxidation products and protect the reinforcing steel more effectively because of their better diffusion properties. Accordingly, the report concluded that, cathodic protection using aluminum anodes, either embedded in concrete or immersed in water, is a feasible method to control the corrosion of chloride contaminated reinforced concrete in the splash zone.



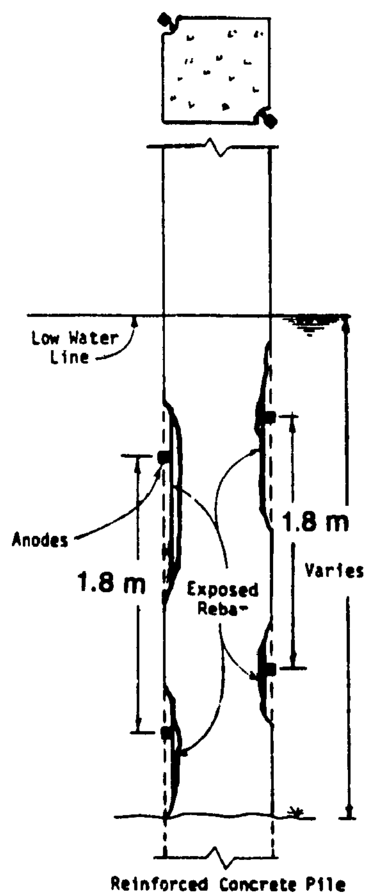


FIGURE 6.20 - ZINC ANODE ASSEMBLY[2]   FIGURE 6.21 - ZINC ANODE SPACING[2]

6.2.5.6 RECENT DEVELOPMENTS

Recent developments with the impressed current system have made cathodic protection more efficient, durable, and cost effective for protecting reinforced concrete structures. In the late 1950s, impressed current systems, which were developed for the protection of reinforced concrete bridge decks, used high silicon cast iron anodes in a conductive asphalt overlay. To reduce the thickness and thus the weight of the overlay, anodes consisting of platinum-clad niobium wire and graphite fibers were placed into grooves and set in a conductive polymer grout.[12]

Anode systems have been developed primarily for use with vertical and soffit surfaces of reinforced concrete structures, such as wharfs and bulkheads. These include conductive surface coatings, conductive (copper) polymer mesh, and titanium expanded mesh embedded in a shotcrete overlay and sprayed zinc. With these systems, the reinforcement and the anode are connected to the negative and positive terminals of a low voltage DC source, respectively. The density of typical currents used can range between 10 and 100 mA/m^2 (1 and 10 mA/ft^2).[12] Further information on these systems is available in References 13 and 14.

A proprietary water-based conductive coating consisting of a blend of specially treated carbon dispersed in an acrylic resin, was used to protect piers of two bridges in Virginia.[15] The coating is applied in two layers on the concrete surface with brushes or rollers, and is electrically connected to a data acquisition system for monitoring the performance of the system. The system permits the flexibility of monitoring the CP system from anywhere using either a phone, modem, or personal computer. A mixed metal oxide mesh and a conductive polymeric wire encapsulated in a cementitious overlay were recently used to protect reinforced concrete wharf structures in Australia[16] and Saudi Arabia,[17] respectively.

6.3 ELECTROCHEMICAL CHLORIDE EXTRACTION

An alternative to cathodic protection of reinforced concrete structures involves the removal of chloride ions from the contaminated concrete, thereby restoring the alkalinity (high pH) level to stop the corrosion process.[18] The method is nondestructive and can be applied with minimal removal of unsound (cracked or spalled) concrete. Upon completion, the surface of the concrete can be treated with a barrier coating to minimize future penetration of chloride ions. A description of this process, which has been used in Europe and adopted in North America, is summarized in the following sections and has been adapted mainly from a review of References 12 and 18.

The process, which was developed in Norway in the early 1980s, and used for the first time in 1989, removes chlorides from the concrete by "electro-migration", using an applied electric field. A direct electric current is created between the reinforcing steel (cathode) and a temporary anode which is mounted on the concrete surface and embedded in an electrolyte (Figure 6.22). The anode could either be a mesh made of steel or titanium. The electrolyte is placed in a paste that can be sprayed onto the surface being treated. An electric field is applied to the external anode which attracts the chloride ions to the electrolyte paste and draws them away from the steel reinforcement. At the same time, alkali ions in the electrolyte paste are drawn into the carbonated concrete by "electro-osmosis". This gradually restores the pH level around the rebar and in the concrete cover.

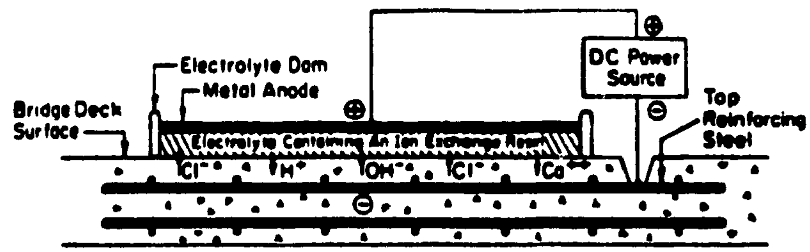


FIGURE 6.22 - ELECTROCHEMICAL CHLORIDE REMOVAL SET-UP[3]

6.3.1 SYSTEM INSTALLATION

To ensure success of the electrochemical process, the system must be properly installed. Prior to installation, the concrete surface must be cleaned thoroughly and any coatings must be removed. To prevent short circuits, all the cracks must be properly sealed with a cement mortar. After the surface is prepared, wooden battens are fixed to the concrete surface, to provide a 12 to 25 mm (½ to 1 in.) gap between the concrete surface and the anode mesh. This allows embedment of the anode between two layers of electrolyte paste. The first layer, which is a cellulose fiber, is sprayed over the surface to the thickness of the battens. The mesh is fastened to the battens and the surface is sprayed again with the electrolyte to an additional 25 mm (1 in.) depth, resulting in a total installation thickness of 50 to 75 mm (2 to 3 in.).

The reinforcing steel within the concrete to be treated and the surface electrode are connected to a low voltage current source by an AC/DC rectifier. The rectifier uses a 100 to 240 volt power source or a generator. Current densities can range between 300 and 1000 mA/m^2 (28 and 93 mA/ft^2).

6.3.2 MONITORING SYSTEM OPERATION

The time it takes for the electrochemical process to reduce the chloride content in the concrete to below the corrosion initiation threshold (0.1% to 0.4% chloride by weight of cement) depends on:

- The amount of chlorides present in the concrete
- Concrete quality
- Concrete cover thickness
- The strength of the induced electric current
- The quality of system installation

The time required to realkalinize the concrete varies between one to two weeks, while chloride removal takes eight to ten weeks. During this period, chemical analyses of the cover concrete should be performed to determine if the desired chloride or pH levels have been reached. The pH levels can be monitored by using a rapid chloride test and a pH indicator. Measuring current density and the voltage that is developed during the process help to determine when the treatment is sufficient. At the end of the treatment the electrolyte paste is removed and a chloride barrier coating is then applied to prevent future penetration of chlorides.

6.3.3 CHLORIDE EXTRACTION (CE) VS. CATHODIC PROTECTION (CP)

The electrochemical removal of chlorides in concrete is a relatively new process that has been used in practice for only a few years, therefore, its effectiveness is not well documented. However, there are some promising advantages which could make it a more cost effective treatment than CP, provided it is applied before deterioration becomes severe enough to require extensive structural repair. The following is a comparison of the two methods as reported by Collins and Farinha:[12]

- CE is not an on-going treatment, whereas CP requires periodic anode maintenance and system monitoring.

- CP requires detailed design to ensure adequate performance while the CE anode is temporary and less attention to detail is needed.
- CE is simpler to apply than CP resulting in lower installation costs.
- CE requires minimal surface preparation. The CP system requires intensive surface preparation and is more time consuming.
- Evaluating CP performance has not been firmly established for reinforced concrete structures while the CE performance criteria is straight forward.
- CE application is subject to wave impact damage.

Chloride extraction technology has been extensively tested by the Ministry of Transportation of Ontario in Canada, and has been under investigation in Australia, and by the Strategic Highway Research Program (SHRP) in the United States. It is reported that no negative side effects have been detected. A more detailed review of the technique is available in References 19 through 22.

6.4 REPAIRING SCOUR DAMAGE

6.4.1 SCOUR RELATED DAMAGE

The erosive action of flowing water in streams around bridge piers and abutments, has been a continuing problem for highway department administrators and engineers.[24] The undermining of bridge piers, abutments, and approaches caused by scour, is a constant threat to the service life of bridge structures. The Federal Highway Administration (FHWA) reports that bridge scour is the leading cause of bridge collapse and closure in the United States.[27] For instance, the collapse of the Route 90 bridge over Schoharie Creek in New York (1987) and the U.S. Route 51 bridge over the Hatachie River in Tennessee (1989) were two of the most notorious examples of bridge-scour disasters.[23]

Scour is a natural phenomenon that is defined as "...the displacement of stream bed material by stream or tidal currents."[24] The problem is generally worsened by the presence of obstructions such as bridge waterways construction, piers, spur dikes, and other similar structures. All streambed material is susceptible to scour but is more serious in areas containing alluvial material. Scour most often occurs during flash floods. The magnitude of scour damage depends on the type of sediment in the streambed, volume and speed of water flow, and the shape and size of the structure.[1]

There are three basic types of scour that may occur at a bridge waterway: general, contraction, and local.[24,25] These are discussed below.

6.4.1.1 GENERAL SCOUR

General scour (or degradation) is a process of erosion which occurs over a long period of time and is caused by changes in the river flow pattern. This usually occurs in alluvial streams where the channel cross-section changes or meanders, resulting in river bed elevation changes. Often, however, the degradation of rivers is the result of man-made flow changes either upstream or downstream of the bridge.

6.4.1.2 CONTRACTION SCOUR

Contraction (or constriction) scour occurs when the river flow is restricted by natural causes or by bridge piers, pilings, abutments or other structures. This reduces the river cross-sectional area, thereby increasing water velocity in the immediate vicinity of the bridge structure.

6.4.1.3 LOCAL SCOUR

Local scour results from localized turbulence around pilings and piers. Vortices that form around the piers remove the streambed material faster than it is replaced, thereby eroding soil and sediment from their bases.

6.4.2 REPAIRING SCOUR-RELATED DAMAGE

Once scour damage is detected, the cause of the scour should be determined and corrective measures must be implemented immediately to avoid any further erosion which could lead to the possible loss of the bridge structure or more importantly, loss of life. Repair of damage caused by river scour may require the replacement of displaced material, or it may require redesign or modification of the structure. Various repair procedures have been developed depending on the nature, type, and severity of the damage. Some of the most commonly used methods are summarized below.

6.4.2.1 CONCRETE JACKETING AND RIPRAP

This repair method is generally used for repairing scour damage under pile supported pier footings, and can either be performed in dry conditions by constructing a cofferdam around the structure and dewatering it, or it can be performed in submerged conditions. It basically consists of placing a new concrete subfooting which is protected by riprap placed around the footing (Figure 6 23).[26]

Riprap placement should be done carefully and evenly around the footing to avoid damaging the concrete and to avoid any unbalanced forces against the pier structure. Also riprap should not be placed above the original streambed elevation so as not to change the flow pattern. In some cases, it may be necessary to perform analyses to ensure that the structure can support the additional riprap loading.

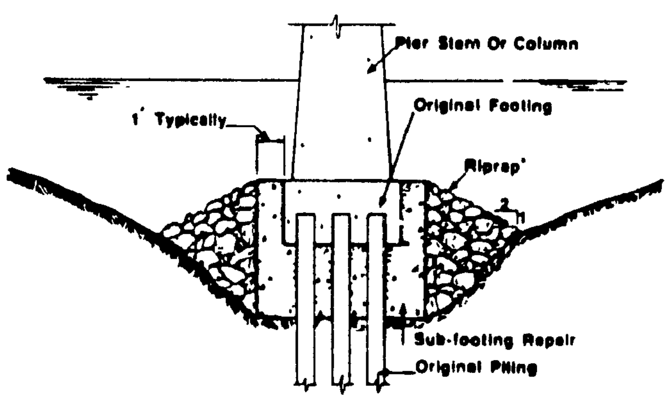
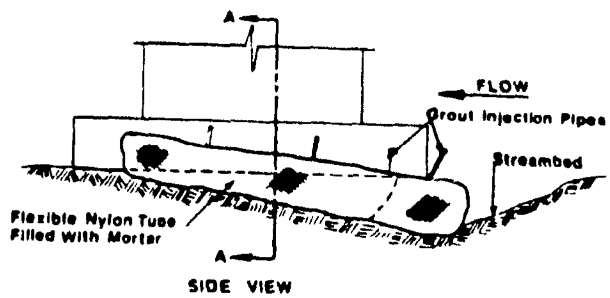


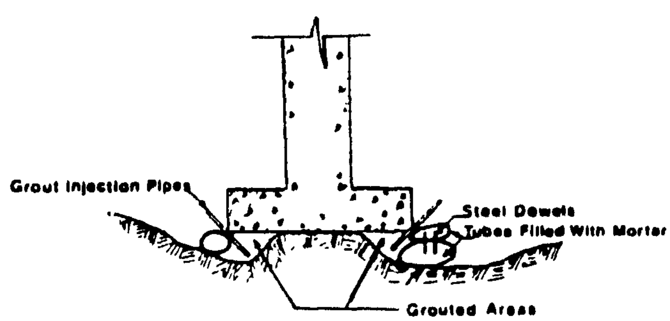
FIGURE 6.23 - CONCRETE JACKETING AND RIPRAP[26]

6.4.2.2 FLEXIBLE TUBE FORMS

This method is used for repairing scour damage beneath spread footings and involves the placement of flexible nylon tubes filled with structural mortar to restore the bearing surface to the footing (Figure 6.24).[26] The forms are fabricated by joining together suitable lengths of fabric with a "high tensile nylon stretching".[3] The tube forms are then placed around the scoured area beneath the footing and are filled with grout. Once the grout hardens, the tubes act as the formwork for the material to be placed beneath the footing. Grout injection tubes are then inserted between the tube form and the footing and grout is pumped into the void space beneath the footing. An adequate number of injection tubes should be provided to allow water to escape during pumping of the grout.



(a) Flexible Tube Forms

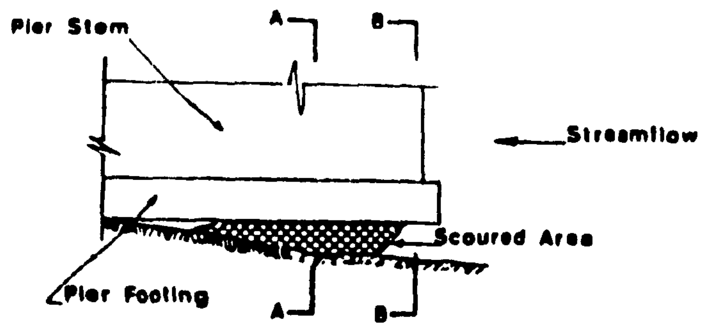


(b) Section AA

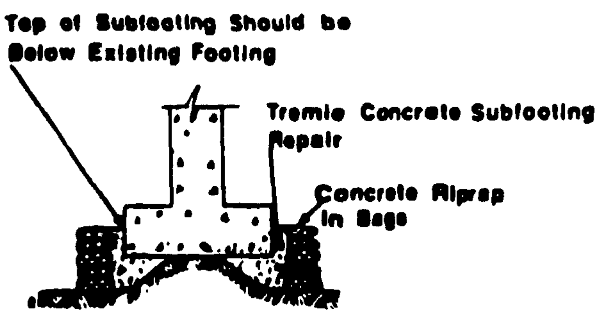
FIGURE 6.24 - FLEXIBLE TUBE FORMS[28]

6.4.2.3 TREMIE CONCRETE SUBFOOTING AND CONCRETE RIPRAP

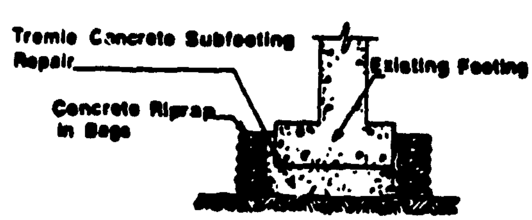
This method is similar to that used for the flexible tube forms and involves filling the scoured area with tremie concrete. Bags are filled with concrete riprap and are stacked around the scoured area along the perimeter of the footing, creating the formwork for the concrete fill (Figure 6.25).[26]



(a) Concrete Riprap and Tremie Concrete Subfooting-Partial Elevation



(b) Section AA -After Repair



(c) Section BB -After Repair

FIGURE 6.25 - CONCRETE RIPRAP AND TREMIE CONCRETE SUBFOOTING[26]

6.4.2.4 REPAIRS FOR HEAVILY UNDERMINED STREAMBEDS

Significant changes in the river flow pattern may sometimes cause severe scour of the entire streambed. This usually occurs during peak runoff periods where a sudden change in the river flow pattern downstream significantly increases the water flow velocity upstream, resulting in severe erosion. The corrective measure typically involves rebuilding the entire streambed to the original elevation with crushed stone subbase material topped with a heavier stone riprap as shown in Figure 6.26.[26]

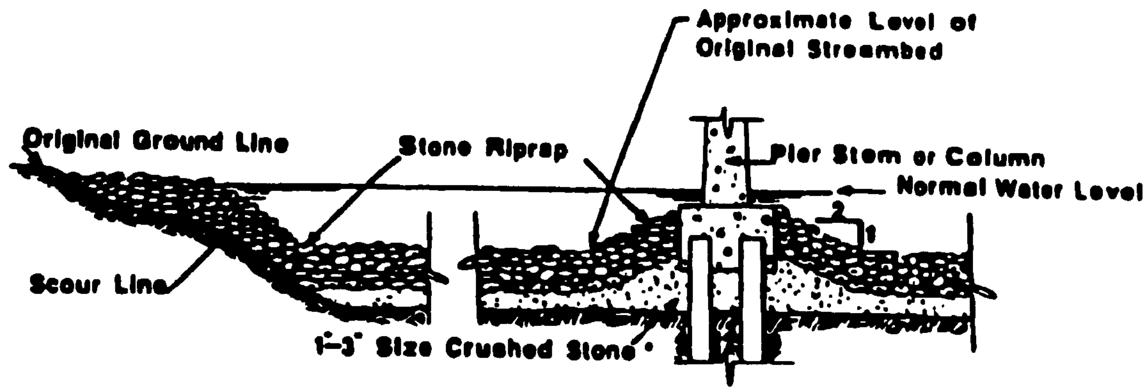


FIGURE 6.26 - RECONSTRUCTION OF SCOURED STREAMBED[26]

6.4.2.5 REPAIR OF ABUTMENT FOUNDATIONS

Abutment foundations are usually found at a higher elevation than pier foundations and are more susceptible to scour damage. For abutments not supported on piles, a new concrete subfooting is constructed below the existing footing (Figure 6.27a). A connection between the two footings is made by installing machine bolts in the existing abutment concrete at approximately 450 mm (18 in.) centers, also shown in Figure 6.27a. Bolting is not required for pile supported abutments (Figure 6.27b). Also, stone riprap is often placed around the new footing to prevent future scour damage.[3]

Alternatively, the scoured area can be filled with a sand backfill and protected by concrete filled fabric bags (Figure 6.28). The stability of the bags may be increased by placing them in interlocking brick bond fashion and driving reinforcing bars through them before the concrete sets. A filter fabric is required under the bags to prevent scour from occurring through the spaces of the individual bags. The most commonly used materials are synthetic fiber, non-woven and woven fabrics.[9]

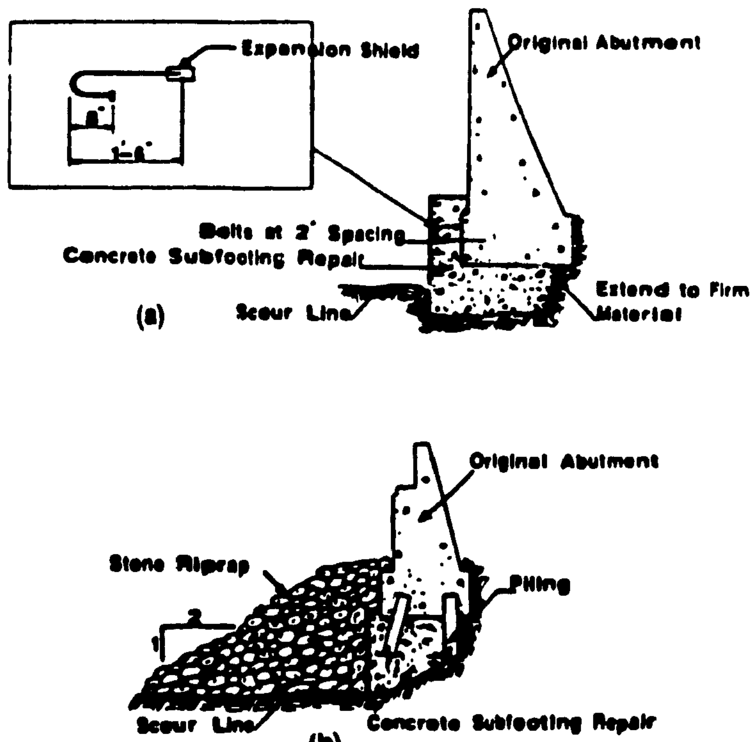


FIGURE 6.27 - ABUTMENT REPAIR: (a) SOIL BEARING TYPE; (b) PILE BEARING TYPE[26]

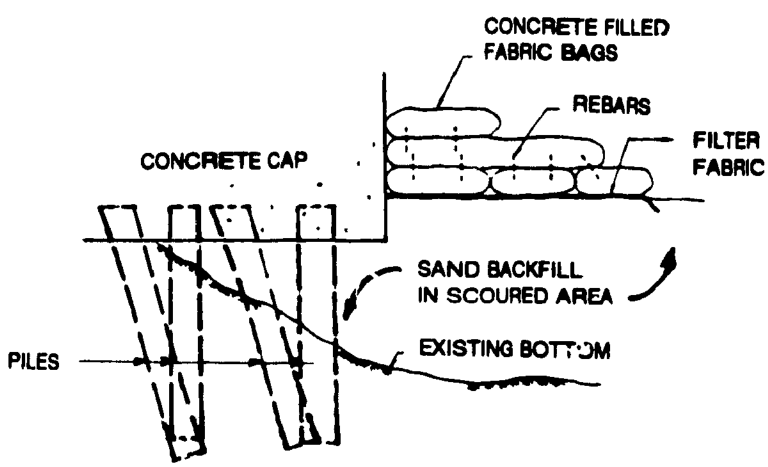


FIGURE 6.28 - FABRIC FORMS AND FILTER FABRIC SCOUR PROTECTION[9]

6.4.2.6 SHEET PILE DRIVING

The driving of steel sheet piling in front of piers and abutments may be used to retain material in place or to prevent further scour. It also allows the placement of any lost foundation material to be replaced behind the sheeting.[9] The toe of the sheet piling should be driven to sound rock or nonerodible soil (Figure 6.29). In some cases, sheeting may be difficult to install due to the overhead clearance required to drive the sheets.[2]

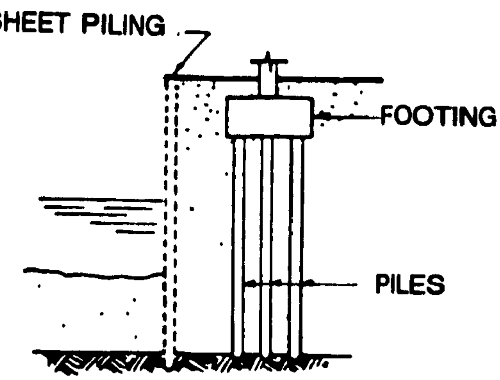


FIGURE 6.29 - SCOUR PROTECTION USING SHEET PILING[2]

6.4.2.7 TRAINING WORKS

Spur dikes, jetties, deflectors, and other structures are often constructed to redirect water flow away from a bridge pier or abutment (Figure 6.30). However, these structures need careful design considerations to avoid causing scour damage to adjacent areas.[2]

6.4.2.8 REPAIRING TILTED PIERS

Severe scour may cause bridge piers to tilt, causing distress in the superstructure which may lead to failure if not repaired. The first step in the repair procedure involves drilling and grouting a series of dowels through the bridge deck and into the pier as a temporary measure to prevent further tilting of the pier until a permanent repair is made (Figure 6.31a).[3]

The pier cross-section is then enlarged to provide a seat for jacking the bridge deck to its original elevation. Once the deck is brought to its original elevation, it is supported on steel shims until the underwater portion of the footing is repaired (Figure 6.31b). During this stage, additional holes are drilled through the deck and the pier so that they can be connected monolithically with the second stage concrete placement. The steel shims are left in place.[3]

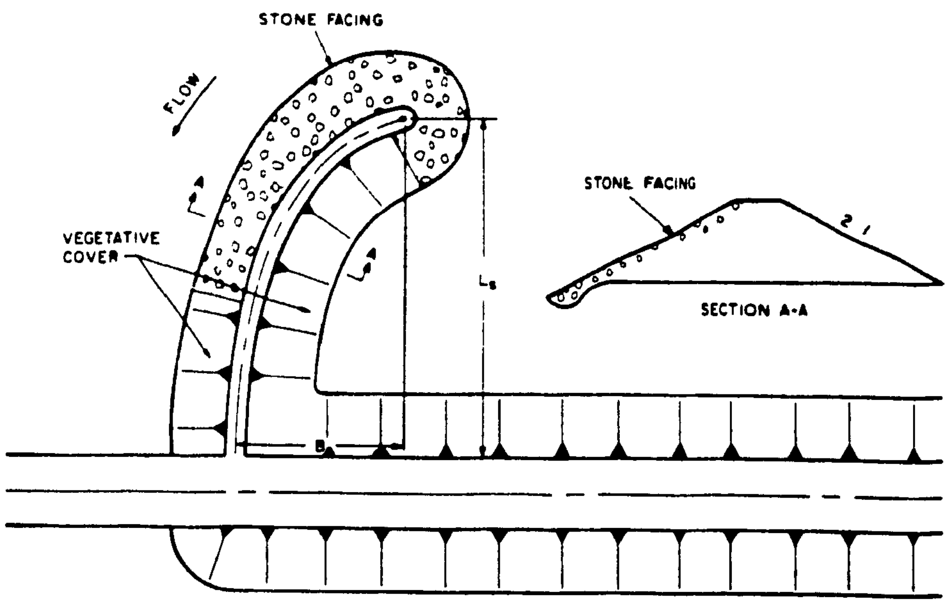


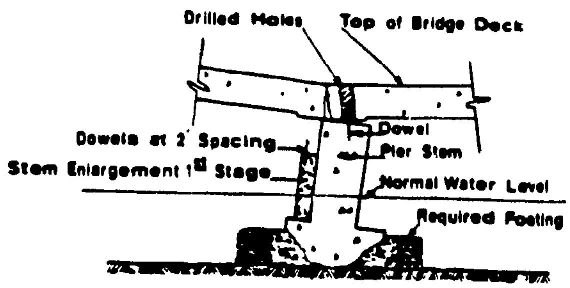
FIGURE 6.30 - SPUR DIKE[2]

6.4.3 SCOUR INSPECTION AND MONITORING

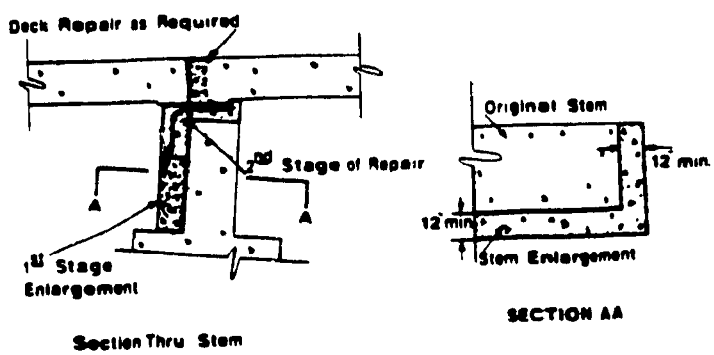
A number of federal and state transportation agencies in the United States are testing new technologies and instrumentation systems that can monitor and measure the extent of scour that occurs at a bridge substructure. These systems can be grouped into two categories: "postflood measurement systems, and real-time systems that monitor the streambed during floods".[27] A comparison of these systems, regarding costs, manpower efficiency, overall effectiveness, and other factors is provided in Table 6.3.

Postflood surface assessment methods used to detect scour holes include subsurface interface radar (SIR), also called ground penetrating radar (GPR), and several continuous seismic profiling

(CSP) systems, such as tuned transducers and both monochrome and color fathometers. Each system can be used during normal river flow conditions to measure scour depth. This method is especially useful for determining the amount of material that fills a scour hole after a flood. This prevents bridge inspectors from underestimating the degree of scour that has actually occurred.



(a) Repairs to Tilted Piers - First Stage Repair -Section through Pier Stem



(b) Repairs to Tilted Piers - Second Stage Repair

FIGURE 6.31 - REPAIRS TO TILTED PIERS: (a) FIRST STAGE REPAIR; (b) SECOND STAGE REPAIR[26]

Real-time systems can be used to monitor scour activity during floods. These come in two types: permanently installed (or fixed) and portable. Permanently installed systems are used to take measurements where future bridge scour is expected to occur. The advantage of this system is that the device takes measurements before, during, and after the erosion, and can provide valuable information on how the scour hole develops over time. A disadvantage is that it provides data only at the location where it is installed. Also, fixed systems are susceptible to impact

TABLE 6.3 - COMPARISON OF SCOUR MONITORING SYSTEMS[27]

	Cost ($ US)	Manpower efficiency	Overall effectiveness	Other factors
SIR and CSP systems	Complete system with operators $1,000 - $1,500 per day.	Require two or more people to conduct surveys.	Combination of SIR and CSP will provide accurate subbottom and stratigraphic analysis of almost all sites.	Price and training requirements may make systems cost-effective for users with frequent need.
Real-time testers (fixed)	Several hundred to several thousand dollars per site.	Once installed, does not need anyone to be present during the scan; to acquire data, someone must return to the site.	Testers provide accurate real-time picture of erosion before, during and after flood, but only of areas where sensors are installed. They provide no subsurface data, thus no data on infill.	Permanent installation means the pieces of hardware needed is equal to the number of bridges.
Real-time testers (portable)	$2,000 - $5,000	Requires operators to visit bridge or site during floods.	When crew can reach the site and navigate waters, testers will provide accurate picture of erosion in progress. They provide no subsurface data.	Access to sites during flooding could be hazardous.
Physical probes/visual inspection	Minimal hardware costs	Requires a skilled crew on-site five to ten times longer than time needed for SIR or CSP systems to conduct inspection.	Probes provide excellent surficial and subsurface data at points of installation or inspection.	Skilled labor may now be more expensive than hightech equipment. Method was more effective before advances in SIR and CSP technology.

damage by floating debris during a flood. Although these nondestructive testing tools are generally expensive, the scope and speed with which the data is obtained often justifies the cost. A brief description of each system follows.

6.4.3.1 SUBSURFACE INTERFACE RADAR (SIR)

SIR systems are best suited for use in fresh water less than 6 m (20 ft.) deep. They transmit electromagnetic waves into the riverbed to provide high-resolution continuous subsurface profiles. The pulses are reflected at subsurface interfaces and are recorded by the SIR system, enabling the inspector to map subsurface conditions. The data is displayed as a continuous profile on a graphic recorder or color monitor. SIR systems can provide data up to depths of 30 m (100 ft.) in low conductive subsurface materials. Highly conductive materials tend to limit signal penetration to only a few meters. SIR systems generally perform better in freshwater streams with granular bed materials. They do not perform effectively in dense, moist clays and they do not function at all in salt water.

6.4.3.2 CONTINUOUS SEISMIC PROFILING (CSP)

CSP systems can function in salt or brackish waters and can penetrate deeper than SIR systems. They transmit acoustic waves through the water and riverbed materials. When the transmitted wave hits a subsurface interface, part of the wave is reflected back to the system.

The monochrome fathometer clearly defines the limits of scour holes, but will not penetrate the infill material. Color fathometers, on the other hand, can clearly map subsurface profiles too a greater depth than SIR systems. For this reason, CSP systems are often used in conjunction with SIR systems. Tuned transducers can penetrate streambed material to depths from several centimeters to a few meters in coarse material and several meters in fine materials.

6.4.3.3 FIXED REAL-TIME SYSTEMS

With these systems, falling rods which follow the scour depth to measure the amount of fall, are fixed to the piers to determine the depth of the scour hole. The bottom end of the rod must be large enough so that it does not settle into the streambed, and the top end must extend above the footing. For the rod to provide accurate measurements, it must be installed vertically.

6.4.3.4 PORTABLE REAL-TIME SYSTEMS

There are numerous types of portable real-time units available and they all operate in three phases: deployment, sounding, and horizontal positioning. If a crew of workmen can install this system during a flood, it can provide a great deal of information. These systems "offer the real-time advantages of fixed systems with the postflood system's flexibility of movement".[27]

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CHAPTER 7

IN-SITU REPAIR OF CONCRETE HYDRAULIC STRUCTURES

7.1 INTRODUCTION

Concrete hydraulic structures such as dams, spillways, and lock chambers make up a significant portion of North America's infrastructure. For instance, the U.S. Army Corps of Engineers (USACE) operates and maintains 536 dams and 260 lock chambers at 596 sites. Of these, more than 40 percent are over 30 years old and 29 percent were constructed before 1940.[1] In addition, nearly one-half of the 260 lock chambers will reach the end of their 50 year design life by the year 2000[2] and many of these structures will be kept in service well beyond their design lives.[3] Periodic inspections of these structures reveal that many of the older structures require significant maintenance, repair, and rehabilitation. The newer structures must also be maintained to ensure their continued service and operation.

According to the results of a survey conducted between 1982 and 1985 by the USACE,[61] the three most common types of deterioration found in concrete hydraulic structures are: cracking, spalling, and seepage. These three categories accounted for 77 percent of the 10,096 deficiencies identified during a review of available inspection reports for the USACE's civil works structures. Cracking was observed the most and accounted for 38 percent of the total. Since many of the structures were constructed using non-air-entrained concrete, much of the cracking and spalling is attributed to deterioration resulting from freezing and thawing effects. The initial cracking, however, may have been initiated by any of several different causes including drying shrinkage, thermal stresses, alkali-aggregate reaction, corrosion of embedded metals, and differential structure movement.[3] Since there are several phenomena which can cause cracking, no one repair technique will be appropriate in all cases. To develop the proper repair solution, the cause and extent of cracking must be clearly identified.

A report by ACI Committee 224[4] recognizes twelve techniques most commonly used for the repair of cracks in concrete structures. A summary of these techniques and materials is provided in Tables 7.1 and 7.2, respectively. A number of methods and materials have also been used to repair surface spalling and scaling[5] and are listed in Tables 7.3 and 7.4. From an evaluation of these repair techniques and materials, three crack repair procedures and two techniques for repairing spalled concrete were identified as being the most appropriate for in-situ repair of

TABLE 7.1 - CRACK REPAIR TECHNIQUES FOR CONCRETE[3]

Repair Technique	Type of Crack: Dormant	Type of Crack: Active	Comments
Pressure Injection	X		Little surface preparation is needed, scar marks may be left on surface where crack was injected Limited to areas where concrete has not yet spalled Structural quality bond is established but if large structural movements are still occurring, new cracks may open Process can be used against a hydraulic head
Routing and Sealing	X		Simplest method available for repair of cracks with no structural significance Process not applicable to repair of cracks subjected to hydraulic head
Stitching	X	X	Process will not close or seal cracks but can be used to prevent them from progressing Generally used when it is necessary to reestablish tensile strength across crack
Addition of Reinforcement	X	X	Primarily used to restore or upgrade structural properties of cracked members.
Drilling and Grouting	X		Technique applicable only when cracks run in straight line and are accessible at one end
Flexible Sealing	X	X	Technique is applicable where appearance is not important and in areas where cracks are not subjected to traffic or mechanical abuse
Grouting	X		Wide cracks may be filled with portland-cement grout Narrow cracks may be filled with chemical grouts
Drypack Mortar	X		For use in cavities that are deeper than they are wide Convenient for repair of vertical members
Crack Arrest	X	X	Commonly used to prevent propagation of cracks into new concrete during construction
Impregnation	X	X	Technique can be used to restore structural integrity of highly deteriorated or low quality concrete Can be used to seal small crack networks
Overlays and Surface Treatments	X	X	Slabs containing fine dormant cracks can be repaired using bonded overlays Unbonded overlays should be used to cover active cracks
Autogenous Healing	X		A natural process of crack repair has practical applications for closing dormant cracks in moist environments

concrete hydraulic structures.[3] The selected methods for crack repair include pressure injection, polymer impregnation, and addition of reinforcement. Thin reinforced overlays and shotcrete can also be used to repair spalled concrete and to resurface structures after crack repair. This chapter provides a discussion of each of the above and other techniques used for repairing concrete hydraulic structures and has been adapted from a review of different available references, especially 3, 21, 28 and 31.

TABLE 7.2 - MATERIAL SELECTION FOR CONCRETE REPAIR[7]

	Large Spalls, cover (mm)			Small spalls, cover (mm)		Crack sealing	Structural crack repair	Bonding aids	Honeycombed concrete	Permeable concrete
	25	12-25	6-12	12 25	6 12					
Concrete Sprayed concrete Sand/cement mortars	X									
Polymer modified cementitious mortars		X	X[1]	X						
Epoxy resin mortars			X		X					
Polyester resin mortars					X					
Moisture tolerant epoxy resins								X		
SBR, acrylic and co polymer latices						X		X		Depends on permeability
Low viscosity polyester and acrylic resins						X			X	Depends on permeability
Epoxy resin low viscosity							X		X	Depends on permeability
Penetrating polymer systems, in surface sealers										X
Special coatings and penetrating in surface sealers										X
Universal bonding aids, PVA PVA modified mortars	not suitable for external repairs									

[1]Depending upon service conditions the application of an anti-carbonation protective coating may be required

TABLE 7.3 - TECHNIQUES FOR REPAIRING SPALLED CONCRETE[3]

Repair Technique	Comments
Coatings	This technique is generally used when the scaling or spalling is limited to a very thin region at the surface of the concrete
Concrete Replacement	This technique is one of the most commonly used and is appropriate for applications where the cause of deterioration is nonrepeating or has been eliminated
Grinding	This technique can be used when the deterioration is limited to a thin region at the surface of the concrete
Jacketing	This technique entails fastening a material to the existing concrete that is more resistant to the environment that is causing the deterioration
Shotcreting	This technique is practical for large jobs, on either vertical or horizontal surfaces, where the cavities are relatively shallow
Prepacked Concrete	This technique is suitable for inaccessible applications, such as submerged concrete or deteriorated concrete that is being jacketed
Thin-Bonded and Un bonded Overlays	Thin overlays are often used to repair surfaces that are basically sound structurally but have deteriorated because of cycles of freezing and thawing, heavy traffic, or other exposures which the original concrete was unable to withstand

TABLE 7.4 - MATERIALS FOR REPAIRING SPALLED CONCRETE[3]

Repair Material	Comments
Bituminous Coatings	Asphalt- or coal-tar-based bituminous coatings are used to waterproof concrete or protect it, to some extent, from weathering
Concrete, Mortar, or Grout	Portland cement concrete, mortar, and grout have a number of advantages as a repair material, including thermal properties similar to the existing concrete, similarity in appearance, comparatively low cost, availability, and familiarity
Epoxies	Epoxies are most often employed in repair work for the following uses as an adhesive to bond plastic concrete to hardened concrete or other rigid materials, for patching, and for coating concrete to protect it from aggressive environments
Expanding Mortars, Grouts, and Concretes	These materials are generally proprietary materials to counteract the problem of shrinkage by incorporating ingredients which produce an expansive force approximately equal in magnitude to the shrinkage stresses
Linseed Oil	Linseed oil is generally used to prevent or minimize additional scaling from occurring
Latex-Modified Concrete	Latex-modified concretes have generally been used for resurfacing deteriorated floors and bridge decks They typically develop higher strengths, bond better to existing concrete, have higher resistances to chloride penetration, and are more resistant to chemical attack than plain concrete
Polymer Concrete	Polymer concrete has been used extensively to repair highway bridges and pavements It has a number of advantages over normal concrete, including rapid curing characteristics, high early strength, good bond strength, and excellent durability through cycles of freezing and thawing

7.2 STRUCTURAL REPAIR OF CRACKS

7.2.1 EPOXY INJECTION

Pressure injection of cracks with a low-viscosity epoxy resin can restore the original tensile/shear strength of the uncracked concrete, providing the crack interface is clean and sound.[6] This repair technique has been successfully used for about 30 years to repair cracks in bridges, buildings, dams, lock chambers, and many other types of concrete structures. However, unless the crack is not moving or the cause of cracking can be eliminated, the concrete will probably crack again elsewhere in the structure. If it is not possible to establish and eliminate the cause of the original cracking, it is recommended to use a sealant or other material which allows the crack to function as a joint.[3,4] Alternatively, a movement joint can be cut out adjacent to the crack and the crack injected with epoxy resin, or the crack itself can be made into a movement joint[7]. ACI Committee 504[8] provides a good guide to joint sealants in concrete structures.

Epoxy injection has been used to repair cracks that vary in width from 0.05 mm (0.002 in.) to 6 mm (¼ in.). It consists of drilling holes at close intervals along the length of the crack, installing injection ports, sealing the surface of the crack between the ports, and injecting the epoxy resin under pressure into the crack as previously described in Chapter 5. Injection progresses from port to port (usually beginning at the lowest point of vertical or inclined cracks and at one of the ends of a horizontal crack) and continues until the entire length of the crack is filled. Prior to injection, the crack should be flushed with a high pressure water jet to remove any loose concrete, dirt, grease, or contaminants which can reduce the bond strength and effectiveness of the repair.

Unlike other techniques, the main advantage with using pressure injection is that it seals cracks externally and internally. Sealing a crack completely will prevent moisture penetration, thereby reducing the potential for freeze-thaw damage.[3] It has been reported that epoxy resin injection techniques can completely fill cracks finer than 50 microns. However, in these cases, as the resin penetrates the crack, significant back pressures can develop and should be carefully controlled to avoid blowing the surface seals. For injecting very fine cracks, state-of-the-art metered dispensing machines are available which can mix and deliver small amounts of resin/hardener at a time.[7] In some cases, concrete surfaces with a large area of very fine cracks can be filled using a combination of a vacuum (to remove the air in the cracks) and pressure injection (Section 7.2.4).[9]

Although epoxy injection is considered to be one of the most viable techniques for repairing in-situ concrete, its performance is affected by ambient temperature and the level of skill of the applicator[4] Since the width of the crack changes with temperature, repairs should be done during the cooler months, when cracks are at their widest. This technique has the advantage that during the summer months when the cracks become narrower, the sealant in the cracks will always be in compression. While moist cracks can be injected with epoxy, this technique cannot be used to repair cracks that are actively leaking and cannot be dried out.[4] In these cases, specially formulated chemical grouts (Section 7.3) will usually provide an effective solution. However, these grouts lack sufficient bond strength and cannot be used for repairing structural cracks.

7.2.2 POLYMER IMPREGNATION

Laboratory tests[10] and field applications[11,12,13] have shown that polymer impregnation is an excellent in-situ method for repairing highly cracked or deteriorated concrete. In general, the procedure consists of filling cracks in concrete with a monomer liquid (such as methyl methacrylate) and then polymerizing (solidifying) the liquid in place. Test results show that polymer impregnation significantly improves the overall physical and mechanical properties of highly deteriorated, low-quality, or non-air-entrained concrete. Compressive strengths can be increased by as much as five times their original value.[11] Permeability to water and chloride ion penetration is reduced, while resistance to freeze-thaw cycles, abrasion, and chemical attack is greatly improved.[3]

There are several systems that can be used for impregnating concrete. A monomer system is a liquid that consists of small organic molecules that, when polymerized, combine to form a clear solid plastic. Monomer systems used for impregnation of concrete contain a catalyst and the basic monomer or combination of monomers such as acrylates or styrenes. Monomers are not compatible with water, and therefore, it is essential to thoroughly dry the concrete to the desired depth of monomer penetration. If a volatile monomer is used and it evaporates before polymerization, it will also be ineffective.[4] An effective polymer impregnation process, as developed by the U.S. Bureau of Reclamation Engineering and Research Center, consists of four basic steps:[3]

(a) Sandblasting the concrete surface to remove contaminants or films that would prevent or reduce monomer penetration.

(b) Drying the concrete with heat (at a high temperature) to the desired depth of monomer penetration.

(c) Soaking and impregnating the concrete with liquid monomer (methyl methacrylate) to the desired depth, and

(d) polymerizing the monomer within the pores of the concrete.

Research on the basic properties of polymer-impregnated concrete (PIC) has been in progress for about 23 years, and practical in-situ applications on existing structures have been performed over the last 20 years. Structures which have been repaired using PIC include highway bridge decks, structural floor slabs, roadways,[13] dam outlet tunnel walls,[11] and stilling basins.[12] Most of these applications have been experimental in nature and due to several limitations, the method is not commonly used. These limitations, which make polymer impregnation a relatively expensive method of repair, include the following:[3]

- Specialized equipment and materials are required.
- A relatively high level of expertise and supervision by trained personnel is required for a successful repair.
- The monomer systems currently being used are flammable and toxic requiring specialized safety procedures.
- Monomers are not water compatible, and the concrete surface must be thoroughly dried for a successful repair.

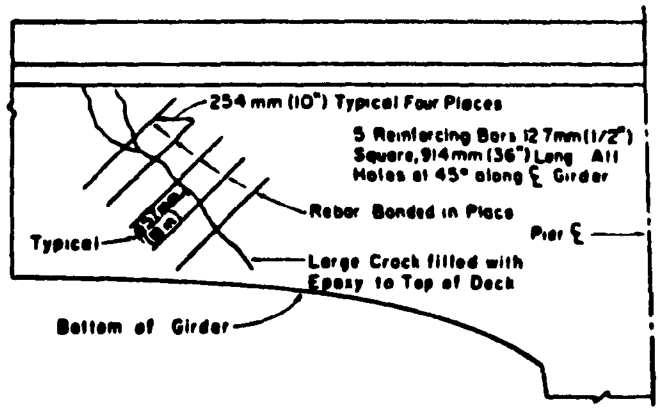
Polymer impregnation techniques are more suitable for repairing horizontal concrete surfaces. However, vacuum impregnation techniques, which are similar to polymer impregnation procedures, are used for impregnating overhead and vertical surfaces. In these method, the liquid monomer is drawn into the pores under a negative pressure created by a vacuum.[7] Vacuum impregnation can also be used on horizontal surfaces to increase the depth of monomer penetration.

7.2.3 ADDITION OF REINFORCEMENT

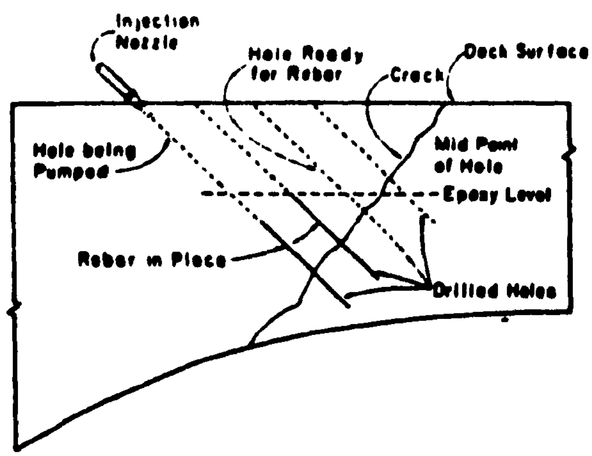
Cracked concrete structures have been successfully repaired and upgraded by adding either internal (post-reinforcement) or external reinforcement.[3,4] The following sections provide a summary of variations of this technique which may be used to repair concrete hydraulic structures.

7.2.3.1 POST-REINFORCEMENT

Post-reinforcement was developed by the Kansas Department of Transportation to repair cracked bridge deck beams and girders. The method basically involves drilling and grouting reinforcing rods through the concrete surface to bridge the crack. The holes are drilled at an angle to the concrete surface so they cross the crack plane at about 90° (Figure 7.1). The epoxy bonds the bar to the walls of the hole and fills the crack plane, thereby restoring the structural integrity of the concrete. The reinforcing bars can be spaced and placed in any desired pattern to meet the specific needs of the repair.



(a) Epoxy Injection - Level of Epoxy when Rebar Should be Inserted in the Hole (60)



(b) Epoxy Injection - Typical Sketch and Description

FIGURE 7.1 - REPAIR OF CRACK BY POST REINFORCEMENT[3]

7.2.3.2 PRESTRESSING STEEL

Modifications to this technique have made it possible to repair cracked concrete hydraulic structures, such as lock walls and dams, using prestressing steel. Post-tensioning is often used to strengthen or stabilize a major portion of the structure, either to close cracks or prevent them from becoming wider. This technique uses prestressing strands or bars to induce compressive forces to close cracks within the structure. The strands are inserted in large-diameter drilled holes, grouted under pressure and tensioned. Adequate anchorage must be ensured for the prestressing strands. The effects of the compressive (or tensile) forces that are created within the structure must be carefully analyzed to ensure that the problem is not transferred to some other part of the structure.[4] Examples of this procedure are the repair of the John Day Navigation Lock structure in Oregon, Washington[14,62] (Figure 7.2), and the repair of Big Eddy Dam in Ontario, Canada.[15]

7.2.3.3 EXTERNAL REINFORCEMENT

External reinforcement, such as steel rods and reinforcing tendons, has also been used to strengthen an under-reinforced or highly cracked concrete structure. As with prestressing steel, this method can also be used to close cracks. However, the major disadvantage with this technique is that strengthening and stiffening the structure where the crack is being repaired may cause cracking in other parts of the structure.[3] Also, the external reinforcement will be subjected to corrosion and may need to be enclosed in an impermeable overlay.

7.2.3.4 STITCHING

This method has not been specifically used for repairing concrete hydraulic structures, but can easily be adapted as a temporary repair solution. This method generally involves drilling and grouting in metal U-shaped rods (stitching dogs) with short legs that span across the crack (Figure 7.3).[4] Stitching may be used to restore the tensile strength across the crack or to prevent the crack from propagating further. As with the previously described methods, stitching will often stiffen the structure and cause cracking elsewhere in the structure. This may require strengthening the adjacent sections also.

The stitching procedure requires holes to be drilled on both sides of the crack, pressure-washing the holes with a water jet, and anchoring the legs of the stitches in the holes with a non-shrink grout or epoxy resin. The stitching dogs "should be located to distribute the tension across the crack over a large area".[4] Spacing of the stitching dogs should be tightened at the ends of the crack. Since the stitching dogs will be exposed to a corrosive environment, they must be embedded in a suitable overlay. In the case of active cracks, stabilizing the structure prior to stitching may provide a more effective solution.

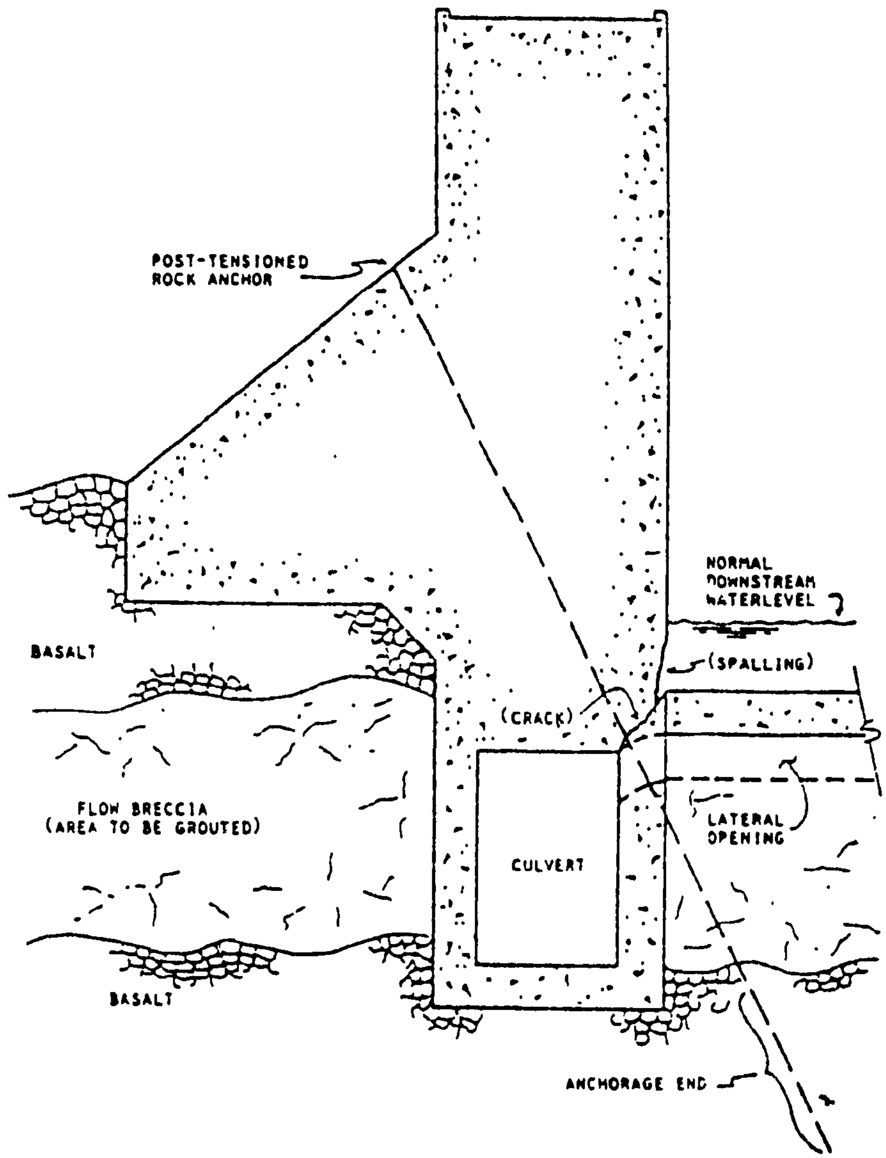


FIGURE 7.2 - REPAIR OF CRACK BY POST-TENSIONED ROCK ANCHOR AT JOHN DAY NAVIGATION LOCK AND DAM, WASHINGTON[62]

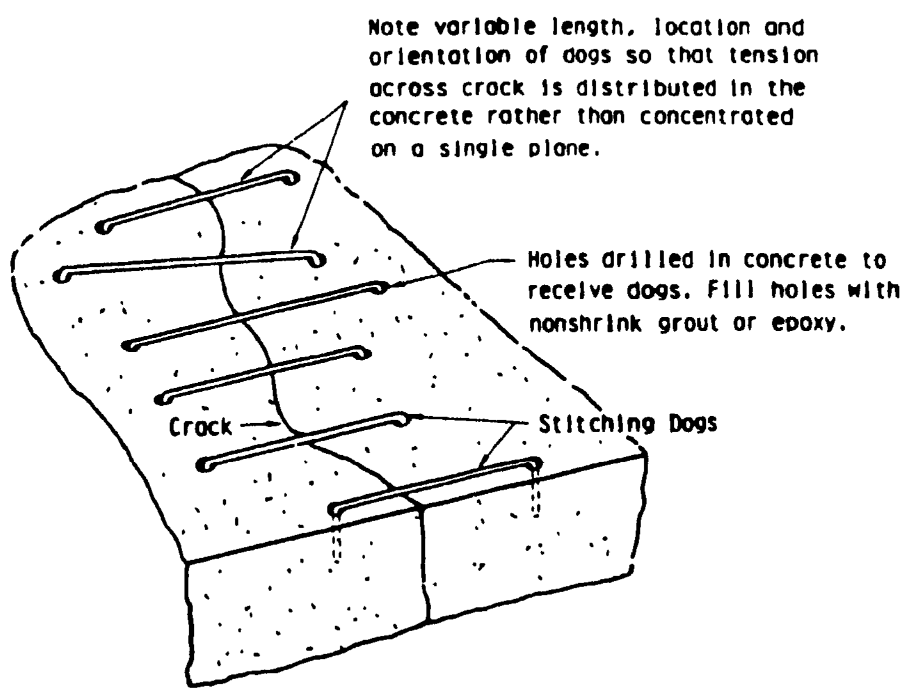


FIGURE 7.3 - REPAIR OF CRACK BY STITCHING[4]

7.2.4 VACUUM IMPREGNATION

Vacuum impregnation is a patented process also known as the "BALVAC" process. The method uses a combination of a vacuum and low pressure to inject cracks or to fill pores with resin. It is especially suitable for repairing large, highly cracked concrete surfaces, which could not be economically or practically repaired by injecting each crack individually. It is also used to reduce the permeability of low quality concrete or masonry and as a preliminary treatment prior to patching spalled concrete.[7] The method has existed for a long time as a factory process for treating and impregnating timber piles or electrical components. Field application of the process was initially performed by engineers to repair deteriorated masonry bridges in India. The process is particularly useful for repairing highly cracked concrete water tanks, but the same principles may be applied to repair a variety of concrete structures.[9] The following is a summary of the basic steps and materials used in the technique.

A netting is spread over the cracked area and is covered with a clear "polythene" sheet. Small ducts are placed along the edges beneath the covering to allow air to escape when the vacuum pump is connected to the ducts and the perimeter of the polythene sheet is sealed with a mastic compound, as shown in Figure 7.4. When the vacuum is applied, the polythene sheet is drawn down tightly against the concrete surface. After the desired level of vacuum is established, the resin is allowed to flow onto the concrete surface through airtight connections in the polythene sheet. The netting beneath the polythene sheet allows the resin to flow across the surface of the repair area. The vacuum is then reduced to allow atmospheric pressure to force the resin into the surface cracks or pores. If the surface being repaired is vertical, the vacuum level can be balanced against atmospheric pressure to keep the resin in position until it begins to set so that it will not seep out of the cracks. Once impregnation is complete, the cover and netting are removed before the resin hardens.[9] It should be noted that if the cracks penetrate the full depth of the member, it must be sealed on the opposite side to prevent ingress of air while the vacuum is being applied.[7]

Vacuum impregnation can also be used in combination with conventional resin injection methods to seal cracks that do not penetrate the full depth of the member. This will reduce the risk of air pockets being trapped behind the resin. Evacuating the air in the crack before injecting the resin will also increase the depth of resin penetration, as shown in Figure 7.5.[9] It should be noted that, very little is gained from using higher pressures, which consequently, could cause damage to some structures. The materials which appear to have the most suitable properties for use with this technique were reported to be low viscosity methylmethacrylate (MMA) acrylic resins. One of the disadvantages of this resin is its high vapor pressure, which if not adequately ventilated, can build up to dangerous levels. The vapor is not toxic but it can cause narcosis and is highly flammable.

7.3 SEALING WATER-BEARING CRACKS

7.3.1 CEMENT-BASED GROUTING

Cement-based grouting techniques, if properly designed and executed, can be a reliable method for repairing water-bearing cracks in concrete structures. Cement grouting has also been

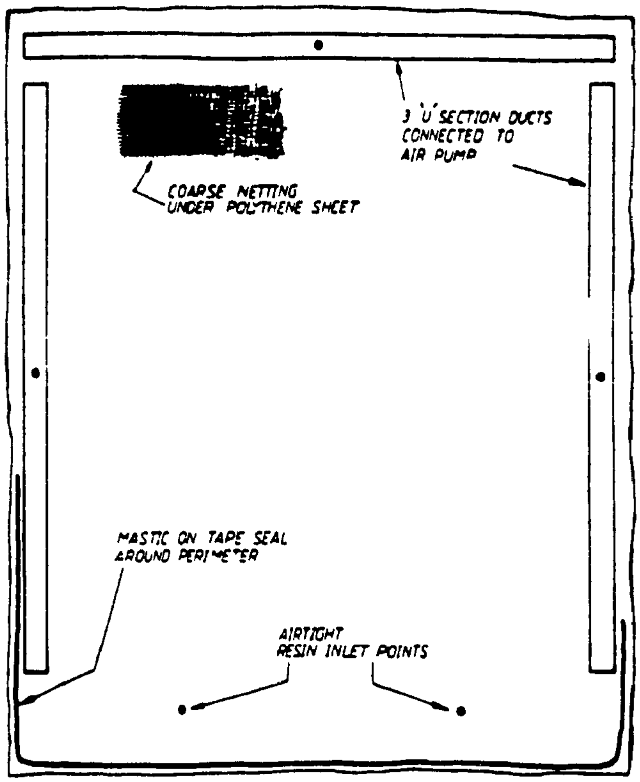


FIGURE 7.4 - VARIOUS COMPONENTS OF THE VACUUM IMPREGNATION TECHNIQUE (BALVAC)[9]

extensively used to construct grout curtains in dams for eliminating or reducing seepage problems.[16] Examination of some case histories proves that cement grouting can also be used successfully for sealing joints and wide cracks in dams,[15,17] and thick concrete quay walls.[18] Cement grouting involves implementing procedures similar to those used for epoxy injection, but can vary significantly depending upon the specific application. For simple applications, the

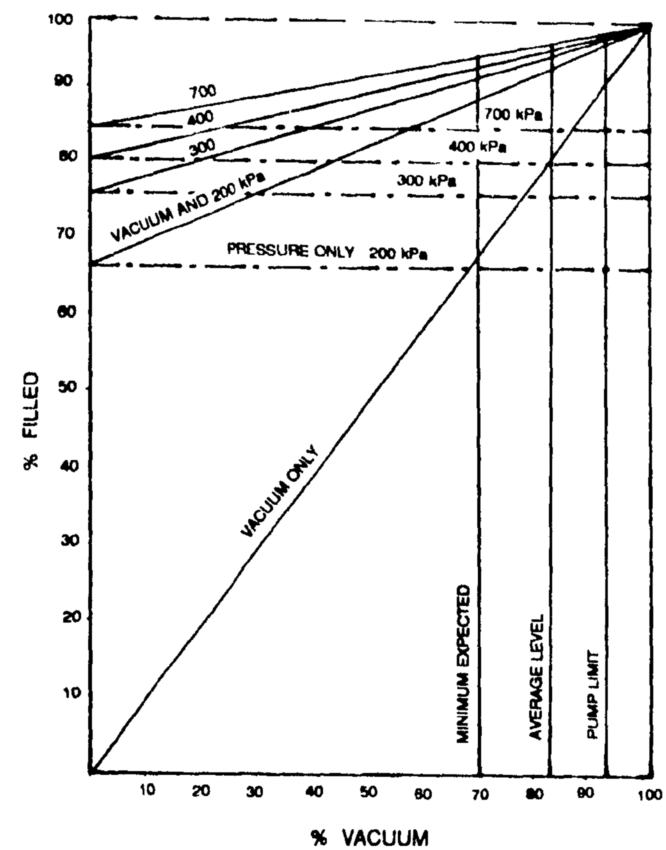


FIGURE 7.5 - EFFECT OF VACUUM ON RESIN INJECTION OF CRACKS[9]

procedure generally consists of cleaning the concrete along the crack or joint, installing grout nipples at regular intervals along each side of the crack, sealing the crack between the injection ports, flushing the crack with water to clean and test the seal, and then injecting the grout.[4]

An alternative and more extensive procedure used for solving seepage problems in dams consists of drilling a series of vertical holes through the crest along the entire length of the dam to form an internal grout curtain as was done at Aswan Dam in Egypt.[59] Holes, typically 60 mm (2-½ in.) to 115 mm (4-½ in.) in diameter are drilled by rotary or rotary percussive rigs. Grouting is then performed through mechanical packers placed either at the top of the hole or at predetermined depths as dictated by the flow patterns. Both single and double packers have been used, although "stage grouting and split spacing" is also commonly used.[16]

7.3.1.1 CONVENTIONAL GROUT

Conventional cement grouts containing mixtures of cement and water, fly ash or sand, and sometimes bentonite, have been used for simple leakage problems.[16] The specific mixture used for any repair depends primarily on the width of the crack and the amount of grout take. In any mixture, the water-cement ratio should be kept as low as possible to maximize strength and minimize shrinkage.[4]

An example of conventional grouting was used to repair cracks in Big Eddy Dam in Ontario, Canada.[15] All 60 mm (2-½ in.) diameter holes were pressure grouted through a single mechanical packer at various levels within the hole. Grouting was generally started using a thin water-cement mix (usually 4:1) and was progressively thickened (with cement up to a 1:1 mix) if the particular section of the hole showed no reduction of grout take. Grouting continued until refusal (less than 4 liters (1 gal.) of grout take in 10 minutes) was achieved.

However, these types of grouts have long setting times and are not effective in flowing water conditions or for penetrating into fine cracks. In addition, these grouts have a tendency to bleed at higher water-cement ratios which makes it difficult to fill larger cracks. Also, once hardened, they become brittle and are unsuitable for repairing moving cracks.[16]

7.3.1.2 QUICK-SETTING GROUT

To reduce setting times, accelerators, such as sodium silicate or calcium chloride, are often added to cementitious grouts to induce "flash setting".[16] This was initially used to seal drillholes that intercepted flowing or artesian water, and has been successfully used for structural repair of dormant cracks. A recent example is the sealing of Morris Sheppard Dam in Texas.[20] Fissures which developed in the foundation transition beam permitted very high water flows. Grouts with accelerators were used to seal the fissures, significantly reducing water flows.

7.3.2 CHEMICAL (POLYURETHANE) GROUTING

Chemical grouts are more suitable than epoxy or cement-based materials when sealing water-bearing cracks. Polyurethane grout, for instance, has been used to solve a wide range of water leakage problems in dams and many other concrete structures. They are "designed to react with water and expand in-situ, forming a tight, impermeable, elastomeric seal that immediately stops the flow of water."[21] The hardened sealant is very flexible and allows the crack to widen while maintaining a tight impermeable seal. Cracks in concrete as narrow as 0.05 mm (0.002 in.) have been filled with chemical grouts.[4]

Several polyurethane types are available, and are classified by their reaction with water and elongation characteristics. Most polyurethane grouts used for controlling leaks consist of one component and are water activated. They are prepolymers that can form either hydrophobic or hydrophillic gels, which can be rigid or flexible.[22] Two component polyurethane elastomers (hydrophobic or hydrophillic) have also been used.[16]

A recent example of the use of a two-component polyurethane grout was at Easton Dam in Connecticut.[60] The foam grout was pressure-injected in 150 mm (6 in.) diameter holes that were drilled vertically down through each monolith joint. Similarly, one component foam grout was used in sealing the construction joints at Norway and Oakdale Dams in Indiana.[24] The joints were of a "labyrinth type" consisting of numerous keyways oriented both horizontally and vertically. Most recently, flexible hydrophillic polyurethane resin was effectively used to seal the lift joints at Soda Dam, Idaho[25] and Upper Stillwater Dam, Oregon.[26] A hydrophobic polyurethane foam grout was also successfully used to restore the monolith joint waterstops at Chief Joseph Dam in Washington, D.C.[27]

7.3.2.1 HYDROPHILLIC AND HYDROPHOBIC GROUTS

Hydrophillic polyurethanes are like sponges and will absorb water until they cannot hold any more. If the amount of water available is inadequate, the grout will not react completely. In this case, additional water can be injected into the crack to allow the grout to react completely. The disadvantage with this grout is that when water is no longer available, the grout will shrink (as much as 20 percent). However, the grout will expand to its original volume once the water returns.[21]

Hydrophobic polyurethane grouts on the other hand, require only a small amount of water to start their foaming reaction. In most cases, the moisture contained within the concrete being repaired is adequate to cause a foaming reaction. Unlike the hydrophillic grouts, hydrophobic grouts do not shrink once the water goes away.[21]

7.3.2.2 RIGID AND FLEXIBLE GROUTS

Polyurethane grouts can be used to repair nonstructural dormant cracks or moving cracks. For dormant cracks, a higher strength rigid grout with less elongation should be used. The tensile strength for rigid grouts ranges from 35 kPa (5 psi) to 100 kPa (15 psi), and the elongation can vary from 5 to 15 percent. Rigid grouts can expand by a factor of 15 to 20 times their original volume.[21]

For moving cracks or joints, a flexible seal should be used, because once it hardens, it can elongate as much as 250 percent. This characteristic maintains a tight, impermeable water seal even when the crack width increases. The tensile strength for flexible grouts varies from 0.86 to 1.20 MPa (125 to 175 psi), and can expand by a factor of five to eight times their original volume[21]

7.3.2.3 EXPANSION CONTROL

The rate of the chemical reaction (or induction period) between the grout and the water can be controlled by the use of a suitable accelerator. Proprietary polyurethane grouts are usually provided with accelerators that are compatible with the resins in the grout and are also provided with recommended dosage rates for obtaining the desired reaction times. Once polyurethane grout comes into contact with water, it can begin foaming in three seconds. In practice typical foaming times are varied from one to three minutes with the gel forming in two to five minutes.[21]

The time it takes the grout to foam and gel is also influenced by the temperature of the water it reacts with. For instance, an increase in water temperature from 10°C (50°F) to 30°C (86°F) will increase the gel time by about 25 percent. In most cases, trial batches will be required to establish the correct mix proportions to produce the desired reaction times.[21]

7.3.2.4 VARIABLES INFLUENCING PERFORMANCE

Application techniques often depend on material selection. The use of highly skilled grout applicators are essential to the success of the repair. In principle, all that is required is to permanently fill a crack or joint with a watertight seal which can withstand water pressure or structural movements. For most structures, this is relatively simple, however, remedial grouting of massive structures, such as dams and locks, a higher level of monitoring and engineering is required. To increase the rate of success in repairing water-bearing cracks or joints, there are several factors which should be considered as reported by Waring:[28]

(a) Reduce Variables. High water flow rates and pressures during grouting makes the injection process more difficult. To reduce these effects, "operational modifications, temporary dewatering, or mechanical chinking" is often required (Figure 7.6). For example, lowering the tailwater in a lock chamber can reduce water flow through the crack or joint being repaired (Figure 7.7). This makes it easier to determine the flow path of the grout without it being diluted by the water flow.

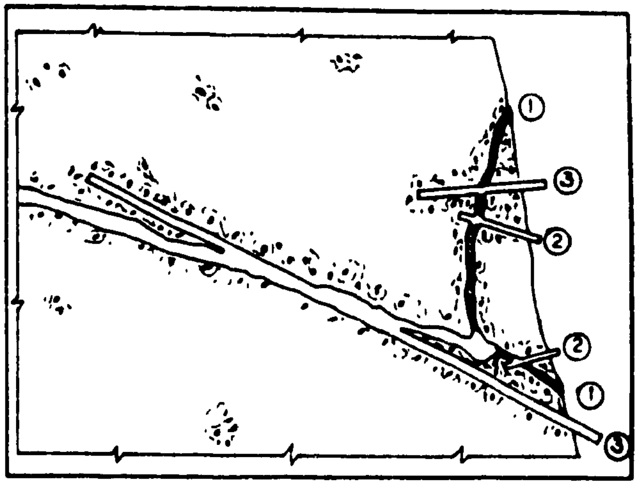


FIGURE 7.6 - MECHANICAL CHINKING: DRILL CRACKS AT (3) TO ALLOW DRAINAGE; PLACE CHINKING AT (1); AND DRILL SHORT HOLES AT (2) TO PLACE CHEMICAL GROUT BEHIND CHINKING[28]

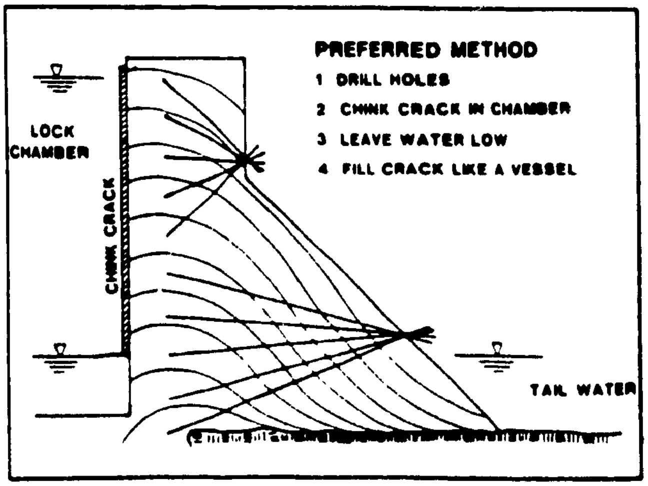


FIGURE 7.7 - GROUTING CRACKS IN LOCK MONOLITHS AT LOWER MONUMENTAL LOCK AND DAM[28]

High flowing water through monolith joints in dams can be canalized or controlled through valved joint drains (plastic, copper tubing, etc.) installed into the joints and sealing the joint between the drains (Figure 7 8)[27] In some cases, gasket-backed steel plates can be bolted to the wall to temporarily slow down the flow.[28]

FIGURE 7 8 - VALVED JOINT DRAINS AT CHIEF JOSEPH DAM, WASHINGTON[27]

(b) Seal Close to the Source Sealing close to the source is one method of increasing the time the grout remains in the crack and it is important for several reasons:

Firstly, this will allow the grout to travel through the full depth of the crack, providing more surface area to seat the grout plug Secondly, once the full depth of crack is sealed, the static water pressure is moved outside of the structure. Thirdly, in highly cracked concrete, water exiting from several locations may enter at one common point. If this point is sealed, all exit points will also be sealed *(c) Material Selection.* Prior to material selection, the behavior of the crack or joint should be clearly understood. The material which is selected should be able to match the characteristics of the problem. In general, polyurethane grout used for repairing water-bearing cracks or joints should meet the following criteria:[22]

Firstly, polyurethane grout must be 100 percent solids (or contain no solvent), since solvents cause shrinkage of reacted material when it evaporates. Secondly, it must be compatible with water and it should have a good adhesion to concrete. Thirdly, it should be hydrophillic and have a variable gel time. For example, if the distance between the point where the grout is delivered to the point where it is injected is very long, the induction period must be adjustable to compensate for the longer travel time.

In applications where the grout is likely to dry out, such as in hot, dry climates, one alternative is to select a polyurethane resin which does not form a gel. These grouts do not shrink due to "water vapor pressure equilibration."[28]

(d) Grout When The Crack is Widest. Most polyurethane grouts have a very low bonding capacity and have tensile strengths less than 3.5 MPa (500 psi). However, they can provide satisfactory results in shear or compression. This can be achieved by grouting when the crack is at its widest, causing the seal to remain in compression throughout its life. In actively leaking cracks, this will make the injection operation more difficult to execute since the flow volume of water is higher. However, this will significantly improve the long term performance of the seal.[28]

7.3.2.5 GROUTING TECHNIQUES

Because the nature of a crack and field conditions vary widely for each application, installation techniques for polyurethane grouts also vary. Most of the procedures involved in polyurethane grouting of dry cracks are similar to those for epoxy injection. For instance, port hole diameter and spacing usually follow the same guidelines for both epoxy and polyurethane injection. However, polyurethane grouting of water-bearing cracks can differ in several respects. There are some standard procedures that can be followed:

(a) Injection Holes. Injection holes are usually 12 mm (½ in.) to 15 mm (⅝ in.) in diameter, and are staggered on each side of the crack at a 45° angle to the concrete surface, as shown in Figure 7.9. However, for very thin and thick concrete sections, the drilling angle and hole depth

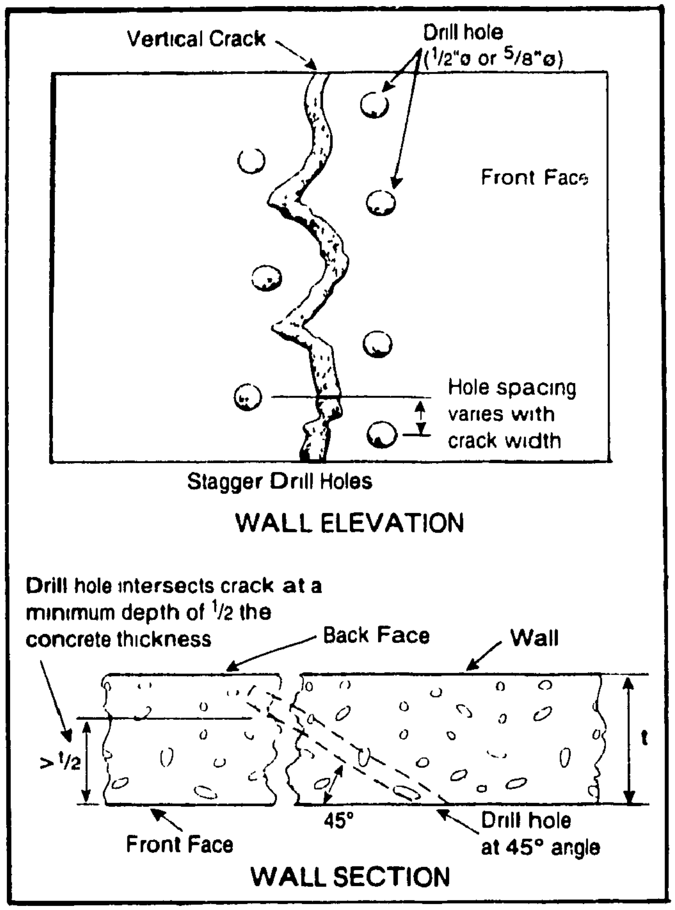


FIGURE 7.9 - PROCEDURE FOR DRILLING INJECTION HOLES[21]

change. For example, to minimize spalling in concrete sections less than 150 mm (6 in.) thick, the holes should be drilled directly into the crack and perpendicular to the concrete surface. For sections thicker than 900 mm (36 in.) the depth of the hole can be kept constant at about 450 mm (18 in.).[21] For massive concrete structures, larger diameter holes are commonly drilled to intercept the crack at considerable depths, as was done at Upper Stillwater Dam in Utah.[26] A self-powered hydraulic rotary drill was used to drill 50 mm (2 in.) diameter core holes to intercept the crack at depths ranging from 6 to 28 m (19 to 92 ft.). Once the hole was drilled, a pneumatic packer was set a few meters above the hole-crack intercept and the hole was injected with polyurethane resin.

Each grouting job will require a different drill hole spacing depending on the conditions and the nature of the crack. For instance, fine cracks will require closely spaced injection holes. In general, hole spacing varies from 150 to 900 mm (6 to 36 in.) but can also be as close as 100 mm (4 in.) and as far apart as 1.5 m (5 ft.). The holes should be staggered on each side of the crack and should not be too close to the crack.[21]

(b) Crack Cleaning and Sealing. Cracks are often filled with debris and mineral deposits left by water leakage and need to be cleaned. All the loose and soft surface concrete should be removed. A 75 mm (3 in.) wide strip of concrete on each side of the crack should be cleaned so that sealing and grout monitoring will be easier.[27] The standard concrete cleaning tools (such as grinding wheels, pneumatic wire brushes, steel scrapers, and high-pressure water jets) are usually adequate for performing the task.

In cases where the crack is wide or high water flows are encountered, the crack must be sealed at the surface to prevent the unreacted grout from being pushed out of the crack.[21] A variety of materials have been used to seal crack and joint entrances, and include: lead wool, chinking, oakum saturated with polyurethane grout, hydraulic cement, epoxy gel, and compressed wood particles (prestologs). Recently, polyurethane and silicone sealants were successfully applied by remote-controlled underwater methods to seal the joint entrances at Chief Joseph Dam in Washington.[27]

(c) Packers. Packers (or injection ports) are devices used to inject grout into the crack. There are several proprietary types of packers which can be basically grouped as mechanical, pneumatic, or "balloon" type. For small grouting jobs, mechanical packers are sized to fit a 12 and 15 mm (½ and ⅝ in.) drill hole, and vary in length from 50 to 65 mm (2 to 2-½ in.) If the quality of the surface concrete is poor, the packers can be inserted deeper into the structure by the use of commercially available "extenders" which can increase the packer length by 75 to 100 mm (3 to 4 in.) The packer is inserted in the hole and tightened with a wrench. As the packer is tightened, the rubber sleeve around the packer expands and prevents grout from leaking out of the hole. The packer is fitted with a male zerk (grease) fitting which prevents backflow of grout by a one-way ball valve. A pump pressure of at least 1.7 MPa (250 psi) is required to push the grout through the fitting.[21]

For massive structures, where deep and larger diameter holes are drilled, pneumatic or balloon-type packers are usually employed to deliver large volumes of grout.[27] Balloon packers are equipped with inflatable sacs which, once inflated, form a tight seal to prevent grout from exiting the hole. The packer can be positioned at predetermined depths (usually above the lowest crack in the hole) and grout is pumped through the packer until it fills the crack and reaches the packer. The packer is then repositioned at the next higher crack and the process is repeated to the top of the hole.

(d) Injection Equipment Depending on the type of grout used, a grout pump that can deliver a one - or two - component polyurethane resin is required.[21] For actively leaking cracks, the pump capacity must be adequate for injecting the grout into the crack and forcing the water out. If the flow rate can be significantly reduced, smaller pumps can be used.[28] There are three basic types of pumps that are used for polyurethane grouting: hand-operated, air-driven, and electrical positive-displacement. They can be either portable or truck/trailer mounted.

Injection pressures vary from 1.7 MPa (250 psi) to 20 MPa (3000 psi) and delivery rates vary from 4 to 20 liters (1 to 5 gal.) of grout per minute.[21] Most jobs require injection pressures between 3.5 MPa (500 psi) and 7 MPa (1000 psi). For large projects, where a high volume of polyurethane grout is needed, intermediate storage containers can be directly connected to the pump. In this case, the container is sealed and pressurized with dry nitrogen to between 140 and 275 kPa (20 and 40 psi) to maintain an oxygen and moisture-free environment for the grout and to deliver the grout supply to the pump under pressure.[27] When the ambient temperatures are low, electrical heat tape is very useful for preventing equipment from freezing and maintaining viscosities at a pumpable level.[10]

(e) Dye and Flow Tests. For most jobs, drill holes are expensive and it is essential to successfully seal a crack on the first attempt. Therefore, each drill hole should be thoroughly "prequalified" prior to injecting the grout. This can be achieved by performing a dye test and a pressure/flow test.[28]

The pressure/flow test is used to determine whether the expected pumping rate of the pump is adequate to deliver the required volume of grout. The dye test is useful for determining the amount of time the grout will take to penetrate the full depth of the crack. This is also useful for determining travel path and the appropriate gel time for the grout. There are two important points to consider when conducting these tests:[28]

Firstly, the same pump that is used to perform the tests should be used to perform the actual grouting operation. When using water-reactive grouts, a wet pump and a dry pump will be required. Secondly, these tests should be repeated after each hole is grouted, since the flow within the crack changes after each injection, which in turn changes the required gel times and pumping rates. To account for these continuous changes, dye and flow tests must be repeated. A generalized flow chart for drilling, testing, and grouting that was used for repairing cracks at Hells Canyon Dam[28] is shown in Figure 7.10.

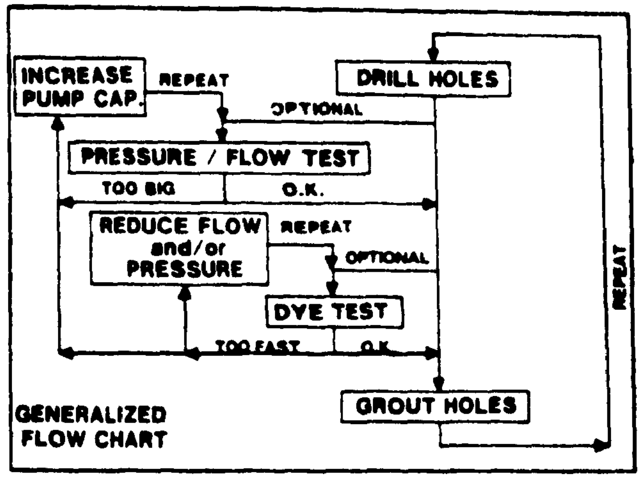


FIGURE 7.10 - FLOW CHART FOR DRILLING, TESTING, AND GROUTING AT HELLS CANYON DAM[28]

7.3.2.6 DIRECT AND INDIRECT INJECTION METHODS

There are two methods of injecting an actively leaking crack: direct and indirect injection.[7] The direct method consists of injecting the sealant against the flow of water from the downstream side. In the indirect approach, the grout is injected into the flow path of water on the pressure side, so that the crack network is filled by the action of water pressure. When using the direct approach, the first step is to confine the water flow through tubes through which the sealant may be injected. Injection tubes are installed at regular intervals along the crack or joint and leakage between them is sealed with a suitable material. The anchorage between the tube and the concrete must be able to withstand both the injection and hydrostatic pressures.

Two types of anchorages are typically employed: adhesive and mechanical systems.[7] Adhesive systems consist of applying putty or mortar to keep the injection tube in place. There are several proprietary methods available which confine water flow through tubes, allowing the use of adhesive anchorages. However, these systems are difficult to use and their performance depends on the skill of the worker. Mechanical anchorages are much easier to use and are not affected by flowing water conditions. For this reason, mechanical anchorages are usually preferred over adhesive systems. The most commonly used mechanical anchorage system is the injection lance (or wall spear). The lance comes with an expanding collar which forms a watertight seal when fitted into the drill hole. Its main advantage is it can withstand much higher injection pressures.

The indirect approach of sealing water-bearing cracks requires more careful planning. If not properly executed, the situation may be worsened. However, indirect injection methods have considerable advantages:[7]

- Large volumes of water leakage can be controlled
- Surface-sealing the crack or joint is not always required
- Higher injection pressures can be used, resulting in greater grout penetration
- When the grout is injected at the source of the water leakage, multiple exit points for the water can be sealed, resulting in considerable cost savings.

7.3.2.7 SPECIAL APPLICATIONS

(a) Sealing Leaks in Vertical Joints. Depending on accessibility, two methods are generally used for sealing vertical joints in concrete dams.[22] If the face of the dam is easily accessible, the joints can be sealed without using drill-hole grouting techniques. At Richard B. Russell Dam in Savannah, Georgia, the vertical joints were cleaned and cut out into a "V" shape, and a specifically patented (INJECTO) tube was installed in the V-groove along the entire length of the joint. A sealant was applied to the outside of the joint over the tube, as shown in Figure 7.11. After the sealant hardened, the grout was injected from the bottom of the joint through the tube until it emerged out from the top. By increasing the injection pressure, the grout was forced out through openings in the grout tube and into the joint to form the seal.

For situations where access is difficult, a combination of drill-hole and chemical grouting techniques can be employed. The procedure consists of drilling 75 or 150 mm (3 or 6 in.) diameter vertical holes through the entire length of the joint. The hole is injected with a flexible polyurethane grout through packers to form a new waterstop. A recent example was at Easton Dam, Connecticut[16] where a two component polyurethane grout was used to seal 16 vertical joints.

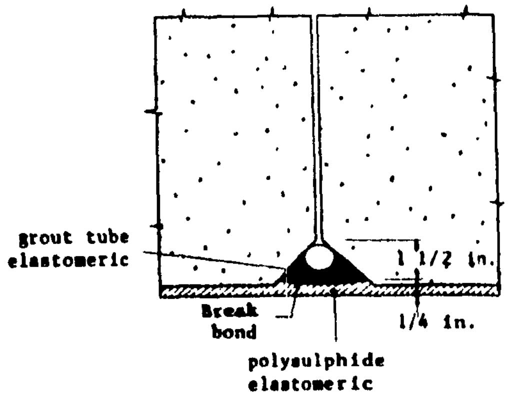


FIGURE 7.11 - REPAIR OF VERTICAL JOINTS IN RICHARD B. RUSSELL DAM - SAVANNAH, GEORGIA[22]

(b) Repairing Waterstops in Construction Joints. Waterstops embedded in concrete construction joints sometimes develop leaks which may be detrimental to the safe operation of the structure. At Applegate Dam[28] for instance, broken waterstops had developed leaks in two joints of the regulating outlet tunnel (Figure 7.12). The leaks were considered dangerous to the structure because of the potential for piping to occur. Grout holes were drilled at an angle to intercept the joint beyond the waterstop. The waterstops were embedded up to 600 mm (24 in.) deep into the concrete.

(c) Preventing Leakage in New Construction. An alternative to installing waterstops in construction joints is to place INJECTO tubes between separate lifts of concrete.[28] After the concrete hardens, the tubes are injected with the polyurethane grout. The grout penetrates into the construction joint and, once hardened, it forms a permanent watertight seal. This system can be used for grouting vertical, horizontal, or circular construction joints.

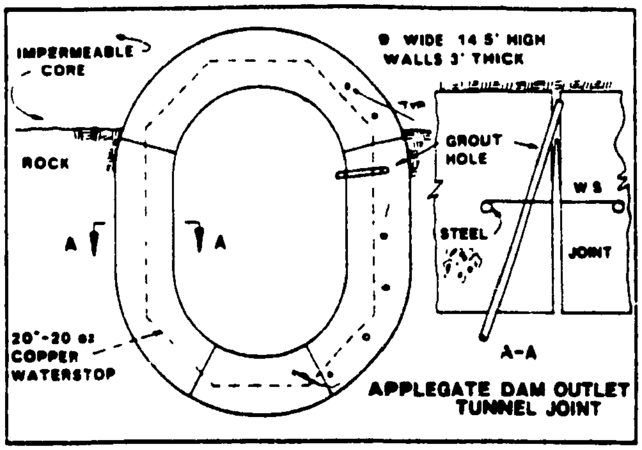


FIGURE 7.12 - WATERSTOP REPAIRS IN JOINTS AT APPLEGATE DAM OUTLET TUNNEL[28]

7.4 REPAIR OF DAM STILLING BASINS

Concrete hydraulic structures, such as spillway aprons, stilling basins, sluiceways, and outlet tunnel linings are susceptible to abrasion-erosion damage as previously described in Chapter 2. Abrasion-erosion damage results from high-velocity water flow containing silt, sand, gravel, rocks, and other debris. The rate and extent of erosion depends on several factors including the size, shape, quantity, hardness of particles being transported,[29] flow velocity, direction, and pattern and duration of exposure.[30] In some cases, abrasion-erosion damage ranging in depth from a few centimeters to a few meters can occur.[29]

There are several materials and techniques available for repairing abrasion-erosion damage to dam stilling basins. Some materials will perform better than others and no one technique is the most efficient and cost effective for all rehabilitation jobs. Many technological improvements have been made with materials and techniques which will allow for higher quality repairs, thus reducing repeat damage and life-cycle costs.

In selecting a repair alternative, several factors must be considered, including the time available to perform the repairs, site access, logistics in material supply, material compatibility, available equipment, interference with facility operation, and skill and experience of the local labor force.[29] Since dewatering of hydraulic structures for repair is usually difficult and very expensive, materials and methods which allow in-situ repair of these structures should be considered first.[32]

The following sections present a summary of materials and techniques used in repairing stilling basins subject to erosion damage. Although a majority of the repairs are performed in dry conditions, some materials and techniques have been developed for repairing these structures under water, avoiding the high cost of dewatering. A good review of several case histories involving the rehabilitation of navigation lock walls is provided by McDonald in Reference 62.

7.4.1 SURFACE PREPARATION

As with all concrete repair jobs, proper surface preparation is important for adequate bonding between the substrate and new material. Virtually all surface cleaning techniques described in Chapter 3 can be used for cleaning stilling basins in the dry or under water.

7.4.1.1 SEDIMENT REMOVAL

The first step in repairing stilling basins is the removal of sediment and debris. The removal method employed depends on the sediment and debris present.[31] The three most commonly used methods are: air lifting, dredging, and jetting.[33] The best method for sediment and debris removal depends on the following factors:[34]

- The type of material to be removed: soft or hard, fine grained or coarse grained, and the maximum size of the particle.
- The horizontal and vertical distances through which the material must be moved.
- The volume of material to be removed.
- Water depth, currents, and wave action.

Each of the techniques is discussed in more detail in the following sections and general guidance on the suitability of each method is provided in Table 7.5.

(a) Air Lifting. The air lift uses a suction pipe to remove the material. A "density differential" is created by the air introduced into the lower end of a partially submerged pipe (Figure 7.13). When the air bubbles combine with the water in the pipe, a mixture with a density less than the

water outside the pipe is created. This causes a suction at the inlet as the less dense mixture rises in the pipe. The quantity of material lifted depends on the size of the air lift, the submerged depth of the pipe, the air pressure and volume, and the discharge head. The size of the discharge pipe used depends on the type and amount of material to be removed and can vary from 75 to 300 mm (3 to 12 in.) in diameter. Generally, the air lift can be from 3 to 21 m (10 to 70 ft.) long, but is reported to be inefficient in lengths less than 9 m (30 ft.).[33]

TABLE 7.5 - GUIDANCE ON UNDERWATER EXCAVATION TECHNIQUES[33]

Excavation Factor	Excavation Method		
	Air Lift	Jet	Dredge
Type of seabed material	mud, sand, silt, clay, cobbles	mud, sand, silt, clay	mud, sand, silt, clay
Water depth	7 6 to 22.8 m (25 to 75 ft)	unlimited	unlimited
Horizontal distance material moved	short	short	short to long
Vertical distance material moved	short to long	short	short to medium
Quantity of material excavated	small to large	small to medium	small to medium
Local current	not required	required	not required
Topside equipment required	compressor	pump	pump
Shipped space/weight	large	small	medium

(b) Dredging Dredging is a useful method for removing large quantities of soft material. This is used when the water is too shallow for an air lift to be used effectively. A typical underwater dredging arrangement consists of a tube or pipe with a 30° bend near the intake end (Figure 7.14). A water jet is connected at the center of the bend and directed towards the discharge end along the centerline of the pipe to create a suction at the intake end. The size of the pipe and the output of the pump will both influence the height to which the material can be lifted.[35] A pump with a capacity of 0.75 m^3/min. (200 gpm) and a 150 mm (6 in.) diameter pipe will lift material as high as 18 m (60 ft.) above the bottom surface. When the pipe operates only a few meters above the bottom, the dredge can move as much as 7.6 m^3/hr (10 cy/hr) of mud, sand, and loose gravel.[34]

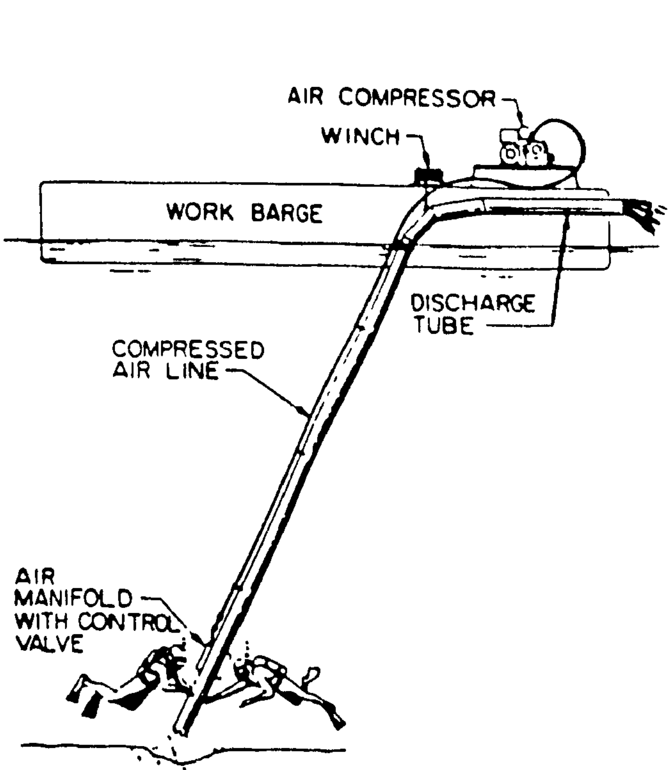


FIGURE 7.13 - AIR LIFTING FOR REMOVING SUBMERGED FOULING MATERIALS[33]

(c) Jetting. Jetting is generally employed to move large quantities of silt, sand, or mud by directing a high-velocity water jet at the material to be moved (Figure 7.15). Two different jetting techniques are commonly used in practice.[33] In the first method, bed material is moved by erosion with the use of a large jet mandril. This method is useful for moving mud and some noncohesive materials like sand. In the second method, noncohesive sandy soils are "fluidized" and moved by using several small jets.

(d) Jet-Dredge Tool. In general, sediment removal using a jetting technique is not very efficient, because although the jet fluidizes the sediment, it has no way of transporting the material. On the other hand, dredging does not have a mechanism for fluidizing the sediment, and often requires a diver to break up and fluidize the material in front of the suction tube.[33] As a result, a diver operated jet-dredge tool was developed by the Naval Civil Engineering Laboratory (NCEL) in the United States which combines the fluidizing jet and a dredging jet.[36] A jet "eductor" is a device which increases the flow of water through the jetting nozzle and reduces the amount of

pump horsepower needed to counteract the backthrust which is developed within the tool. The excavation tool consists of a jet eductor, a jet nozzle, and a hydraulically powered sump pump. The tool can achieve an average excavation rate of 25 m^3/hr. (35 cy/hr.) depending on the soil characteristics and the underwater conditions.

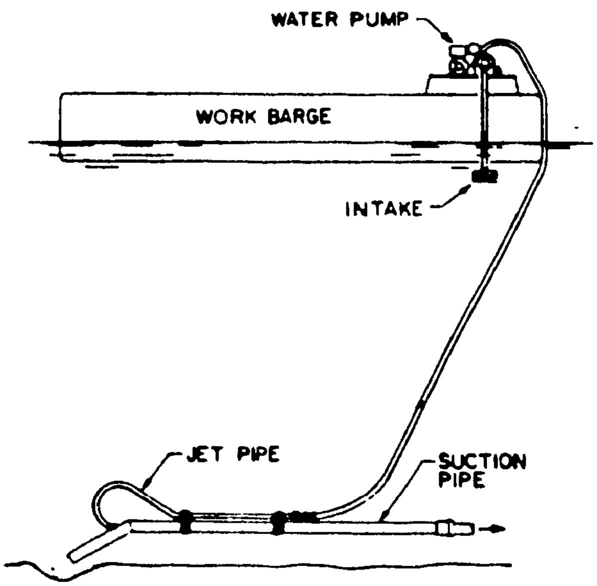


FIGURE 7.14 - UNDERWATER DREDGING SYSTEMS[33]

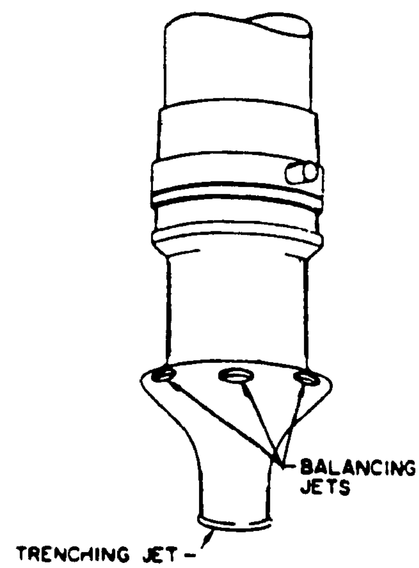


FIGURE 7.15 - JETTING NOZZLE[33]

7.4.1.2 CONCRETE REMOVAL

High-pressure water jets can be used to break up and remove deteriorated concrete, roughen the concrete surface for better bonding, and cut a neat edge around the perimeter of the eroded area. These tools can either be used by divers or remotely controlled from the surface.[31] Controlled blasting may also be used to remove concrete and large obstacles such as rocks and boulders.[33] For most applications, hydraulic breakers (or hoe rams) can provide a cost effective solution when performed in dry conditions.

7.4.1.3 DOWELING

Doweling is sometimes necessary to ensure adequate bond between the new and existing concrete. Doweling consists of drilling holes into the underlying surface and bonding a reinforcing bar into the hole with cement or epoxy grout.[31] An example of this procedure was used in making repairs to Kinzua Dam stilling basin in western Pennsylvania where erosion damage reached a depth of 1.1 m (3.5 ft.).[37] In this case, 25 mm (1 in.) diameter U-shaped dowel bars were installed to help anchor the new concrete overlay to the underlying concrete floor.

A more extensive procedure was used during repairs made to Dworshak Dam stilling basin near Orofino, Idaho, where erosion damage varied between 30 mm (1.2 in.) and 2.7 m (9 ft.) deep.[12] To relieve uplift pressures which developed as a result of clogged uplift pressure drains, prestress anchor bars were installed into the existing floor at specified locations to a depth of 10.3 m (33.8 ft.). The eroded areas were then filled to within 380 mm (15 in.) of the final floor elevation with 40 mm (1-½ in.) maximum size aggregate structural-grade concrete. The 380 mm (15 in.) deep fiber concrete overlay was held in place with 25M (Nº 8) anchor bars placed at 1.5 m (5 ft.) centers and hooked (180°) to a light mat of horizontal 15M (Nº 6) reinforcing bars placed on 380 mm (15 in.) centers (Figure 7.16).

In situations where dewatering is not possible or expensive, doweling has been typically done by divers. Since this is labor intensive and costly, only a small number of dowels can be placed. To compensate for this disadvantage, the NCEL has developed a method for placing power actuated dowels under water.[31] This system can install a greater number of dowels over a large area. For example, almost 600 prestressed rock anchors were installed in the stilling basin floor at Tarbella Dam in Pakistan to counteract hydrostatic uplift forces.[38] CemBol T, which is a cartridge product marketed by Sweden, provides a quick way for installing large anchor bolts under water. The bolt is totally embedded and corrosion-proof. At 10°C (50°F), the bolt can reportedly support a load of up to 5 tons/m after three hours and more than 15 tons/m after 24 hours.[31]

7.4.2 CONCRETE MIX PROPORTIONS AND PROPERTIES

A variety of materials and material combinations are used for the repair of stilling basins. A major factor which is vital to the success of the repair is the relative volume change between the repair material and the substrate. Ordinary portland cement concrete is usually the least expensive repair material and its thermal properties are compatible with the substrate concrete.[29] Two other key factors that control the type of concrete to be used for underwater repair are: "the workability of the fresh concrete and the abrasion resistance of the hardened concrete."[31] In general, denser, stronger, and more ductile materials provide greater abrasion resistance. The use of admixtures and pozzolans, such as condensed silica fume, have significantly increased the abrasion resistance of concrete. The method chosen for placing and finishing the repair concrete will dictate the degree of workability of the concrete required.

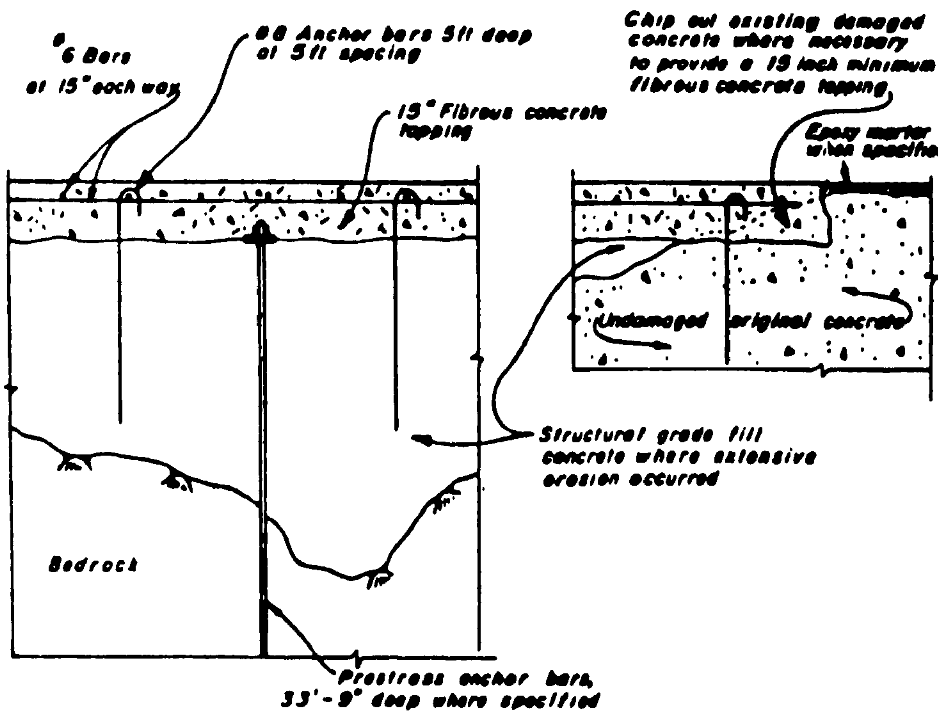


FIGURE 7.16 - REPAIRS TO STILLING BASIN AT DWORSHAK DAM, IDAHO[12]

7.4.2.1 CONVENTIONAL CONCRETE

Conventional portland cement concrete (PCC) is easy to obtain and place, and is usually the least expensive repair material available.[29,31] In the past, this type of concrete typically performed poorly when exposed to severe impact and erosion conditions. However, early studies[39,40] showed that through proper mix proportioning, placement, surface treatment, and curing, ordinary PCC can be made more resistant to abrasion-erosion damage and that the concrete resistance to erosion increases as the compressive strength is increased.

More recent work done by Liu[41] and Holland[42] support the general findings that conventional concrete with higher compressive strengths increases concrete resistance to abrasion wear. Since concrete strength depends on water-cement ratio and curing, the Portland Cement Association (PCA)[43] reports that a low water-cement ratio and adequate curing are necessary for abrasion resistance. Further, it states that aggregate type also affects abrasion resistance of concrete. Similarly, the ACI 210R-87[29] report recommends using the maximum amount of hardest available aggregate and the lowest practical water-cement ratio for abrasion-resistant concrete.

Laboratory testing performed during repair work to Kinzua Dam stilling basin in Pennsylvania showed that the abrasion-erosion resistance of concrete with a water-cement ratio of 0.45 containing chert is approximately twice that of concrete with the same water-cement ratio containing limestone.[37] It is interesting to note that the compressive strength of the concrete containing chert (32 MPa) was less than that of the concrete containing limestone (39 MPa). Another example of successful abrasion-erosion repair work using conventional concrete was performed at the Ilha Solteira Dam and Marimbondo Dam stilling basins in Brazil[44] where deterioration varied from superficial erosion to damaged reinforcing steel. In this work, the repair concrete included 19 mm (¾ in.) crushed basalt coarse aggregate and a water-cement ratio of 0.44. The mixture produced 28 day and 90 day compressive strengths of approximately 37 MPa (5400 psi) and 45 MPa (6500 psi), respectively. A thin coat of epoxy resin was applied to the dry substrate to provide good bond to the repair concrete. In areas where reinforcing was damaged, dowels were drilled and grouted into the substrate and the reinforcing steel replaced. Inspections with underwater television cameras revealed that repairs performed satisfactorily.

7.4.2.2 HIGH-STRENGTH CONCRETE

Research has shown that high-strength concretes with 28 day compressive strengths in excess of 80 MPa (12,000 psi) are "excellent" for resisting abrasion erosion damage.[31] However, very high strength concrete is difficult to place in dry or underwater areas without the use of special admixtures, which considerably increase material costs. In some cases, where hard aggregate is not available, high-range water reducers (HRWR) or superplasticizers, and silica fume can be used to develop concretes with compressive strengths in excess of 100 MPa (15,000 psi). The abrasion-erosion resistance of such high-strength concrete depends more on the hardened cement paste than on the hardness of the aggregate.[29]

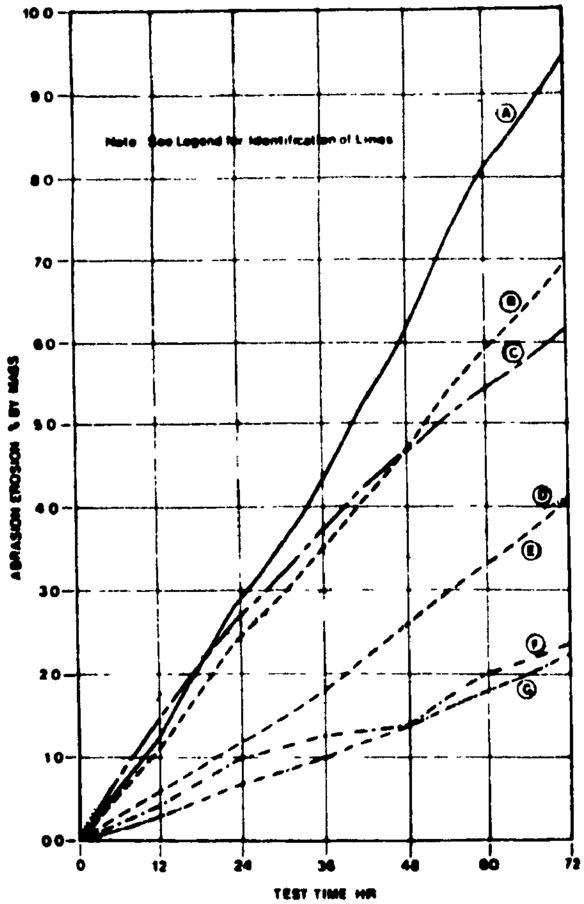
7.4.2.3 SILICA FUME CONCRETE

Studies[45,46,47] conducted by the Bureau of Reclamation in the United States indicate that silica fume concrete offers significant improvement in abrasion-erosion resistance over conventional concrete. Several successful field tests using silica fume concrete as a repair material were performed at three dams between 1985 and 1987.[30] For example, an addition rate of 13 percent silica fume by weight of cement was used in the repair concrete at Palisades Dam in Idaho for repairing the stilling basin floor. The mixture resulted in a water to cement plus silica fume ratio of about 0.35. Inspection after one season of operation revealed essentially no erosion to the silica fume concrete.

The feasibility of placing large volumes of very high-strength concrete containing silica fume was demonstrated during repair work done to Kinzua Dam stilling basin.[37] Abrasion-erosion test data for various concretes tested in the laboratory indicated that the silica fume concretes exhibited less abrasion-erosion than various conventional concretes (Figure 7.17). The mixture proportions and mechanical properties for these concretes are shown in Table 7.6.

The addition of 15 percent silica fume by weight of cement produced a mixture with a water to cement plus silica fume ratio of 0.30. This resulted in a 28 day concrete compressive strength of about 95 MPa (13,775 psi). Subsequent inspection one year later revealed deterioration of about 12 mm (½ in.) only along some cracks which developed after placement.

Although placement of silica fume concrete has proved generally successful, the work at Kinzua Dam demonstrated that large overlays are susceptible to surface cracking. In this project, cracking was attributed primarily to high restraint of volume changes resulting from thermal expansion and contraction, and possible autogenous shrinkage. Since silica fume concrete has a tendency to dry on the surface, difficulty may be encountered during finishing. At Kinzua Dam, this problem was minimized by limiting the number of screed passes to two, and applying a curing compound immediately after screeding.



LEGEND

A. Fiber-reinforced concrete from Kinzua stilling basin

B. Conventional concrete, Pennsylvania limestone aggregate

C. Conventional concrete, Virginia diabase aggregate

D. Conventional concrete, Mississippi chert aggregate

E. Average of silica-fume concrete specimens prepared during actual construction

F. Silica-fume concrete, Virginia diabase aggregate

G. Silica-fume concrete, Pennsylvania limestone aggregate

FIGURE 7.17 - ABRASION EROSION TEST DATA FOR VARIOUS CONCRETES[37]

7.4.2.4 FIBER-REINFORCED CONCRETE

Repairing erosion damage to hydraulic structures with fiber-reinforced concrete (FRC) has seen inconsistent results. Laboratory and field research indicate that the addition of steel fibers may

TABLE 7.6 - CONCRETE PROPERTIES FOR VARIOUS MIX PROPORTIONS[37]

Mix	Aggregate	Cement Content, kg/m^3 (lb/yd^3)	Silica Fume Content % by Cement Mass	Water/(Cement + Silica Fume) Ratio	Compressive Strength, 28 Days, MPa (psi)	Abrasion-Erosion Loss, % by Mass
1	Pennsylvania Limestone	317 (534)	0	0.45	39.4 (5,710)	6.9
2	Pennsylvania Limestone	269 (454)	17.6	0.53	49.5 (7,180)	5.0
3	Pennsylvania Limestone	351 (592)	42.9	0.21	95.5 (13,850)	2.2
4	New York Diabase	317 (534)	0	0.45	40.7 (5,910)	7.7
5	Virginia Diabase	317 (534)	0	0.45	39.1 (5,670)	6.1
6	Virginia Diabase	269 (454)	17.6	0.53	58.5 (8,480)	4.3
7	Virginia Diabase	351 (592)	42.9	0.21	95.2 (13,810)	2.3
8	Chert (WES Reference Concrete)	346 (584)	0	0.45	32.7 (4,740)	4.1
9	Kinzua FRC	NA	NA	NA	NA	9.4

Note The fiber reinforced specimens were prepared from a large fragment of concrete taken from the Kinzua stilling basin
FRC = Fiber Reinforced Concrete - NA = Not Applicable

accelerate abrasion-erosion damage.[48] This acceleration occurs when the steel fibers are pulled out of the concrete when it is subjected to the erosive action of high-velocity water.[31] However, there are some cases where FRC repairs have demonstrated that fibrous concrete can be made to resist abrasion-erosion damage. For instance, repairs performed at Tarbella Dam stilling basin in Pakistan[38] consisted of placing a 600 mm (24 in.) thick FRC slab to fill the scoured area. The repairs have proven to be satisfactory. FRC was also used for repairs to Dworshak Dam stilling basin.[12] The mix contained a heavily sanded 19 mm (¾ in.) maximum-size aggregate concrete into which 25 mm (1 in.) long steel fibers (10 mils x 20 mils in cross-section) were added at a rate of 1.2 percent (by volume) per cubic meter of concrete. These repairs were also reported to be satisfactory.

Proprietary FRC has been developed which exhibits excellent abrasion resistance with a steel fiber content as low as 1.5 percent by volume. This resistance has been attributed to the excellent bond between the steel fibers and the concrete.[31] In conventional concrete, fibers can be useful in controlling cracks in flat surfaces, especially in stilling basin floors. Proper use of fibers can substantially reduce cracking and improve resistance to fatigue, thermal shock, and cavitation damage.[49] The most significant benefits are extended fatigue life, and tremendous ability to absorb energy and resist impact damage.[12] ACI publications that give additional information on current practice include three reports by ACI Committee 544[50,51,52] and the proceedings of a recent symposium.[53]

7.4.2.5 POLYMER CONCRETE

Test results from studies conducted by the Bureau of Reclamation [45,46] indicate that polymer concrete (PC) shows significant improvement in abrasion-erosion resistance over conventional concrete. PC, such as epoxy and resin concrete, is a rapid-setting mixture of fine and coarse aggregate with a polymer as a binding agent between the aggregate particles.[29] PC exhibits good chemical resistance and exceptional bonding characteristics, and has demonstrated excellent abrasion resistance. Polymer concrete has not been used extensively for making large scale repairs of hydraulic structures due to its high cost, however, it has proven to be cost effective in repairing small areas which are unsuitable to conventional portland cement concrete.[31] Currently, PC cannot be placed and cured under water. However, PC may be useful for fabricating precast concrete panels which can be placed under water.

Recent examples of PC repairs to stilling basins include Milburn Dam in Nebraska and Shadehill Dam in South Dakota.[30] At Shadehill Dam, three layers of similar PC mixtures were used to repair a 16 m^2 (172 ft^2) eroded area of the spillway to a depth of approximately 115 mm (4.5 in.). The mixture for use below the reinforcing steel contained 40 mm (1-½ in.) maximum-size aggregate (MSA). The mixture used for covering the reinforcing steel contained 16 mm (⅝ in.) MSA, and the mixture used for the top 12 mm (½ in.) of the repair contained no additional aggregate. Inspection of the repair two seasons after operation indicated minor abrasion to the surface of the polymer concrete.

7.4.2.6 POLYMER IMPREGNATED CONCRETE

Polymer impregnated concrete (PIC) is a hardened concrete that has been impregnated with a monomer which is hardened within the concrete pores. Schrader[47] and Liu[54] have shown that by impregnating and effectively case hardening the concrete surface with a polymer, the cavitation and abrasion-erosion resistance of concrete is greatly improved.

Polymer impregnation of concrete consists of drying the concrete surface (at high temperatures), soaking the dried surface with a monomer (such as an acrylate or styrene) that fills voids and cracks, and then polymerizing (or solidifying) the liquid in place with a heat source. "The depth of monomer penetration depends on the porosity of the concrete and the process and pressure under which the monomer is applied."[11,12] When polymerized, this system becomes a material similar to clear plastic.

Test results indicate there is significant improvement in concrete properties after impregnation. Compressive strengths can be as much as five times their original value. Porosity and permeability are reduced while resistance to freeze-thaw cycles and chemical attack is greatly improved.[11] Surface impregnation was used at Dworshak Dam to repair cavitation and abrasion-erosion damage to the regulating outlet tunnels and stilling basin.[55] Polymer impregnation is more easily used for repairing horizontal surfaces, but it can also be adapted for repairing vertical surfaces. Typical sections through the apparatus used for impregnating the outlet walls (vertical surfaces) and stilling basin (horizontal surface) at Dworshak Dam are shown in Figures 7.18 and 7.19, respectively. Currently, PIC cannot be placed under water[31] and its use for repairing concrete hydraulic structures requires dewatering.

7.4.2.7 CONCRETE ADMIXTURES

Until recently, concrete hydraulic structures that required repair below the water line had to be dewatered. However, recent developments in concrete admixtures have made it possible to place better quality concrete under water with higher abrasion-erosion resistance.[31] Results of a test program conducted at the U.S. Army Waterways Experiment Station indicated that cohesive, flowable, and abrasion-resistant concrete could be placed under water by adding an antiwashout admixture (AWA).[56]

A recent example of a successful underwater rehabilitation project using an AWA was the repair to the eroded end sill of Red Rock Dam.[57] This was the first U.S. Army Corps of Engineers concrete structure to be repaired in the end sill area using underwater concrete placement techniques. In this repair, shown in Figure 7.20, an AWA and a water-reducing admixture were used to produce a flowable and cohesive concrete. A diver was used to control the end of the pump line, keeping it embedded in the newly poured concrete. The AWA helped prevent loss

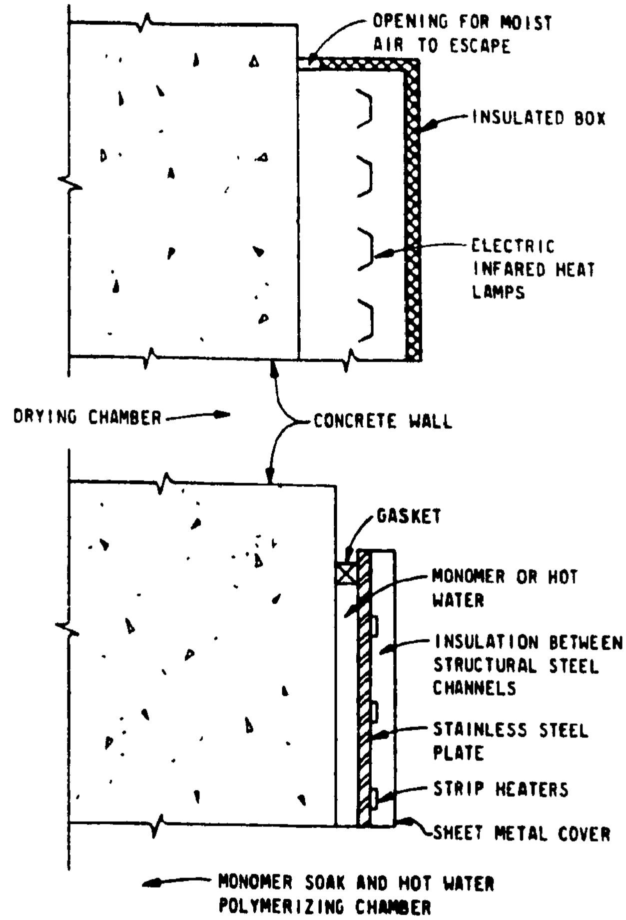


FIGURE 7.18 - TYPICAL SECTION SHOWING THE ENCLOSURES FOR DRYING, SOAKING, AND POLYMERIZING THE OUTLET WALLS AT DWORSHAK DAM[55]

of fines and formation of "rock pockets" when the pump line was accidently removed from the mass of freshly poured concrete.

Conventional water-reducing admixtures and superplasticizers are also very important in producing abrasion-resistant concrete for underwater use. Superplasticizers are a vital ingredient for facilitating the placement of pumped concrete especially when used with an AWA.[31] Pozzolans, such as silica fume and fly ash should also be considered for use in underwater repairs because they produce concrete with higher density, strength, and bond. A superplasticizer, but not silica fume, was used in the concrete for repairing stilling basins at two dams operated by the Swedish State Power Board, in Sweden.[31] Repairs were performed after the structures were dewatered using a concrete with a very low water-cement ratio. Neither anchors, epoxy bonding compounds, nor surface coatings were used, however, repairs appeared to perform satisfactorily.

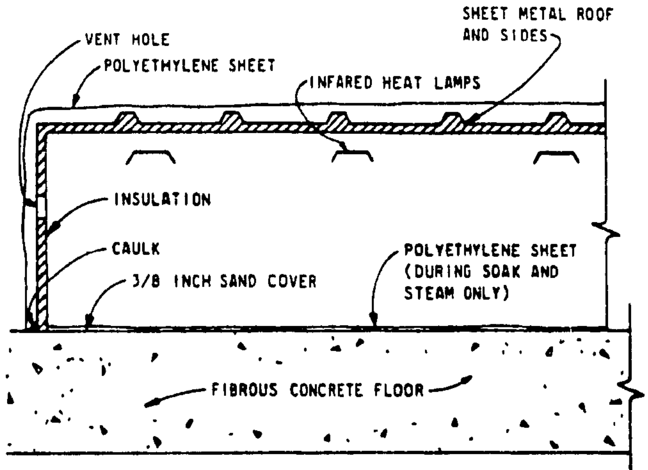


FIGURE 7.19 - TYPICAL SECTION SHOWING ENCLOSURE FOR DRYING, SOAKING, AND POLYMERIZING THE STILLING BASIN FLOOR AT DWORSHAK DAM[55]

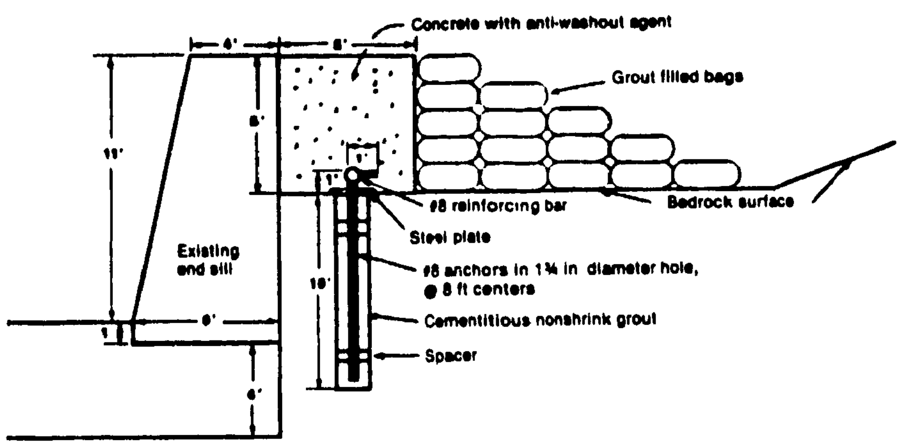


FIGURE 7.20 - END SILL REPAIRS AT RED ROCK DAM USING CONCRETE WITH AWAs.[57]

7.4.3 VACUUM AND PRESSURE PROCESSING

Vacuum and pressure processing is used to increase the strength of the concrete by compacting or removing voids from the freshly poured concrete. It is reported that vacuum processing is usually more efficient because "it removes excess surface air and both surface and internal water from the concrete."[31] Vacuum processing can also reduce the water-cement ratio of the concrete. Work done by Liu[41] has shown that vacuum processing at about 80 kPa (12 psi) for 15 minutes can increase the abrasion resistance of concrete with an initial water-cement ratio of 0.54 by about 40 percent. Since vacuum processing uses the ambient pressure as a source of compaction, a portion of the fresh concrete must be exposed to the atmosphere so that the excess water can be pushed toward the vacuum by the atmospheric pressure. The vacuum consists of a filter mat with a watertight backing with a gasket around the perimeter. The space formed by the gasket is filled with layers of porous filter material which prevent the cement and fines from being drawn out of the concrete with the excess water. Pressure processing requires the use of an external source to provide the pressure, such as a pressure plate, a roller compactor or hydraulic pressure.[31]

Although these methods have not been commonly used in submerged conditions, both processes could be easily adapted for treating concrete under water. Using this process under water can be expected to yield better results than on land, because the additional hydrostatic water pressure available would maintain a higher suction pressure than that available on land. Using vibration in conjunction with vacuum or pressure processing can help to remove voids within the concrete.[31]

7.4.4 PLATE LINERS

Refacing stilling basins with stainless steel liner plates has been used with some degree of success for protecting concrete against cavitation erosion.[29] Studies performed by Colgate[58] showed that stainless steel is about four times more resistant to cavitation damage than conventional concrete. Currently, the preferred stainless steel material is standardized by ASTM A 167, S30403 (formerly SS304L), with regards to corrosion and cavitation resistance, and weldability.

One important aspect of using this method is the steel plates must be securely anchored in the underlying concrete to prevent or minimize the effects of vibration. Vibration of the liner plate will eventually lead to failure of the anchors or the underlying concrete. It should also be noted that the liner plates usually hide early signs of concrete distress.[29] For this reason, this method is rarely chosen as a repair option.

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CHAPTER 8

CASE STUDIES

8.1 SPECIAL UNDERWATER REPAIR OF A COASTAL CONCRETE STRUCTURE IN A TIDAL AREA[1]
Eastern Scheldt, The Netherlands

8.1.1 SUMMARY

This case study describes the special aspects of underwater repairs to a concrete storm surge barrier damaged during its construction. The structure is located in the Eastern Scheldt in the Delta area of the Netherlands. A description of the cause and extent of the damage is provided along with an explanation of the materials and construction methods used for executing the required repairs in the existing difficult hydraulic conditions.

8.1.2 DESCRIPTION OF STRUCTURE

The storm surge barrier, comprising a total of 65 massive concrete piers, is constructed of various concrete elements which form the frame for the steel gates and operating machinery of the barrier. A cross-section of the barrier is shown in Figure 8.1. The concrete structure is deeply embedded in special filter mattresses consisting of stones of various sizes. Many parts of the structure were prefabricated and were fitted together with the aid of special construction equipment. The main components are the concrete piers, bridge box girders, sill beams, upper beams, the steel gates and the operating machinery. The structure was designed for a service life of 200 years.

8.1.3 THE CAUSE AND EXTENT OF DAMAGE

During the construction of the storm surge barrier, five of the 65 piers were damaged by the auxiliary construction that was installed to prevent damage. Since the inner sill was to be constructed by means of stone depositing barges (Figure 8.2), it was anticipated that, as a result of the water current, stones dumped around the piers would land in the recesses of the steel gates and cause unacceptable damage to the special low friction sliding plates (Figure 8.3).

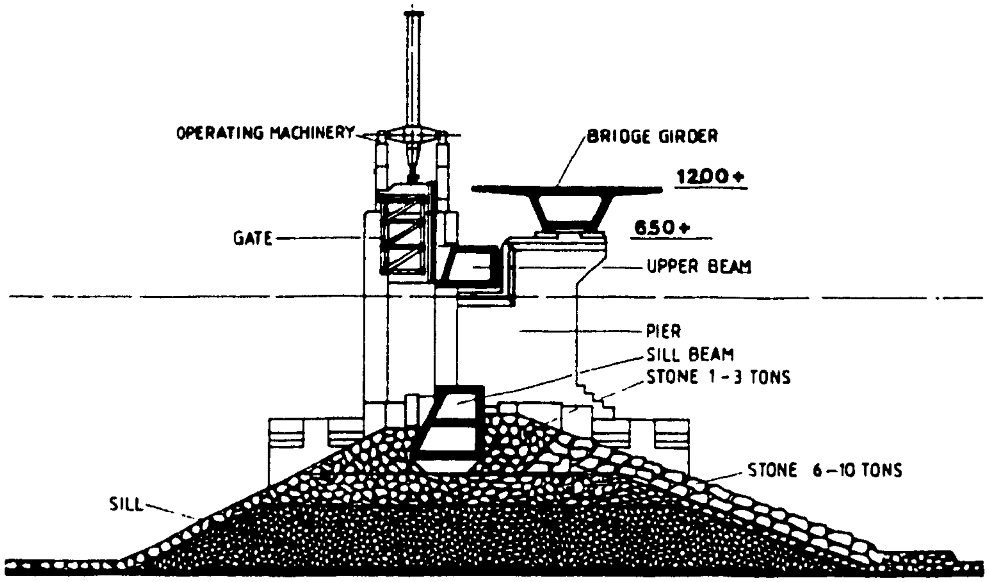


FIGURE 8.1 - CROSS-SECTION OF STORM SURGE BARRIER

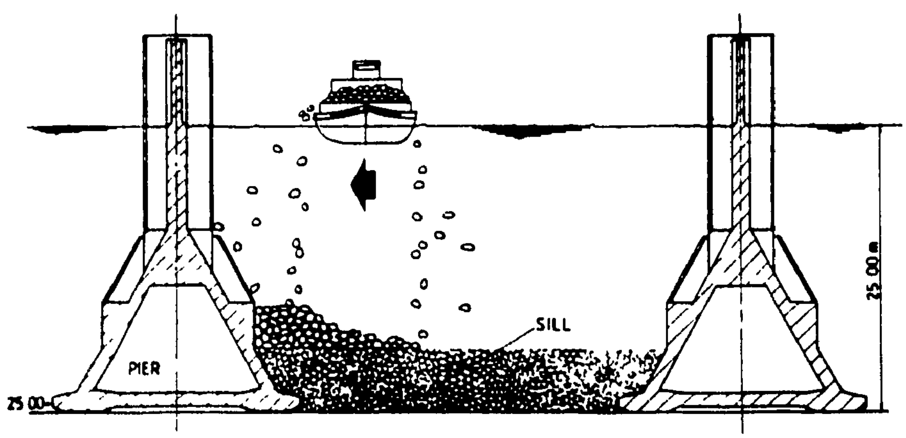


FIGURE 8.2 - DUMPING SILL STONE WITH BARGES

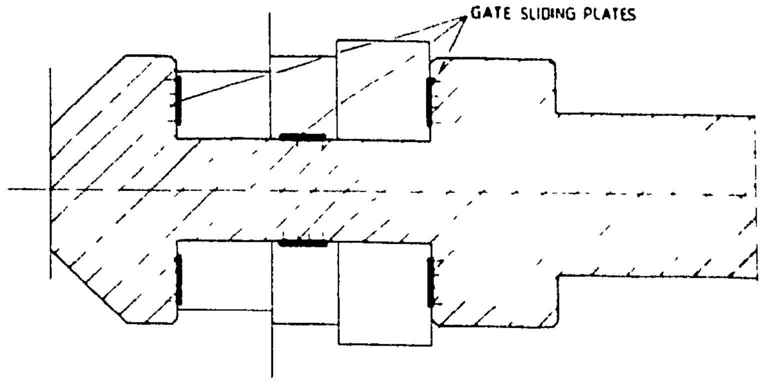


FIGURE 8.3 - CROSS-SECTION OF PIER SHAFT AND GATE RECESSES

In order to prevent such damage, various measures were undertaken to protect the elements of the structure, including the a ication of protecting layers to the piers and the dumping of the stones by specially designed equipment. To prevent damage to the sliding gates, a special safety net was constructed and it was stretched between two beams fixed to the pier structure, as shown in Figure 8.4.

During severe weather conditions, the anchorage between the lower beams and the concrete piers became disconnected. The water current and the waves made the beam sway, causing damage to the bearing corbels of the piers. The damage was located at a depth of between 5 to 8 m (16 to 26 ft.) below the water line, as shown in Figure 8.5. Several square meters of concrete cover had been worn away, exposing the steel reinforcement. Although the damage was not extensive, it was critical because it was located in an area of high internal stresses.

8.1.4 SPECIAL ASPECTS OF UNDERWATER REPAIRS

During the design phase of the barrier, it was anticipated that damage to the structure would occur depending on the method of construction. As a result, an investigation program was initiated for determining the most suitable material and technique for repairing the underwater concrete elements. Since the barrier was designed to provide a service life of 200 years, much attention was focused on the quality and durability of the repairs.

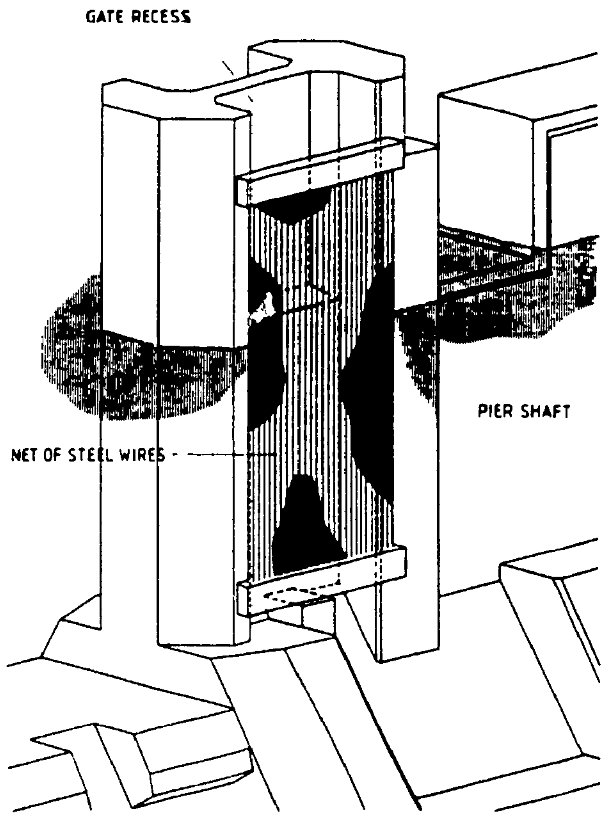


FIGURE 8.4 - GATE RECESS PROTECTION SCREEN

Two types of repair materials were considered: one was based on synthetic resin and the other was based on cementitious mortars. The requirements which were examined are: a good bond, low water permeability, and the development of an alkaline environment within the repair concrete. Previous investigations had shown that repairs made with a resin-based concrete mix were not suitable because it had a high porosity, and did not provide an alkaline environment. As a result, a cementitious mortar mix was selected as the repair material.

A subsequent research program, consisting of semi-practical tests in the intertidal zone of concrete structures situated in the Scheldt Estuary, and laboratory tests, determined that a number of factors may influence the bond strength of the surface between the new and existing material. The semi-practical tests showed that marine growth had an adverse effect on the bond, and that a surface would be covered with marine growth within 24 to 36 hours after it had been cleaned, depending on seasonal conditions. Other parameters which may influence the bond strength include:

- Surface preparation
- Orientation of the bond area (interface)
- Method of concrete placement

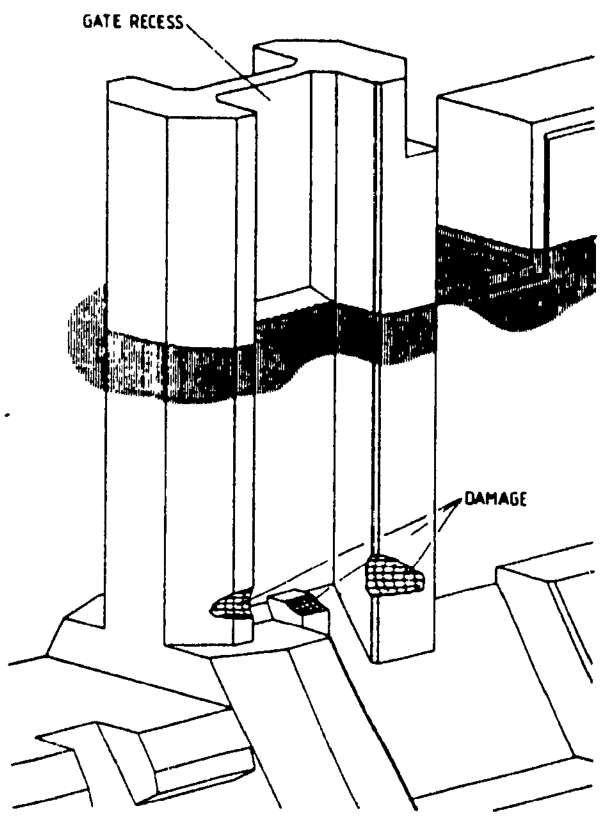


FIGURE 8.5 - LOCATION OF DAMAGE TO PIER STRUCTURE

8.1.5 RECOMMENDATIONS FOR REPAIR

Based on the research, it was determined that all marine growth and damaged or unsound concrete was to be removed. One approach to eliminating the fouling problem was to pour the concrete within 24 hours after surface cleaning. Since this was not always possible, a special coating was developed which could be applied to the surface after cleaning and removed by high-pressure water immediately before pouring the concrete.

A good quality repair could be achieved by the use of pumped concrete with the discharge opening continuously submerged under the fresh concrete surface. It was determined that the repair concrete was to have a high workability, good cohesive properties, and no bleeding. This was achieved by the use of superplasticizers together with a low water-cement ratio and a reasonable cement content. The use of special cellulose or polyethylene-based additives can reduce bleeding and improve the cohesive properties of concrete. Bleeding can also be minimized by increasing the amount of fines in the concrete.

An analysis of the observed damage required that the repair mix were to be modified and applied in thin layers. As a result, supplementary tests on concrete cubes (1m x 1m x 1m) were carried out using repair mixes with and without cohesive additives. These tests revealed that the mix with the cohesive additive was not suitable for the proposed method of repair. However, a standard proprietary mix without cohesive additives, which was developed by a specialist firm, did meet the desired working requirements. Its composition and properties were as follows:

Portland cement content	900 kg/m^3 (535 lb/cy)
Water/cement ratio	0.32
Maximum aggregate size	4 mm (0.16 in.)
Additives (i.e., superplasticizer)	~ 3%
Slump	300 mm (12 in.)
Water penetration	3 mm (0.12 in.)
Compressive strength after 7 days	90 MPa (13 ksi)

8.1.6 FORMWORK

Two options were investigated for the formwork: one was based on fastening the formwork to the concrete structure by anchors glued into drilled holes (Figure 8.6), and the other was based on attaching it to the structure by suction cups (Figure 8.7).

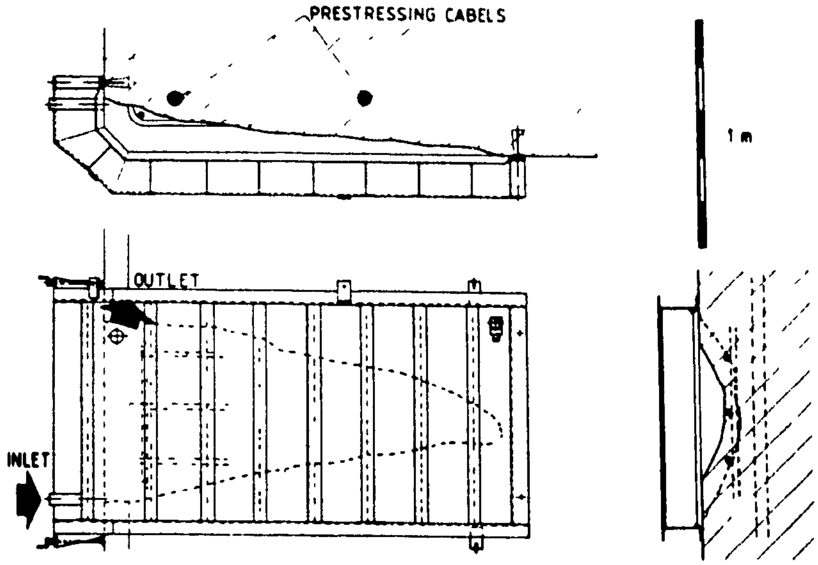


FIGURE 8.6 - FORMWORK FASTENED BY DRILLED ANCHORS

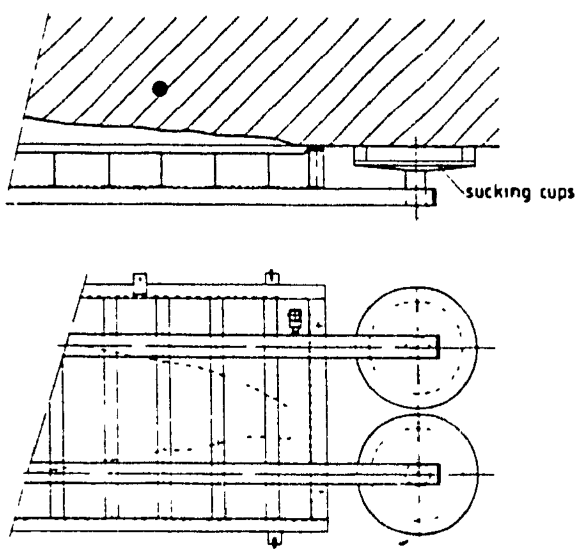


FIGURE 8.7 - FORMWORK FASTENED BY SUCTION CUPS

The first method required extra work to be carried out under water for drilling the anchor holes. More importantly, to reduce the risk of corrosion, the anchors needed to be ground off after the work was completed to minimize exposure to seawater. However, one advantage with using this method is that it is reliable and relatively simple to execute, provided the anchors are installed properly.

The second alternative was developed by a firm (F Nooren, Stadskanaal Holland) specializing in underwater injection repairs. However, due to the size and location of the damage combined with difficult underwater conditions, this system would have been very difficult to install. As a result, the first alternative was selected.

8.1.7 AUXILIARY CONSTRUCTION WORKS

It was determined that the wave and tidal activity at the site would have an unfavorable effect on diver operations. Due to the location of the damage and the shape of the structure, the water flow around the pier is turbulent and could reach velocities of up to 3 to 4 m/sec (10 to 24 ft/sec.). Therefore, for diver safety and to obtain a good quality repair, it was determined that work would only be carried out when water velocities were 0.5 m/sec (1.6 ft/sec.) or less. An analysis of the water velocity with respect to the tidal activity showed that the available working time during each tide cycle would be about two hours.

To reduce water flow velocities and increase the diver working time, the idea of installing wave deflectors at the place of repair was investigated. Initial investigations with a flow screen, shown in Figure 8 8, showed that this structure did not reduce flow velocity sufficiently. Further hydraulic studies lead to the development of a structure which screened off the repair area on all sides. Adjacent to the screen, a special shaft was designed for diver access. This led to a 'shoe type' enclosure, shown in Figure 8.9.

The enclosure was fixed to the pier by forcing out hydraulic jacks in the recess of the pier shaft. To prevent hydraulic forces on the enclosure from becoming excessive, the screen was perforated. The enclosure was also provided with a number of working platforms for the divers. It appeared that with this structure, flow velocities remained below 0.5 m/sec (1.6 ft/sec) during the full tide cycle (Figure 8.10). This permitted diver work to proceed safely without interruption.

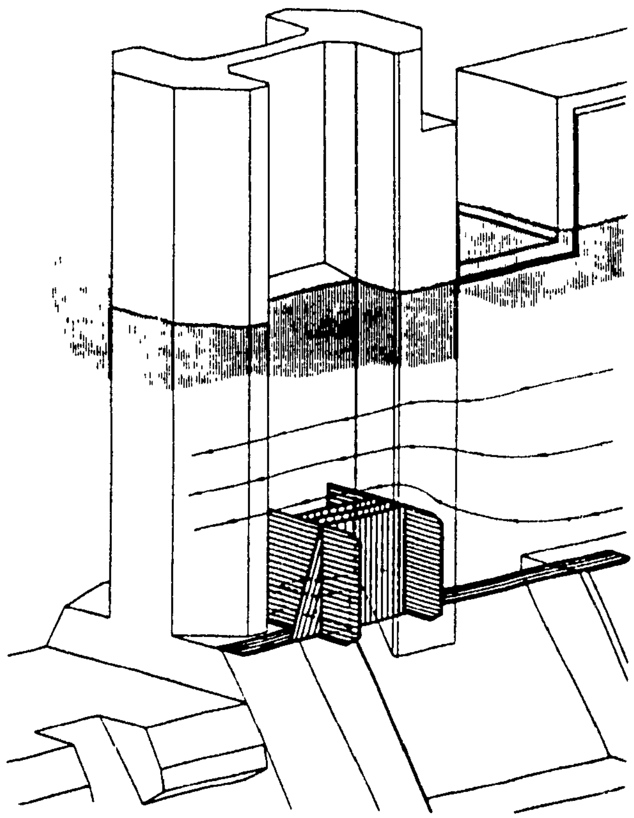


FIGURE 8.8 - OPEN FLOW SCREEN CONSTRUCTION

8.1.8 REPAIR PROCEDURE

The procedure used for repairing the five damaged concrete piers of the storm surge barrier is shown schematically in Figure 8.11. The procedure required an average of 72 hours and consisted of the following:

- Placing the auxiliary enclosure
- Drilling bolt holes for cramping the formwork
- Drilling anchor holes in the interfaces to obtain a better bond between the repair concrete and pier structure
- Gluing the anchors in drilled holes

- Gritblasting the damaged surface
- Installing the formwork
- Flushing the space behind the formwork with fresh water
- Pumping the concrete repair mix through an inlet tube in the formwork
- Hardening of the concrete for 12 to 18 hours
- Removing the formwork
- Visual inspection of the repaired surface
- Removing the bolts for cramping
- Removing the auxiliary enclosure

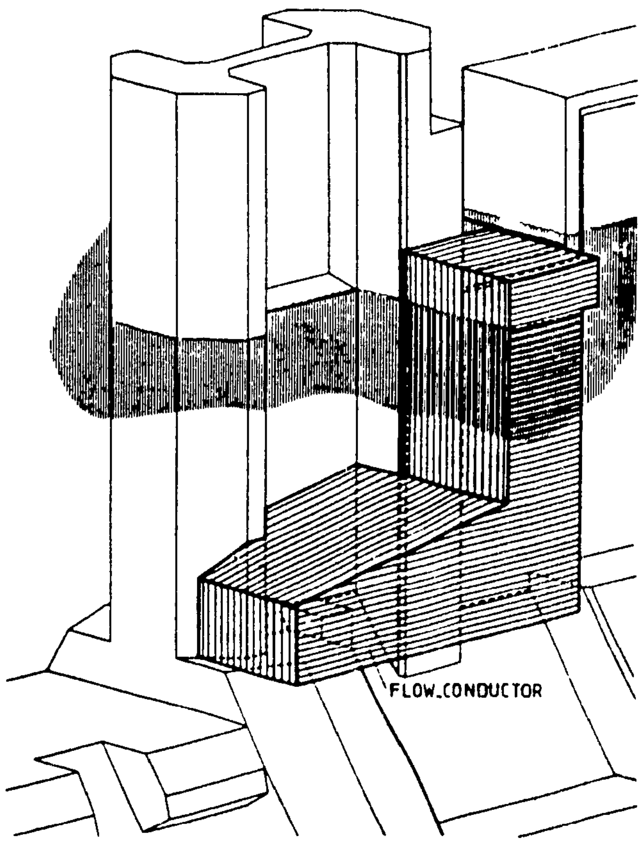


FIGURE 8 9 - HALF-CLOSED FLOW SCREEN ENCLOSURE

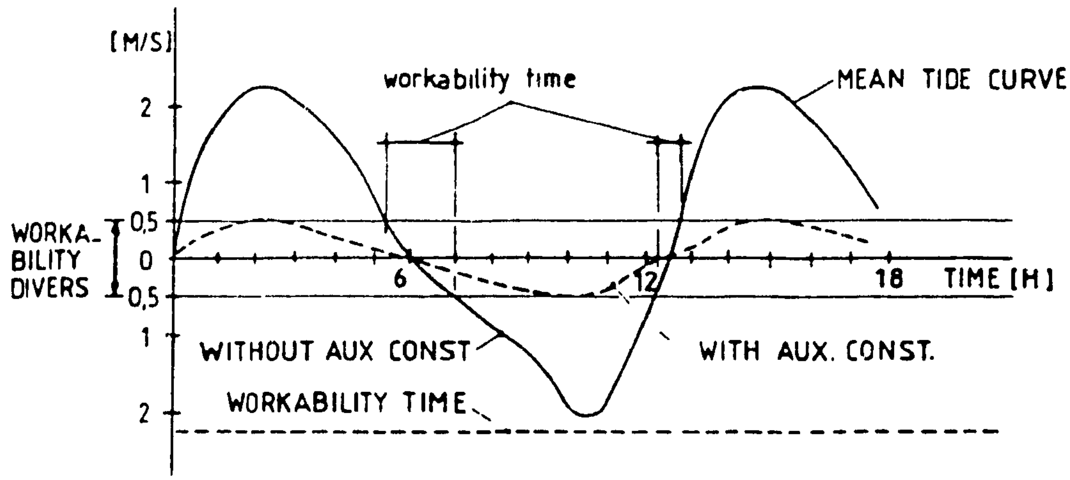


FIGURE 8.10 - INFLUENCE OF AUXILIARY CONSTRUCTION ON DIVER WORKING TIME

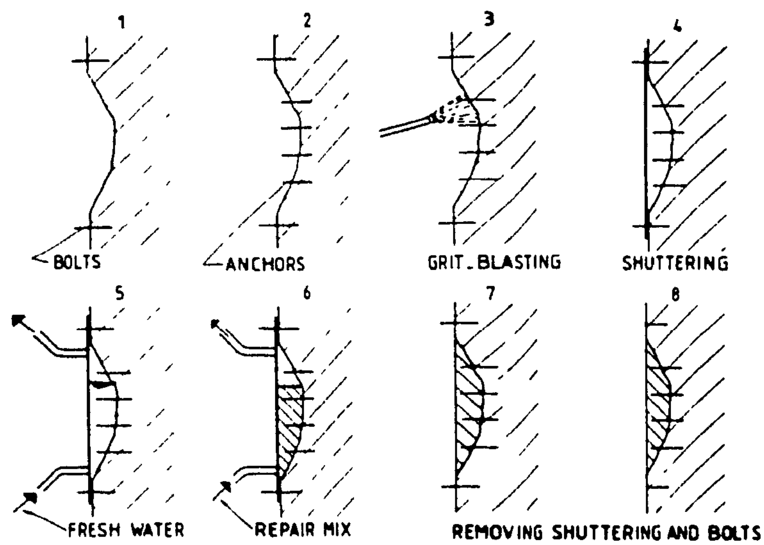


FIGURE 8.11 - UNDERWATER REPAIR PROCEDURE

8.2 CRACK REPAIRS TO UPPER STILLWATER DAM[2]
Utah, The United States

8.2.1 SUMMARY

This case study summarizes the procedure used for repairing a cracked concrete gravity dam by polyurethane resin injection methods. The materials and equipment used for both underwater and above water repairs are presented along with the results of the repairs.

8.2.2 BACKGROUND

Upper Stillwater Dam is located 72 km (45 miles) north of Duchesne, Utah, and was constructed from roller compacted concrete (RCC) and completed in 1987. For ease of construction, the dam was designed without contraction joints and it was anticipated that cracks requiring repair would appear.

In June 1988, when the reservoir was first filled, a continuous crack developed in the foundation gallery at station 25+20. The crack was also observed on the upstream and downstream faces of the dam at approximately station 25+15 As the water level in the reservoir increased, the crack widened and produced excessive leakage into the foundation gallery and out of the downstream face (Figure 8 12). At maximum reservoir level, the crack width measured approximately 6 6 mm (0.26 in.). An estimated 5 m^3/min (1,300 gpm) of water was leaking into the gallery and about 6.5 m^3/min (1,800 gpm) were leaking from the crack on the downstream face.

The crack extended from the foundation to the crest of the dam from the upstream face to the downstream face. Similar, but smaller cracks were observed at stations 30+90, 41+10, and 42+87. The cracking is believed to have been caused by foundation deformation and concrete cooling. When the reservoir level was lowered, seepage from both the gallery and the downstream face decreased and the crack measured approximately 3 mm (0.1 in.) wide

8.2.3 SELECTED REPAIR PROCEDURE

Since the cracks where expected to move with seasonal reservoir level fluctuations, a flexible hydrophilic polyurethane resin grout was selected for crack injection The Bureau of Reclamation opted to inject the cracks in three basic stages. All of the first and second stage work was performed from inside the foundation gallery and from the downstream face of the dam at elevations below the water level The final stage of the repair was performed from the upstream face of the dam Each stage is summarized in the following sections

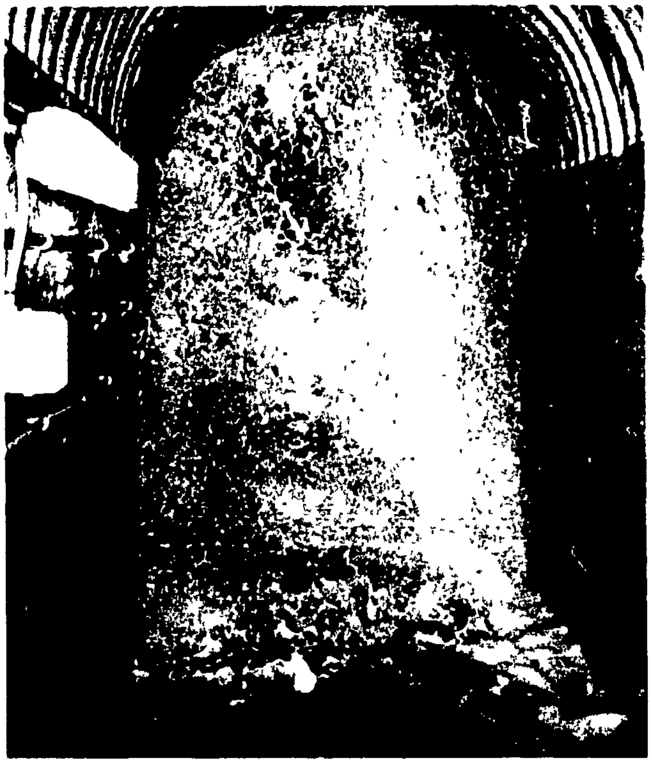


FIGURE 8.12 - WATER LEAKING INTO FOUNDATION GALLERY AT FULL RESERVOIR, STATION 25+20

8.2.4 FIRST STAGE

The main purpose of the first stage was to control or cut off seepage into the foundation gallery. (Figure 8 13) A series of shallow grout holes were drilled to intercept the crack from the foundation walls in an offset pattern. The holes were 16 mm (⅝ in.) in diameter and intercepted the crack at depths of 0.3 to 0.9 m (1 to 3 ft.). After the hole drilling operation was completed, valved injectors or "wall spears" were placed in the holes. The valves were left open to relieve

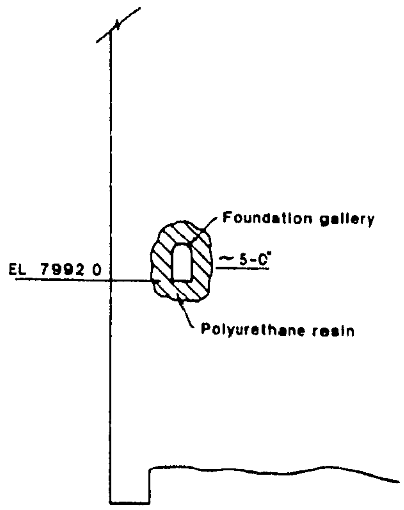


FIGURE 8.13 - FIRST STAGE INJECTION AT STATION 25+20

water pressure in the crack (Figure 8.14).

The surface of the crack was then sealed between wall spears with wood wedges, lead wool, or urethane-soaked jute rope or oakum. Once the flow of water was controlled, the wall spears were individually connected to the urethane resin pump system and injected with resin. After injection, the wall spears were removed and check holes, or water leakage from the crack, were used to determine if reinjection was required.

8.2.5 SECOND STAGE

The second stage repair work involved injecting the crack from the upstream face to the downstream face at elevations below the water level (Figure 8.15).

The crack intercept holes were drilled with a self-powered hydraulic rotary drill with 50 mm (2 in.) diameter (Ax) core bits. The holes varied in depth from 6 to 28 m (19 to 92 ft.). Once the hole depth reached a crack plane, as indicated by drill water loss, a pneumatic packer was placed a few meters above the hole-crack intercept and the hole was injected with resin. To reduce the flow of water during drilling and injection of interior holes (A-line, B-line, C-line and E-line), the D-line holes intercepting the crack 1 5 m (5 ft.) from the downstream face of the dam were drilled first

Water-to-resin ratios varied from 0:1 (neat resin) to 2:1, although most injections were done at a 1:1 ratio The pressure at which resin could no longer be injected into the crack varied from 4 to 8 MPa (600 to 1,200 psi) Check holes were drilled to determine if the crack had been adequately sealed with grout If the crack was not fully sealed, additional holes were drilled and injected with resin.

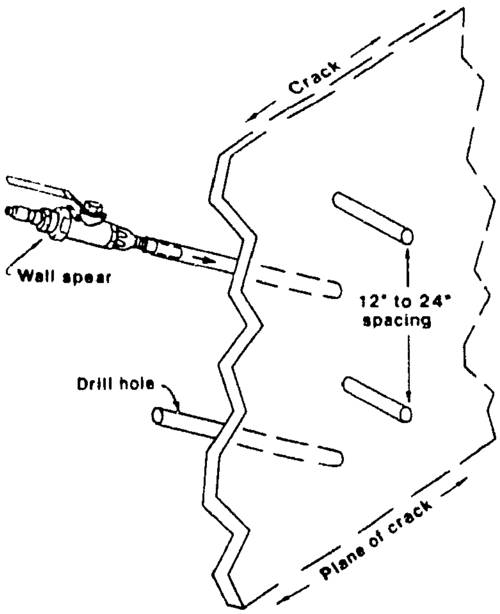


FIGURE 8.14 - GALLERY WALL-CRACK INTERCEPT DRILL PLAN

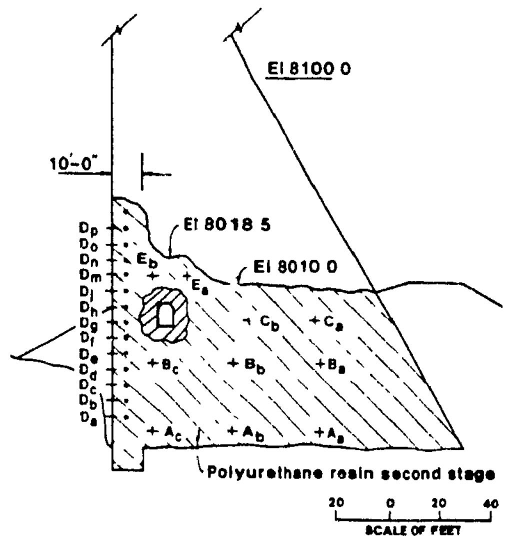


FIGURE 8.15 - SECOND STAGE INJECTION AT STATION 25+20

8.2.6 THIRD STAGE

The final stage consisted of injecting the crack above the water level from the upstream face of the dam using a "spider platform" suspended from the top of the dam (Figure 8.16). Various 16 mm (5/8 in.) diameter injection holes were drilled in a staggered pattern on each side of the crack and angled to intercept the crack at about 0 6 m (2 ft) from the upstream face Hole spacing varied between 300 to 600 mm (12 to 24 in.) apart. Wall spears were installed and the holes were injected with polyurethane resin Injection was started at the lowest hole and progressed upward to the top of the crack or dam

The amount of resin volume was proportioned to allow injection of the outer 3 m (10 ft) of the upstream face of the dam (Figure 8 17) Since the work for this stage was performed at a location where the crack was dry, a water-to-resin ratio of at least 1.1 was required As with the previous stages, check holes were drilled to determine injection efficiency

The cracks at the other locations were repaired using a similar procedure, however these cracks were not as wide and did not experience a high hydraulic head. Therefore, the injection zone for these locations was smaller (Figure 8.18).

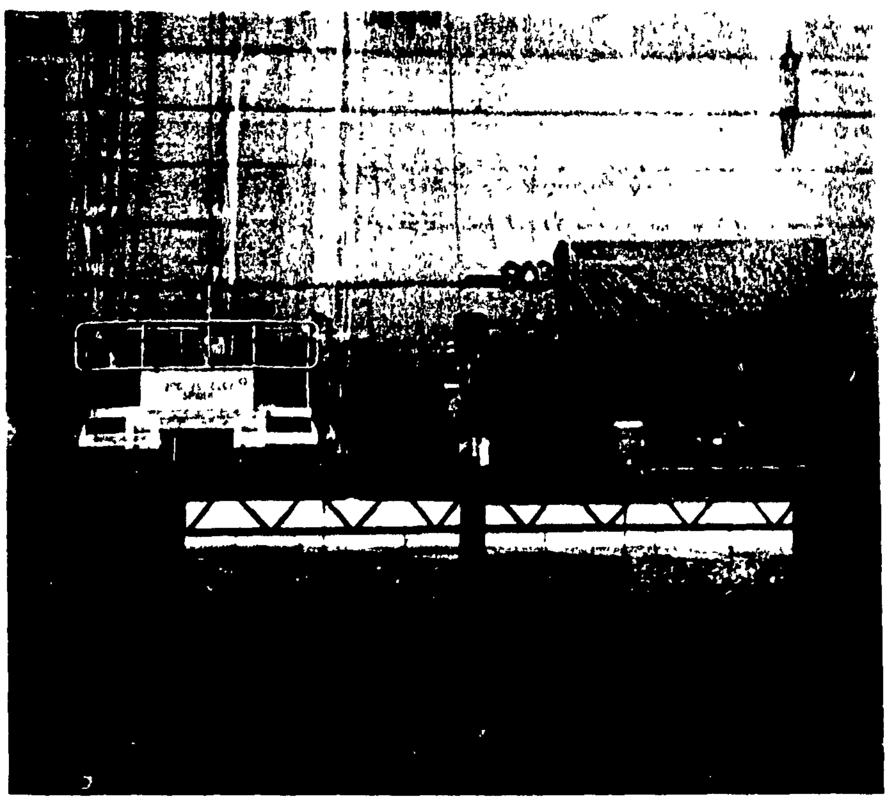


FIGURE 8.16 - "SPIDER PLATFORM" AND BARGE USED TO INJECT THE UPSTREAM FACE

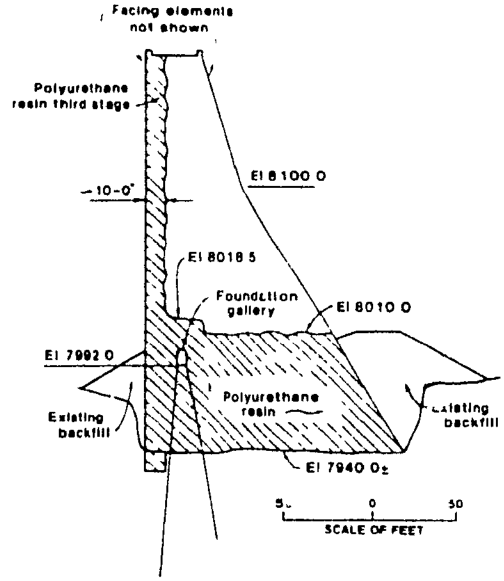


FIGURE 8.17 - THIRD STAGE INJECTION STATION 25+20

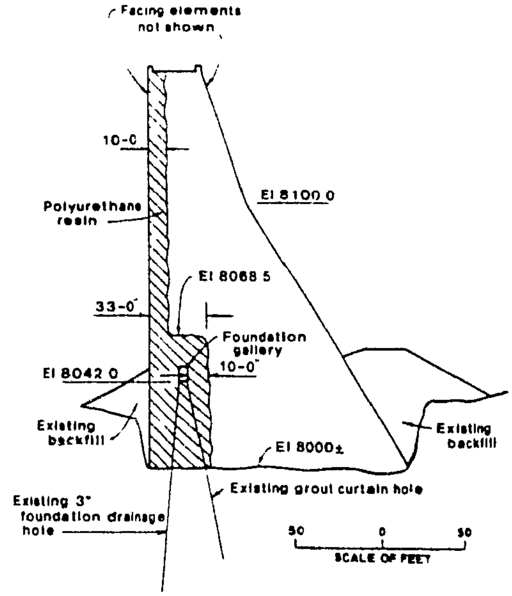


FIGURE 8.18 - ZONE OF RESIN AT INJECTION AT STATION 41+10

8.2.7 REPAIR RESULTS

Before the repairs were implemented, the combined seepage through the foundation gallery and from the downstream face at station 25+20, was more than 12 m^3/min (3,100 gpm) at maximum reservoir level. The combined seepage through the cracks at the other locations was approximately 3.8 m^3/min (100 gpm). After injection, the leakage at these cracks was minimal. The leakage through the crack at station 25+20 was reduced to below 3 m^3/min (800 gpm).

8.3 REHABILITATION OF RAILWAY BRIDGE PIERS IN A MARINE ENVIRONMENT[3] Québec, Canada

8.3.1 SUMMARY

This case study summarizes the various procedures used for repairing concrete railway bridge piers in a marine environment on the St. Lawrence River in the Beauharnois Valley, located south-west of Montreal, which were damaged by severe alkali-aggregate reactivity. The various test procedures used for determining concrete quality are also presented. Rehabilitation of all the 45 heavily deteriorated concrete piers supporting this one kilometer long railway bridge was completed in one working season

8.3.2 BACKGROUND INFORMATION

Sandstone aggregates of the Potsdam group are susceptible to alkali-aggregate reactivity when used in making concrete, especially so in a marine environment where there is a continuous source of moisture. This type of stone is not commonly used as aggregate due to its hardness and abrasiveness. Nevertheless, it has been used with ordinary alkali-rich cement when made readily available from important excavation sites Such was the case when the railway bridge was built in the Beauharnois - Valleyfield area located southwest of Montreal. The bridge was built in dry conditions prior to the subsequent excavation of a water canal to bring water flow from the St. Lawrence to a Hydro-Québec powerhouse. A total of 45 concrete piers are located in the canal waterway.

The railway bridge is almost one kilometer (3300 ft.) long and consists of 44, 22 m (72 ft.) spans built with two steel plate girders (Figure 8 19). Each girder is simply supported on 100 mm (4 in.) thick bearing plates that rest on concrete piers founded on a Potsdam sandstone The height of the piers (from top to the bedrock) varies between 10.5 and 17.4 m (34.4. and 57 ft.) with an average height of 14 m (46 ft.) below the water surface. The pier's rectangular cross section measures 1.8 m (6 ft.) by 3.6 m (12 ft.).

8.3.3 OBSERVED DAMAGE TO CONCRETE PIERS

All the above water portions of the piers showed severe damage due to alkali-silica reaction which caused polygonal surface cracking on map cracking (Figure 8.20). Fracturing was worsened by subsequent freeze-thaw cycles Since in many cases the bearings were not functional, thermal movements caused additional cracking During the winter season the crack width opened up to 35 mm (1⅜ in) A detailed inspection revealed that some bearings had settled into the concrete pier tops approximately 20 mm (¾ in). Subsequent coring determined that the concrete possessed a low bearing capacity and, as a result, the bridge was closed

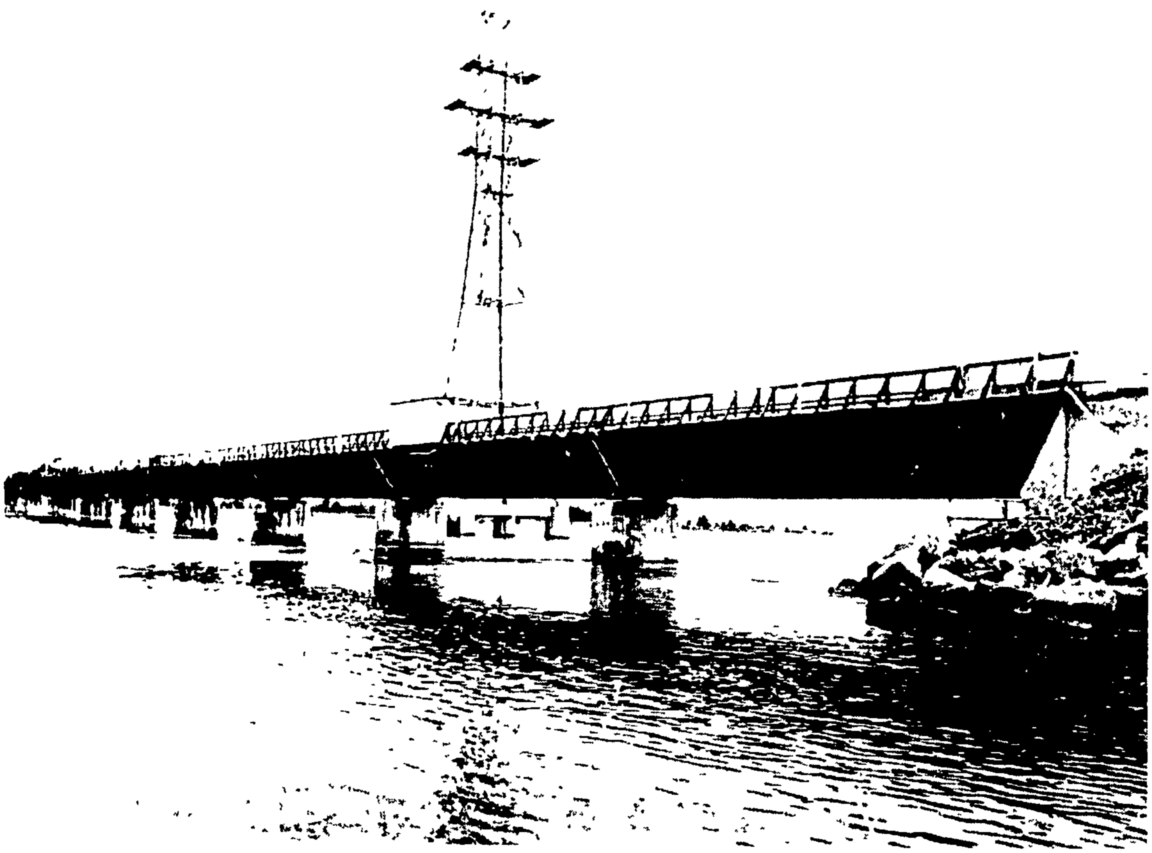


FIGURE 8.19 - GENERAL VIEW OF BRIDGE AFTER REPAIRS

8.3.4 REPAIR ALTERNATIVES INVESTIGATED

The following rehabilitation schemes were considered for restoring the structural capacity of the bridge:

- Building a new bridge with only ten piers and a new steel superstructure
- Demolishing and repairing part of the concrete piers, requiring the removal of all girders
- Temporarily shoring the spans by means of specially designed supports, while damaged concrete was removed and replaced

For economic reasons, the third option was selected. Various methods for shoring the girders were analyzed. The method which was selected is shown in Figure 8.21. Four holes were drilled into the pier down to sound concrete, into which heavy inserts were grouted. The inserts extended above the pier top and served as temporary shoring supports during repairs. The following sections describe the various test procedures that were used to determine concrete quality for verifying the repair scheme.

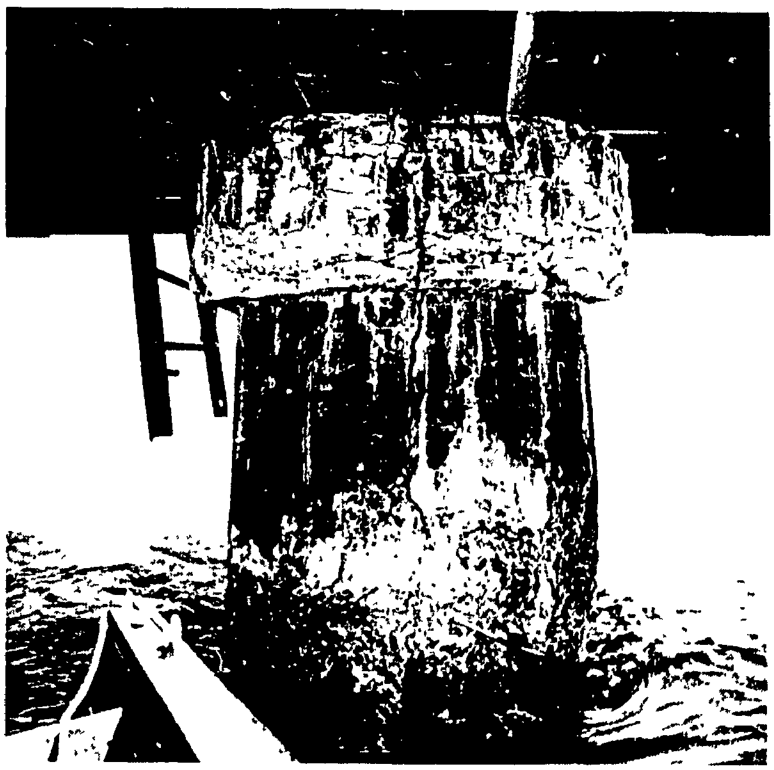


FIGURE 8.20 - TYPICAL PIER DAMAGE

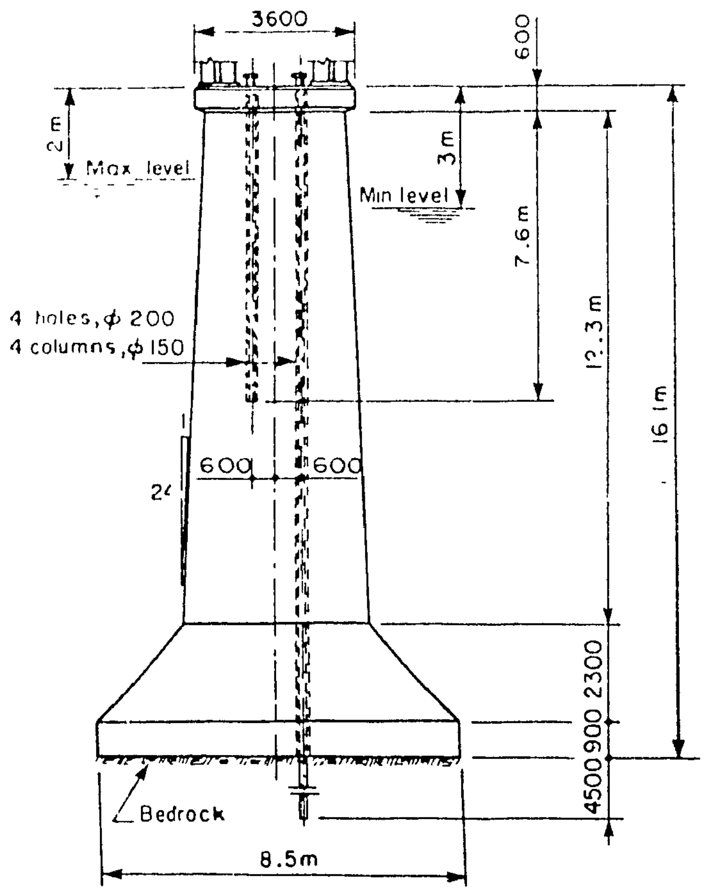


FIGURE 8 21 - PROPOSED LOAD TRANSFER COLUMNS

8.3.5 CONCRETE QUALITY INVESTIGATION

Field and laboratory tests were conducted to obtain the concrete strength and to determine the anchorage length of the steel inserts required to safely carry the bridge loadings Laboratory tests included petrographic examination of coarse and fine aggregate, and compression and splitting tensile tests on 52 cores. The modulus of elasticity was measured on 16 cores.

Large diameter (150 mm/6 in.) cores were obtained to determine concrete quality. The diameter of the borehole (200 mm/8 in.) was drilled to accommodate a large enough insert, such as a H-beam, pipe or solid steel shaft. Two holes, 6 to 7.5 m (19.7 to 24.6 ft.) long were drilled in eight of the 45 concrete piers Three of the 16 holes were drilled down to 15 m (49 2 ft) below the top of the piers. Two of the 16 holes were drilled through the full depth of the pier down into bedrock to check the concrete-rock interface characteristics This revealed that the piers had not been anchored into the bedrock Loss of drilling water occurred at two piers during the drilling operation, indicating that there was a discontinuity between the pier and bedrock The subsurface bedrock was often composed of a superficial rock crust followed by friable sandstone or pockets filled with gravel Based on these results, one hole per pier was drilled to a depth such that one anchor could be grouted in the rock to a minimum depth of 1 5 m (5 ft)

8.3.6 PETROGRAPHIC ANALYSIS

Petrographic analysis revealed that the observed concrete damage was caused by the expansion of both the coarse and fine aggregates as a result of alkali-silica reactivity The aggregates were almost all derived from orthoquartzite sandstone This orthoquartzite is a sedimentary rock which consists of quartz grains having an average diameter of 0 3 mm (0 012 in) The grains were cemented by a reactive siliceous cement which accounted for the observed damage

8.3.7 CONCRETE CORE EXAMINATION

Visual examination of all concrete cores revealed that the concrete above the water level had deteriorated significantly more than the submerged portions of the concrete Concrete specimens were examined from three nominal zones the severely cracked top zone (0 to 2 m), the proposed anchorage zone (3 to 7 5 m); and the lower zone (7 5 to 16 m). The following was observed in all three zones.

- Polygonal cracking
- Alkali-silica reaction rims (coarse and fine aggregates)
- Fractured aggregates
- Presence of silica gel

8.3.8 CONCRETE CORE TEST RESULTS

Compression test results of all 52 cores varied significantly from 22 8 MPa (3306 psi) for concrete above water to 43.6 MPa (6322 psi) for concrete below water (Figure 8 22) Almost all compression test results for concrete in the upper 3 m (10 ft) of the pier were below 30 MPa (4350 psi), varying from 22.8 to 31.6 MPa (3306 to 4580 psi). Compressive strengths of the concrete cores taken below the water level were well above the 30 MPa (4350 psi) value. The mean value of a normal distribution for this group was 36.4 MPa (5278 psi), with a coefficient of variation of 9.8 percent Splitting tensile tests produced a similar variation of strength with depth, as shown in Figure 8.23. Test results for specimens taken below the water level produced an average value of 2.5 MPa (326 psi) and a coefficient of variation of 16 percent. The ratio of this average splitting tensile strength to the corresponding average compressive strength (0 07) was found to be slightly lower than the normal value (0.09) for concrete with similar characteristics.

The moduli of elasticity varied between 13,000 MPa (1.89 x 10^6 psi) and 22,800 MPa (3.31 x 10^6 psi) with an average value of 17,700 MPa (2.57 x 10^6 psi). This value was also found to be slightly lower than normal. Both the strength and elastic modulus properties depend on the type of aggregate and sandstone used in the concrete. Poisson's ratio for the specimens was observed to be 0.17.

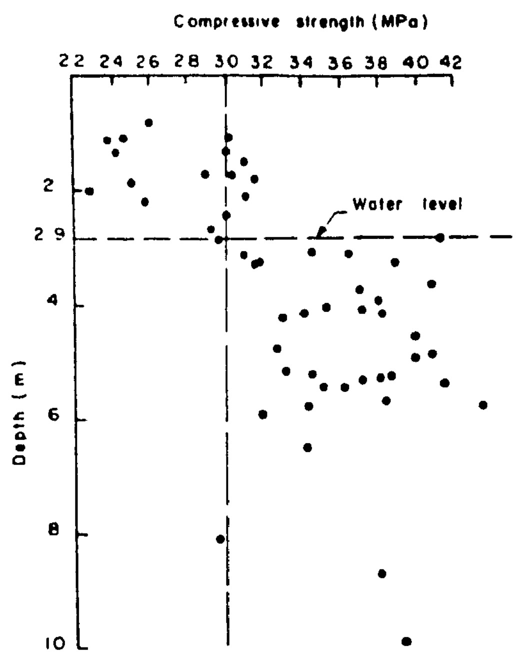


FIGURE 8.22 - VARIATION OF COMPRESSIVE STRENGTH WITH PIER DEPTH

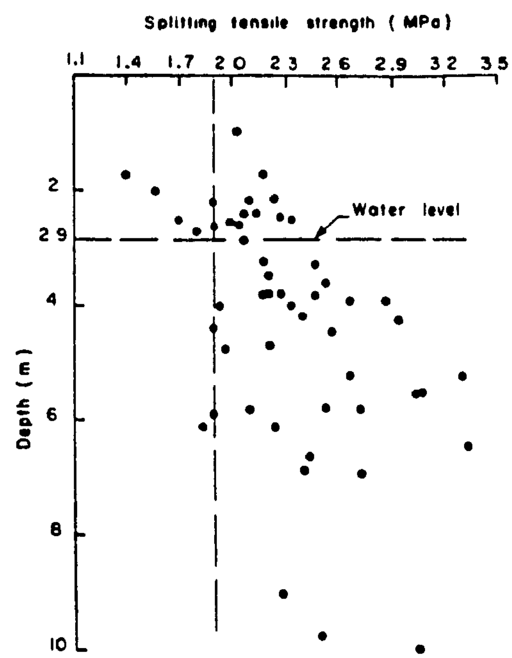


FIGURE 8.23 - VARIATION OF CONCRETE TENSILE STRENGTH WITH PIER DEPTH

8.3.9 BOND TESTS

The rehabilitation scheme that was chosen consisted of drilling and grouting 150 mm (6 in.) diameter steel axles into the pier concrete to serve as a temporary load transfer mechanism during concrete repairs. The permanent bridge loads were to be transferred, by bond, to the sound concrete layers. This required a determination of bond stresses that could be developed between the steel axles and the grout, and between the grout and the pier concrete

According to the former ACI Code (318-56), allowable bond stresses were between 3 and 4 5 percent of the concrete compressive strength Since allowable bond stresses between smooth steel rods and concrete are no longer included in concrete design codes, a small experimental testing program was initiated. The materials used for simulating the tests included 38 mm (1½ in.) diameter smooth rods, a grout mixture with a water-cement ratio of 0 4 and containing an expansive chemical, and concrete cores retrieved from the site The test set-ups are illustrated in Figure 8.24.

(a) Bond Strength of Steel to Grout

This series of tests was done to determine the bond capacity of a 38 mm (1-½ in.) diameter smooth steel pin grouted to a hollow precast concrete cylinder as shown in Figure 8.24a Four push-out tests were undertaken with the steel jacket in place and four with the steel jacket removed.

Each series of four tests produced similar results and the applied load varied between 41.6 and 52.5 kN (9.4 and 11.8 Kips). In all cases, failure occurred by sliding of the pin without cracking the surrounding concrete. At maximum load, the bond stress reportedly reached 15.4 percent of the grout compressive strength.

(b) Bond Strength of Grout to Concrete

Two series of push-out tests were performed as shown in Figure 8.24b. Here the load is applied through the concrete core specimen taken from the site. When push-out tests were conducted without the steel jacket, the grout ring cracked at one-third of the load obtained with the jacket left in place. In the latter case, the ultimate bond stress measured was 14.5 percent of the grout compressive strength.

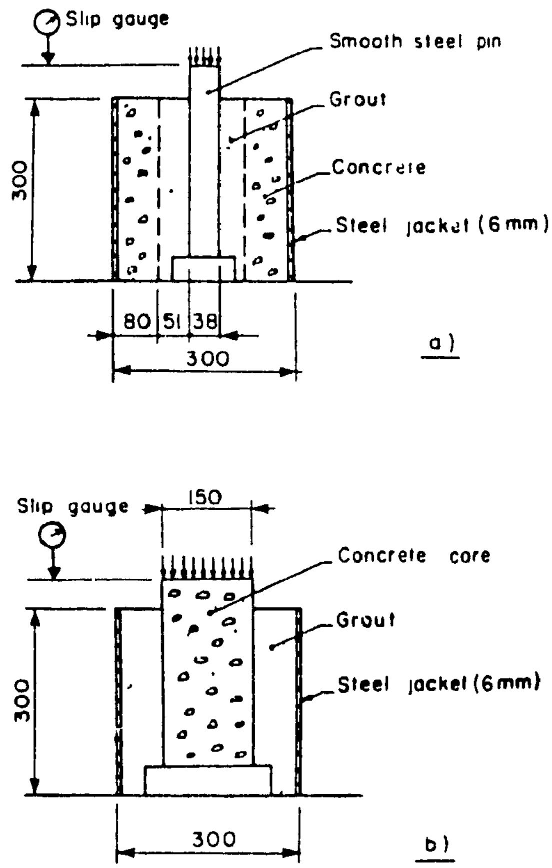


FIGURE 8.24 - PUSH-OUT TESTS: (a) BOND TEST OF STEEL TO GROUT; (b) BOND TEST OF GROUT TO CONCRETE

Since in the actual pier, the load is applied through the steel axle and an allowable bond stress equal to three percent of the expected compressive strength of the grout was anticipated. Since the compressive strength of the grout that was placed in the field was higher than anticipated, the actual bond stress was two percent of the grout compressive strength.

(c) Solution Retained

Based on the test results, it was determined that concrete quality below the water was adequate to allow the embedment of steel columns to transfer the bridge loads. Location of the steel column in the piers was dictated by several factors:

- Moments supplied by the column to resist ice loading
- Small dimensions of the pier caps
- Hole diameter required for installing the steel columns
- Columns needed to be installed as close as possible to the bridge girder to minimize the induced moment.

The two upstream columns in the pier were extended 4.6 m (15 ft.) into the bedrock to resist the moments created by ice pressure. The bond stress that was developed by the pier concrete supports 80 percent of the column load, while the remaining 20 percent is carried by bearing stresses.

8.3.10 REPAIR PROCEDURE

Drilling work was done in the fall and the winter, while the 172 steel columns and 43 permanent transfer beams were being fabricated so that repairs could be implemented during the following construction season.

The 150 mm (6 in.) diameter steel columns were positioned and grouted in the 200 mm (8 in.) diameter drilled holes. The ends of the columns were threaded on a 330 mm (13 in.) length to facilitate handling and support operations. Problems were encountered during the grouting of the anchors which extended into the bedrock. Since grout was being lost in rock seams the grouting operation was divided into two stages: an initial amount of gelatinous grout was used to plug the rock seams, and after the grout hardened, a non-bleeding grout was used to insure good bond with the steel columns.

When the grout had attained sufficient capacity, the bridge girders were lifted and set on temporary support beams which rested on threaded sleeves at the column ends (Figure 8.25). Once the temporary beams were in place, concrete was removed to a depth of 1.5 to 1.8 m (5 to 6 ft.) to allow for sufficient space to install the lifting mechanism. The lifting mechanism was fixed to the steel columns through four split collars, as shown in Figure 8 26.

The bridge girders were lifted in increments of 200 mm (8 in.) using two hydraulic cylinders, dogging pins, and beams. Once the girders were lifted, the temporary support beams were removed and the permanent transfer load beams were installed. After the permanent load transfer beams were installed, the lifting mechanism was removed and the pier reinforcing steel and formwork was installed (Figure 8.27) Concreting of the piers was done with ready-mix concrete trucks from atop the railroad bridge. Since the steel columns were designed to carry the railroad loads, ready-mix concrete trucks could easily be supported This precluded the use of barges to deliver the concrete, resulting in considerable cost savings.

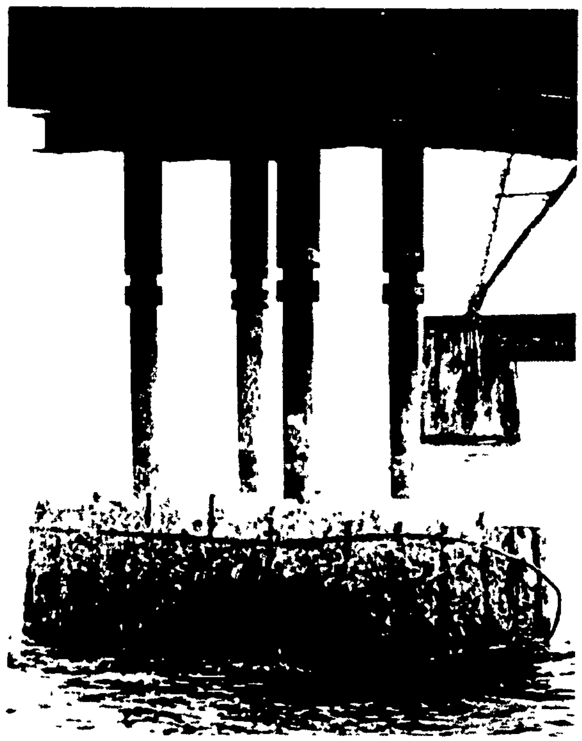


FIGURE 8.25 - BRIDGE SPAN ON TEMPORARY BEAMS SUPPORTED BY STEEL COLUMNS

8.3.11 CONCLUSION

The rehabilitation of the 45 concrete railroad bridge piers was successfully completed in one working season, using only five lifting mechanisms. All field procedures were designed to be completed in one working day. The rehabilitation project was completed at a construction cost of $ 6M (Can), which was considerably less than the cost ($ 60M) of a new bridge.

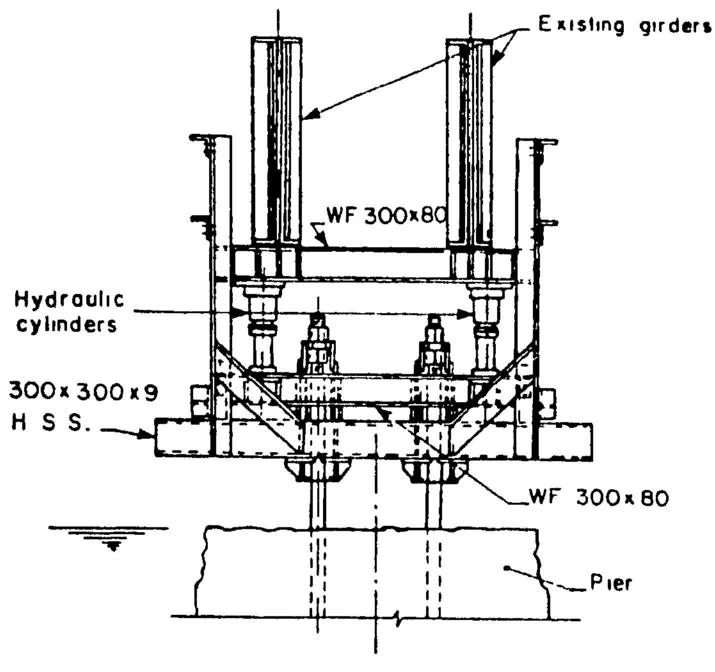


FIGURE 8.26 - LIFTING MECHANISM SUPPORTED ON STEEL COLUMNS

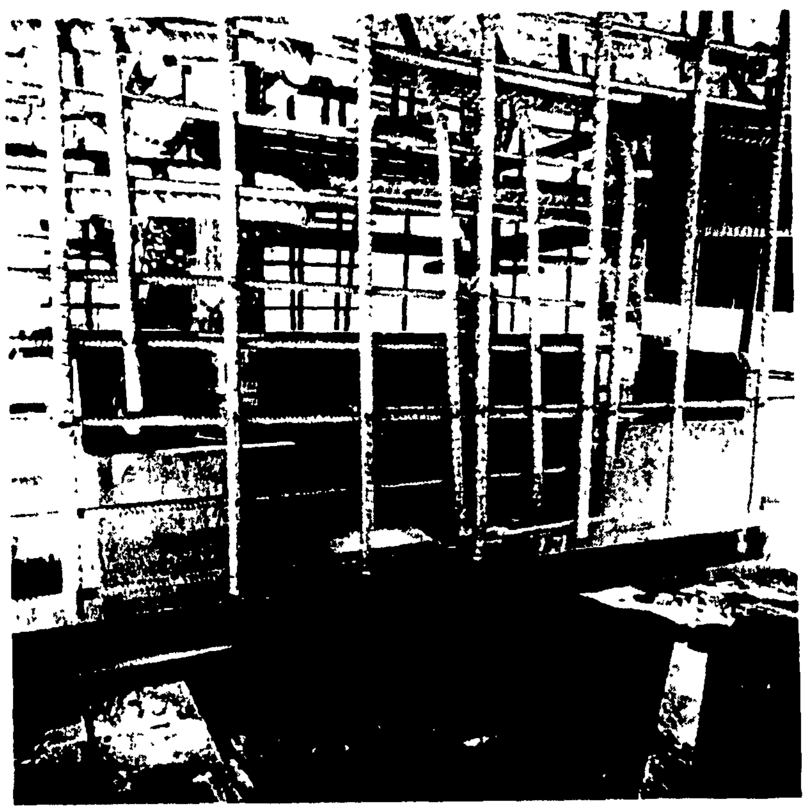


FIGURE 8.27 - REINFORCING CAGE

8.4 UNDERWATER REPAIRS AND INJECTION OF BRIDGE PIERS
Champlain Bridge, Montreal, Canada

8.4.1 SUMMARY

This case study summarizes the procedures used for repairing underwater cracks to six of the forty-four concrete piers supporting the prestressed concrete girders on the north side of the Champlain Bridge in Montreal. Laboratory tests of concrete specimens concluded that alkali-reactivity is one of the causes responsible for the cracking The various inspection and testing techniques that were used to determine concrete quality are presented along with some of the results that were obtained. Repairs to the underwater portion of all of the six piers were completed in one working season. This case study is based on the author's observations during site visits and information provided by the Jacques Cartier and Champlain Bridges Incorporated.

8.4.2 BACKGROUND INFORMATION

The Champlain Bridge was constructed between 1958 and 1962 at a cost of \$35M (Can.). The bridge provides a vital link between the Island of Montreal and the Southshore communities via a large network of approaches, including many satellite structures, such as small bridges, viaducts, and ramps.

The six lane, 3.5 km (2 miles) long bridge consists of 57 spans supported on a total of 56 concrete piers extending to the bedrock. The north side of the bridge, which is located over the St. Lawrence River, consists of simple span prestressed concrete girders. The main span, which is partially located over the St. Lawrence Seaway, is carried on a steel truss structure with a central span which is supported by a cantilevered steel truss superstructure. The total length of the steel superstructure is approximately 762 m (2500 ft.) and is located approximately 36.5 m (120 ft.) above the main shipping waterway. A view of the north side of the bridge, in which the repair work was done is shown in Figure 8.28.

The height of the piers (from top to bedrock) varies between 18.4 to 32.2 m (60.5 to 105.6 ft.) with an average height of 8.4 m (27.6 ft) located below the water surface. The cross-section of the pier has an elliptical shape which measures 9.6 m (31.5 ft.) in length by 3 m (10 ft.) in width. The pier footing is 10.4 m (34 ft.) in diameter and varies between 5 and 8 m (17 and 24 ft.) in depth.

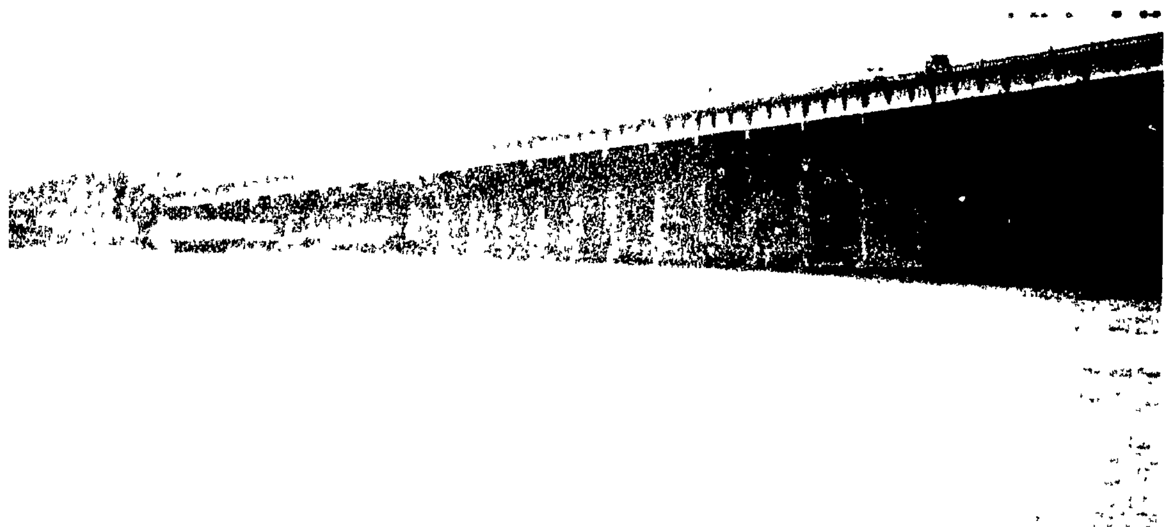


FIGURE 8 28 - VIEW OF THE NORTH SIDE OF THE CHAMPLAIN BRIDGE LOOKING NORTH

8.4.3 DAMAGE SURVEY

Underwater inspections of the footings supporting the north side of the bridge were conducted in 1988 under a separate contract as part of an on-going inspection and maintenance program. The inspections revealed that the submerged sections were severely cracked and spalled and the areas of the piers at the water line were eroded by the action of flowing water (Figure 8.29). The observed conditions were recorded on video-cassettes and the quantitative data which was obtained, such as crack width, length, depth, direction and spalled areas, was recorded on drawings

As part of the repair work performed in 1993, additional underwater inspections were made to update the damage observed in 1988 A survey to determine the depth of water around each footing (bathymetric survey) was also performed. The inspections were performed by professional divers equipped with underwater closed-circuit television cameras mounted on a helmet (Figure 8 30) The divers were in continuous communication with the surface personnel through the use of a telephone line The typical damage and underwater conditions that were observed during the inspection are shown in Figure 8 31

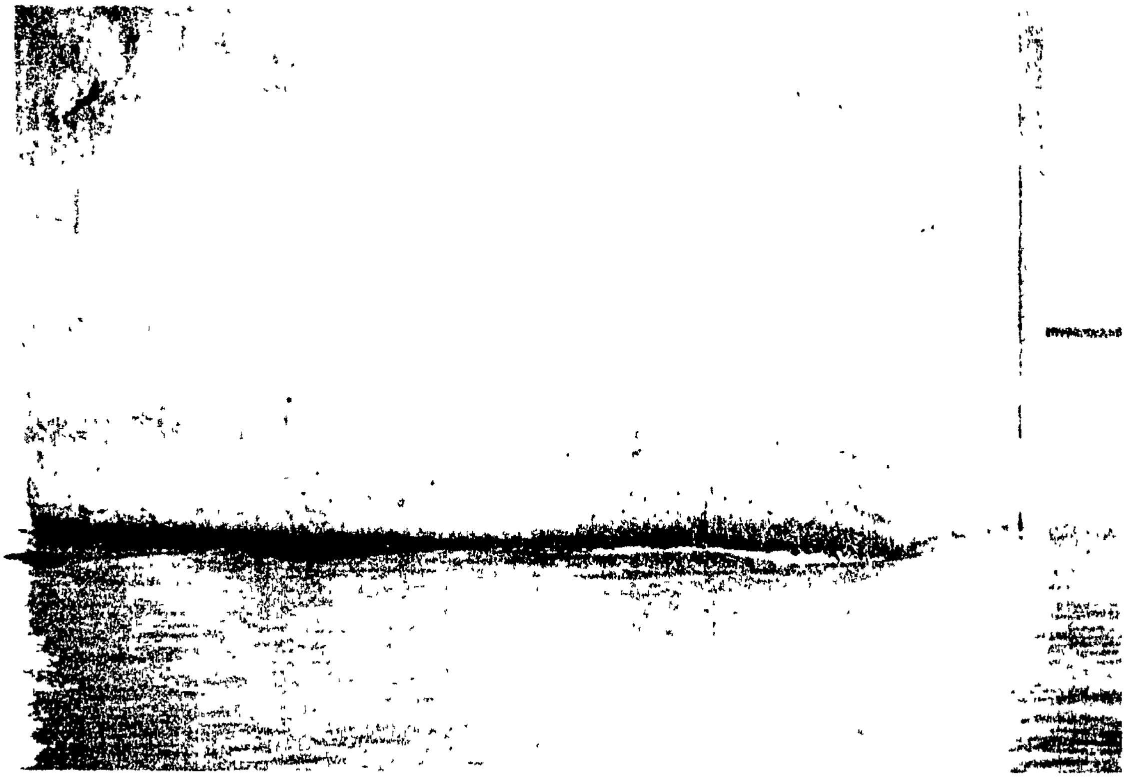
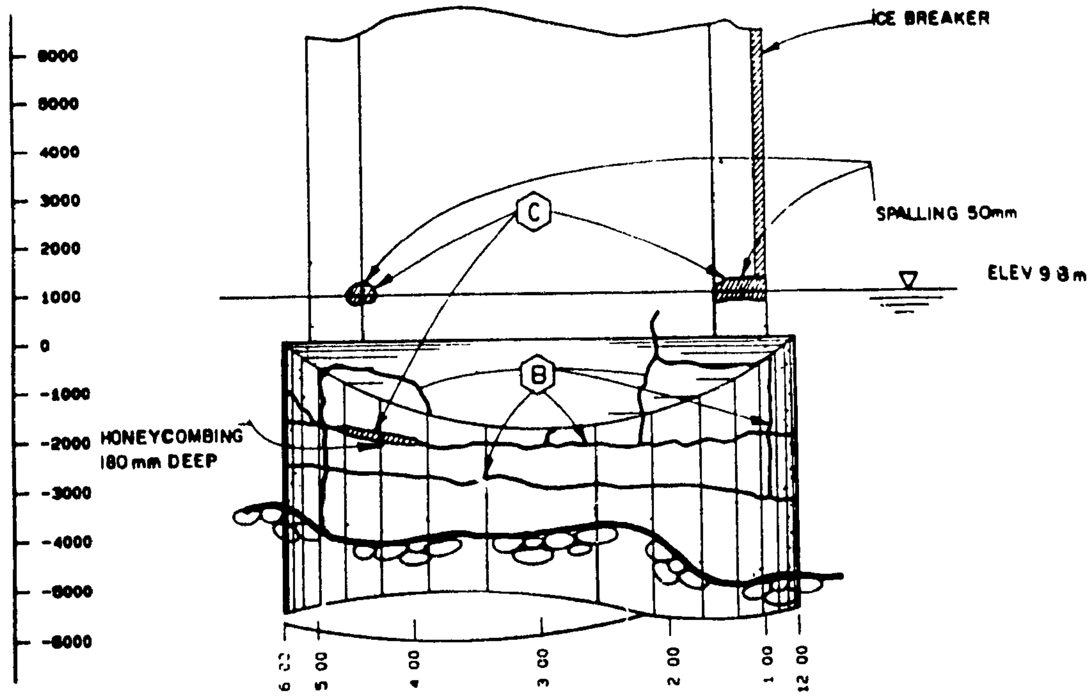
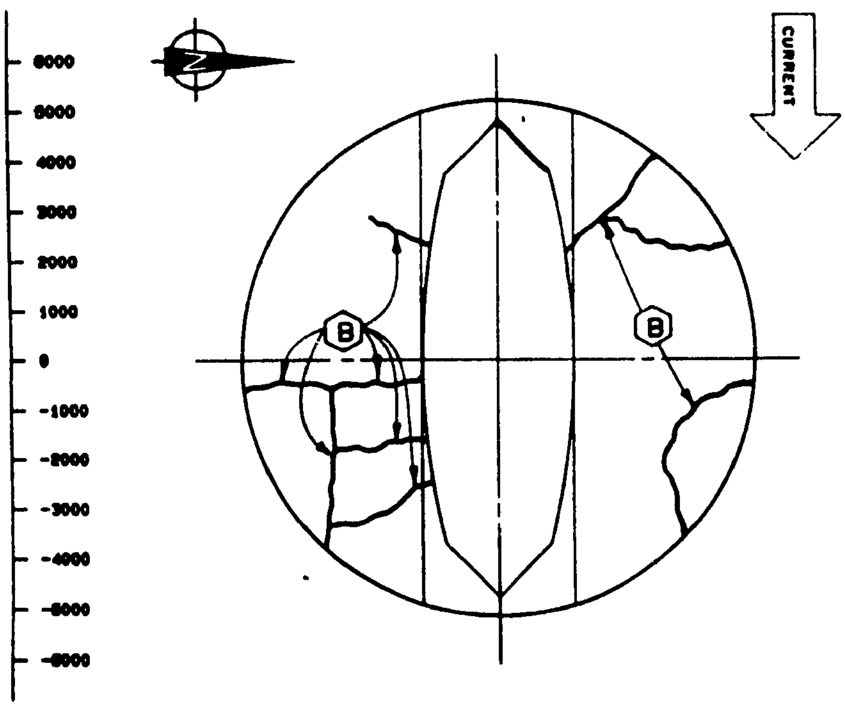


FIGURE 8.29 - EROSION OF THE CONCRETE PIER AT THE WATER LINE

FIGURE 8.30 - HEAD MOUNTED UNDERWATER TELEVISION CAMERA



NORTH ELEVATION



PLAN VIEW

PIER 32-W
PIER TYPE W-1
PLAN VIEW

FIGURE 8.31 - TYPICAL PIER DAMAGE AND DETERIORATION

8.4.4 SPECIFICATIONS FOR REPAIRS

Contract specifications and drawings were prepared and tendered based on two options the first option called for the repair of only three piers, while the second option required the repair of all six piers. Based on economic considerations, the Corporation elected the latter. The work performed for each pier generally consisted of the following:

- Mobilization/demobilization of the work barge
- Installation/removal of a current deflector
- Cleaning the concrete in the submerged zones and the tidal zones
- Performing an underwater inspection and bathymetric survey
- Concrete coring and testing program
- Injecting cracks and repairing concrete surfaces within the submerged and tidal zones
- Cleaning the site

8.4.5 SITE PREPARATION

Since the work was to be performed in areas of the St Lawrence River where current velocities could reach about 3 m/sec (10 ft/sec or 6 knots), the contractor was required to install current deflectors for the diver's safety. The deflectors consisted of metal plates attached to an array of steel framework which in turn was attached to the work barge. The barge and deflector were positioned in front of the pier, and a specially designed steel bracket which extended outward from the barge was bolted to the pier to prevent the barge from moving (Figure 8 32).

The calculations performed by the contractor verified that the forces imparted by the barge on the bridge could be safely carried by the piers. The deflector reportedly reduced the current velocity to below 0.26 m/sec (0.85 ft/sec or 0.5 knot) in the work area.

8.4.6 LIMITS OF REPAIR WORK

The repair work was to be performed in the area extending from the river bed up to 1 m (3 ft) above the high water level (HWL). Cracks which extended down in the river bed were to be repaired to a depth of 150 mm (6 in.) below the river bed. This required cleaning the cracks to a depth of 200 mm (8 in.) by removing the river bed material near the crack.

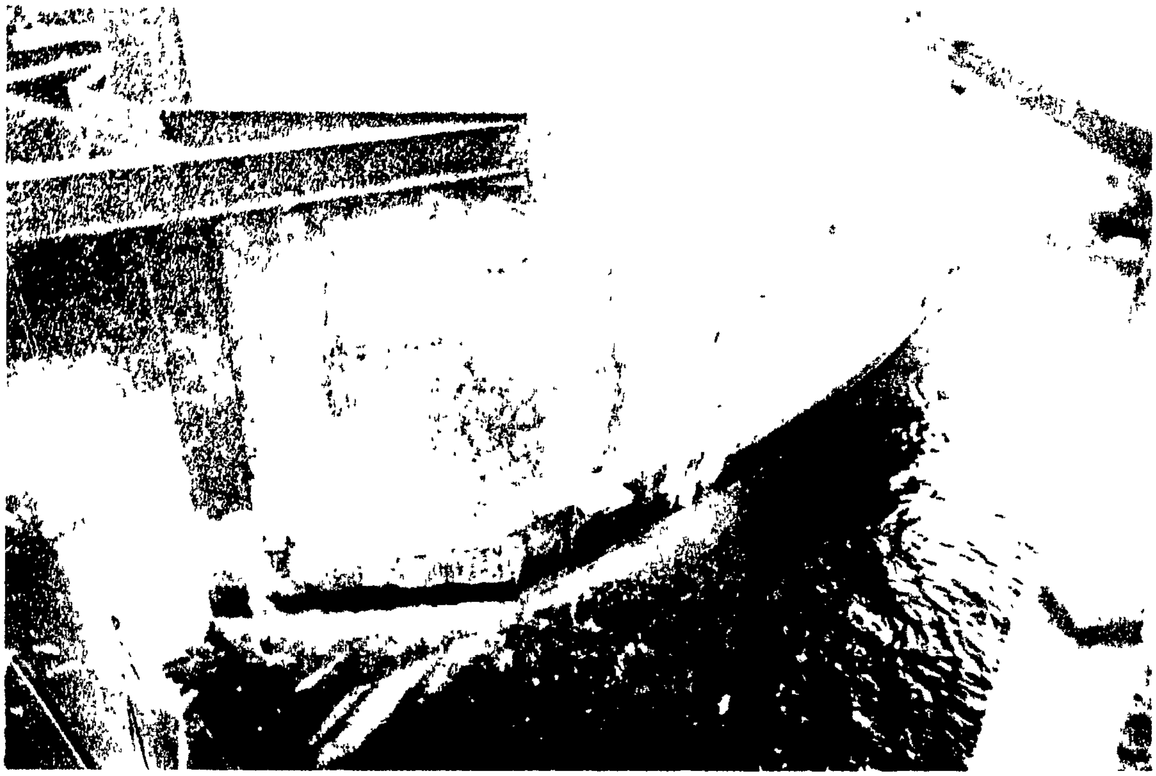


FIGURE 8.32 CONNECTION DETAIL OF THE CURRENT DEFLECTOR TO THE BRIDGE PIER

8.4.7 CLEANING AND INJECTION OF CRACKS

The piers and footings were cleaned by high-pressure water jets within the work limits to remove any marine growth or calcareous accumulations on the concrete surface and within the cracks (Figure 8.33). Once the cracks were cleaned, 50 mm (2 in.) deep injection holes were drilled at a spacing of about 400 mm (16 in.) For cracks 5 mm (3/16 in.) wide or less, the cracks were routed to a V shape which measured approximately 10 mm (3/8 in.) wide.

After the cracks were drilled, 12 mm (½ in.) diameter nylon injection tubes were inserted into the holes, and the crack lengths between the tubes were sealed with epoxy paste (SIKADUR MARINE 30) by divers using a hand trowel, as shown in Figures 8.34. The epoxy is a two component compound which was mixed using a drill motor paddle mixer immediately before applying it to the crack surface (Figure 8.35).

FIGURE 8.33 - CLEANING CRACKS WITH A HIGH PRESSURE WATER JET

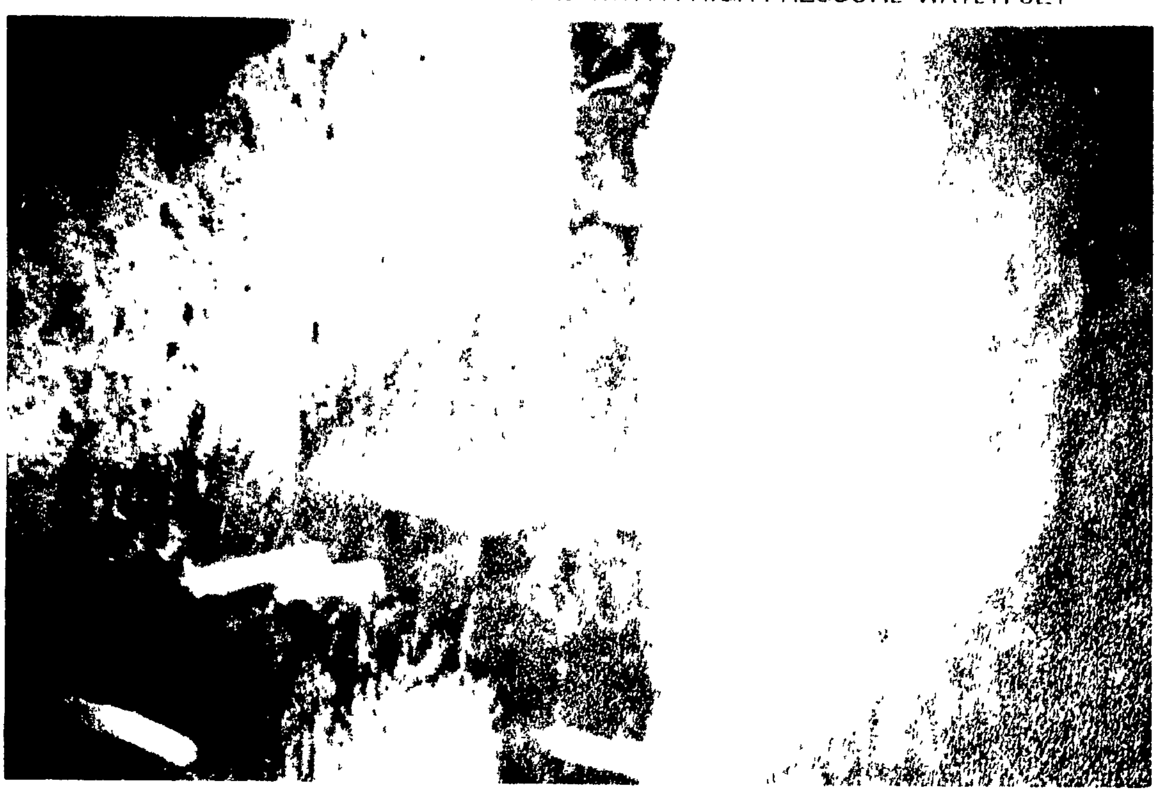


FIGURE 8.34 - NYLON GROUT INJECTION PORTS FOR REPAIRING CRACKS

FIGURE 8.35 MIXING THE EPOXY PASTE WITH A DRILL MOTOR PADDLE MIXER

Once the epoxy seal attained adequate strength to withstand injection pressures (usually about 3 hours), the cracks were flushed with water and injected with a two component epoxy resin compound Four of the piers were injected with SIKADUR 53-ST-1 and the remaining two piers were injected with CAPWELD 624 LE The epoxy was mixed and delivered with a variable ratio metered dispensing machine using a resin to hardener ratio of 3.1 (Figure 8.36). The injection procedure was started at the lowest injection port near the river bed and proceeded upward, making sure that grout emerged from the upper tubes Once this occurred, the port which was being injected was plugged with wooden dowels and injection continued at the next highest port. A typical crack injection nozzle is shown in Figure 8.37. Once injection operations were completed, all the injection tubes projecting from the repaired cracks were cut off flush with the face of the pier footing Typical repair details for wide and narrow cracks are shown in Figures 8.38 and 8.39, respectively

As part of the quality control procedures, the contractor was required to retrieve concrete cores from each pier three days or more after the completion of crack injection for visual examination.

FIGURE 8.36 - CRACK INJECTION DISPENSING MACHINE

FIGURE 8.37 - CRACK INJECTION NOZZLE AND WOODEN DOWEL TO PLUG INJECTION TUBE

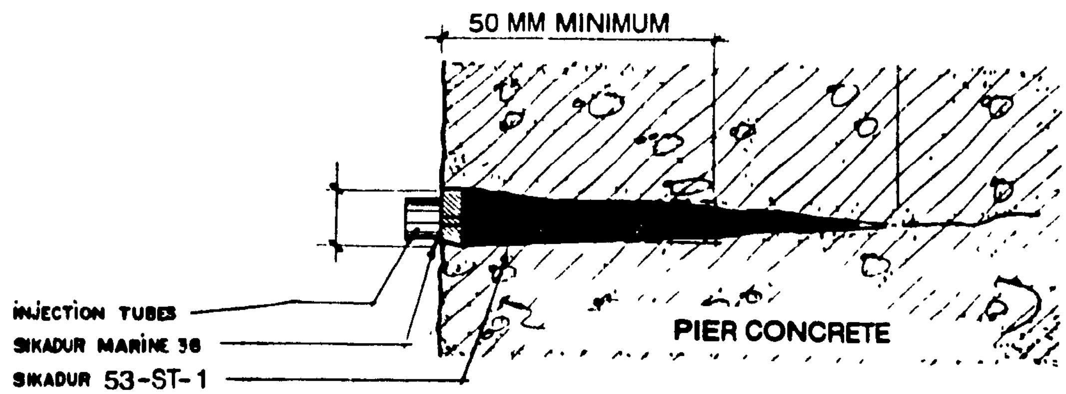


FIGURE 8.38 - REPAIR OF WIDE CRACKS

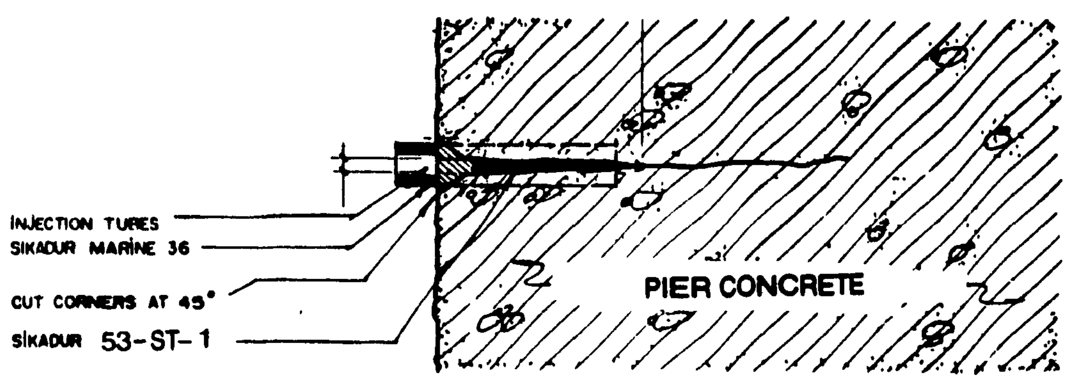


FIGURE 8.39 - REPAIR OF NARROW CRACKS

The specifications called for a minimum of two cores, plus an additional core for each supplemental 100 m (30 ft.) of cracks repaired in each pier. The cores were obtained using a 100 mm (4 in.) diameter diamond-tipped core drill which was lowered into position by a crane mounted on the work barge, and fastened to the footing with anchor bolts. The cores had an average length of approximately 300 mm (12 in.). Typical concrete cores taken before and after crack injection are shown in Figures 8.40 and 8.41, respectively. At the end of the core drilling program for each pier, the core holes were filled with an epoxy paste (SIKADUR 45).

Visual examination of all of the concrete cores revealed that the percentage of the crack filled with the epoxy varied from 50 to 100 percent with a weighted average of 83 percent. Laboratory tests showed that the adhesive strength of the epoxies varied from 0 to 0.60 MPa (87 psi) with a weighted average of 0.23 MPa (33 psi).

8.4.8 CLEANING AND REPAIR OF SPALLED CONCRETE

All spalled or honeycombed concrete surfaces were cleaned using a high-pressure water jet Honeycombed or crumbling concrete was removed with an underwater chipping hammer to a minimum depth of 30 mm (1-3/16 in.). After all the loose concrete was removed, the repair surfaces were given a final cleaning with a high-pressure water jet. Metal formwork with a "bird's mouth" shutter was fastened to the repair area with drilled expansion anchors. The perimeter of the formwork was sealed with a hand troweled epoxy paste (SIKADUR 36) to prevent the repair material from leaking out. Once the formwork was in place, the marine grout (SIKADUR 45) was poured into the formwork by gravity (Figures 8.42 and 8.43) Once the repair material hardened and the formwork was removed, any repaired concrete which projected more than 5 mm (3/16 in.) beyond the face of the pier footing was removed with chipping hammers.

8.4.9 CONCRETE INVESTIGATION PROGRAM

The concrete investigation program consisted of performing an electrical half-cell potential test at one of the piers and taking seven concrete cores at each pier for performing visual and laboratory examinations of the concrete. The half-cell potential test results indicate that there is virtually no corrosion activity within the submerged portions of the piers.

The concrete core specimens measured 100 mm (4 in.) in diameter and 300 mm (12 in.) in length. For each set of seven cores at each pier, the following laboratory tests were performed:

- Two simple compression tests
- Two tests to determine the water-soluble chloride ion content in two cores
- One test to determine the water absorption and unit weight of concrete

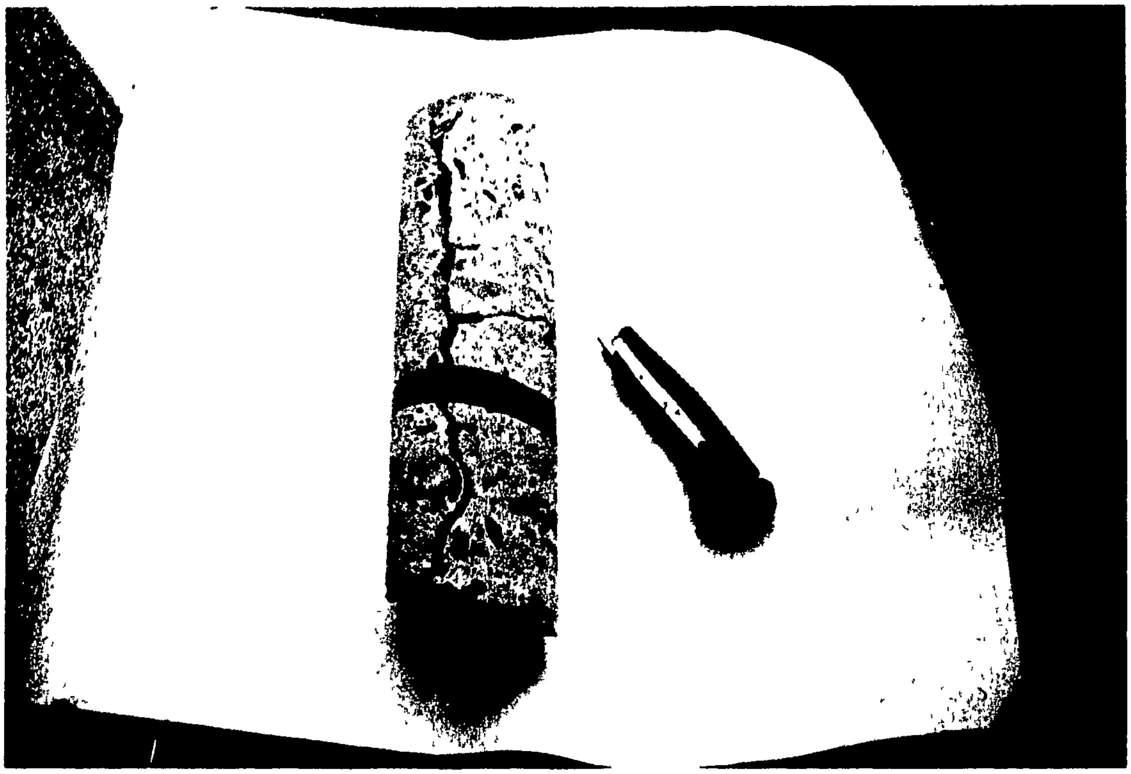


FIGURE 8.40 - CONCRETE CORE SHOWING CRACK BEFORE INJECTION

FIGURE 8.41 - CONCRETE CORE SHOWING CRACK AFTER INJECTION

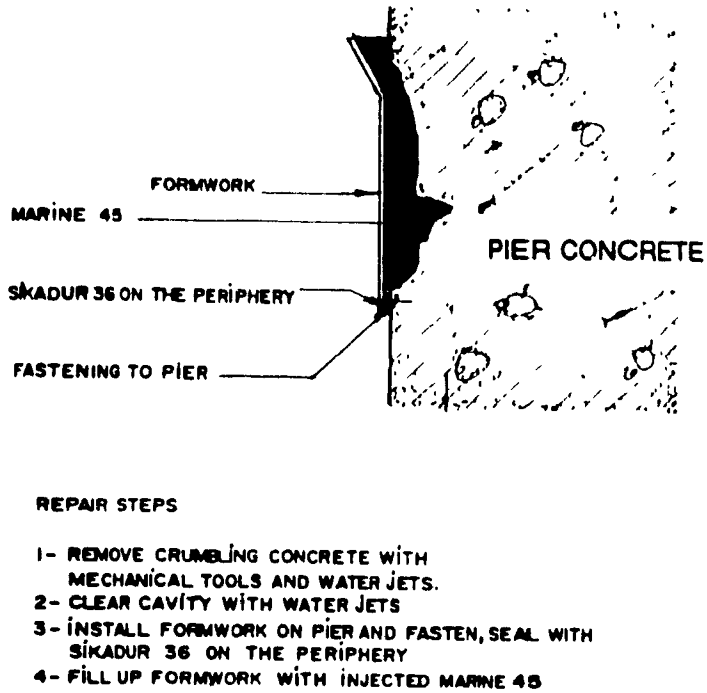


FIGURE 8.42 - REPAIR OF CRUMBLING CONCRETE

At three of the piers, the following additional laboratory tests were performed:

- Petrographic examination in accordance with ASTM C 295 procedures, including evaluation of cement content
- Determination of alkali-silica reactivity using the accelerated concrete prism method (South African Test)

Visual inspection of the concrete cores and preliminary laboratory test results indicate that alkali-reactivity is active in some of the piers. The concrete compressive strength varied from a minimum of 18 MPa (2615 psi) to a maximum of 50 MPa (7260 psi) with an average value of 34 MPa (4940 psi). The air content of the concrete varied between 0.06 and 0.14 percent with an average value of 0.10 percent. The spacing factor varied from 262 to 917 microns with an average value of 614 microns.

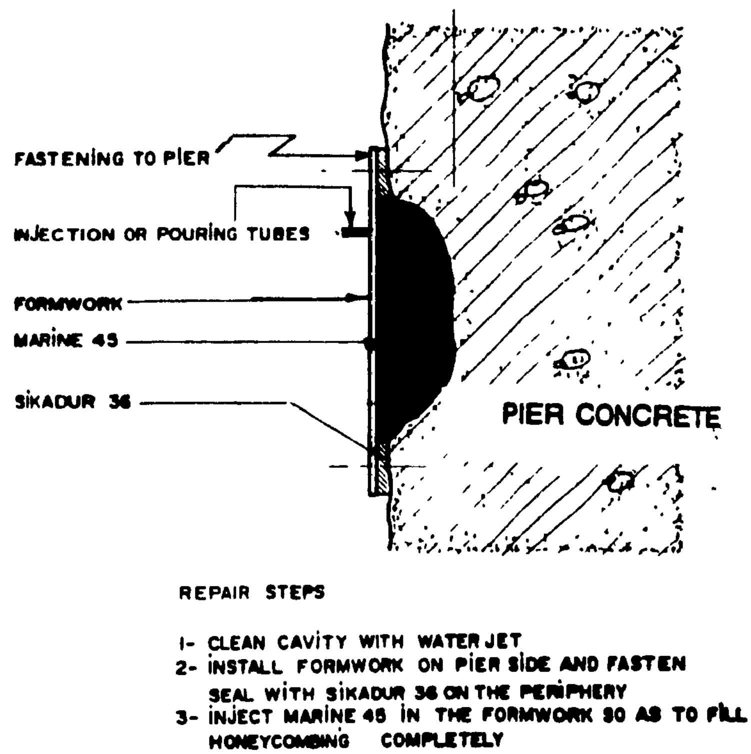


FIGURE 8.43 - REPAIR OF HONEYCOMBED AND SPALLED CONCRETE

The chloride ion concentrations at 25, 50, 75 and 100 mm (1, 2, 3 and 4 in.) were virtually all below the corrosion threshold limit and decreased significantly with increasing distance from the surface of the concrete. A chloride ion concentration of 0.20 percent by weight of cement is generally accepted as the corrosion threshold limit for reinforced concrete structures in a marine environment. For this project, the chloride ion concentrations at a 25 mm depth varied from a minimum of 0.005 percent to a maximum of 0.217 percent, with an average value of 0.059 percent. At a 50 mm depth, the minimum and maximum chloride concentrations were 0.16 and 0.06 percent, respectively, with an average value of 0.038 percent. The average values of the chloride ion concentration at 75 and 100 mm were 0.034 percent and 0.013 percent, respectively.

8.4.10 CONCLUSIONS

The repair of the six piers was completed in one working season during the period between June and October of 1993. A total of 3548 linear meters (11,635 linear feet) of crack and 55 square meters (590 square feet) of deteriorated concrete were repaired at a cost of $1,009,000 (Can) A summary of the unit costs for labor and materials to repair cracks and deteriorated concrete is provided in Table 8.1. The costs were based on the nature and scope of work to be performed, site logistics, and the equipment needed for the successful completion of the work.

Based on underwater inspections of the bridge piers that were performed in 1988, an annual program to inject cracks and repair deteriorated concrete surfaces in the submerged sections and tidal zones of various piers was initiated in 1990. Five piers were repaired in 1990 and an additional 12 piers are scheduled for repair over the next three years.

TABLE 8.1 - SUMMARY OF LABOR AND MATERIAL COSTS*

ITEM	LABOR			MATERIAL		
	QUANTITY	UNIT COST	TOTAL COST LABOR	QUANTITY	UNIT COST	TOTAL COST MATERIAL
Crack Injection	3548 l m	$117/l.m	$415,116	3024 l	$20 34/l.	$61,508
Deteriorated Concrete	55 3 sq m	$1470/sq m	$81,291	6710 l	$6 68/l	$44,823
Total			$496,407			$106,331

*The unit costs shown in the table do not include the cost of mobilization, administrative costs, or taxes.

l.m = linear meter sq m = square meter l. = liter

REFERENCES

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2. Smoak, W.G., "Crack Repairs to Upper Stillwater Dam," Concrete International, Vol. 13, No. 2, February 1991, pp 33-36.

3. Houde, J.; Lacroix, P.; Morneau, M., "Rehabilitation of Railway Bridge Piers in Marine Environment," Marine Concrete '86, Proceedings International Conference on Concrete in the Marine Environment, The Concrete Society, London, England, September 1986, 10 p.

CHAPTER 9
SUMMARY AND CONCLUSIONS

Marine structures form a very important part of Canada's infrastructure and their ability to perform adequately to meet the demands of everyday life is vital to the national economic growth Yet, there is convincing evidence that a majority of these structures are in a state of disrepair requiring billions of dollars to upgrade them to meet the current demands. However, through sustained and combined efforts of the Federal government and the private sector, innovative reinvestment schemes, viable rehabilitation strategies, and continued support in research and development needs, the infrastructure crisis can be solved over a reasonable amount of time

Marine structures are located in one of the harshest environments known to man The causes of distress or deterioration which affect the durability of a concrete structure are many and can be loosely grouped into two main categories: physical and chemical. Within each category there are several sub-categories, and most often one is the direct cause of the other The most common causes of concrete deterioration found in a marine environment include corrosion of embedded reinforcing steel, freeze-thaw damage, alkali-silica reactivity, sulfate attack, and erosion However, through proper design and mix proportioning, and use of good quality materials and construction procedures, concrete structures can be made more durable

To develop an effective repair and maintenance program for the submerged portion of a marine structure, the causes of distress and deterioration must be clearly understood This requires a selective underwater survey, using a range of in-situ and laboratory testing techniques, to obtain the necessary information to perform an evaluation of the structure. Three basic levels of underwater inspection can be used to evaluate marine structures depending on the extent of the information required. These inspections can be performed by three basic modes of underwater diving techniques: manned diving, remotely operated vehicles (ROVs) and manned submersibles The extent of concrete deterioration, site logistics and the client's needs normally dictate the method of inspection to be used. The usefulness of an inspection depends on establishing a clear, concise and complete record of the findings This can be achieved by the use of standardized forms and report formats. Frequent and well organized inspections serve as a data-base and provide an effective means of keeping maintenance and repair costs to a minimum

The qualitative data obtained from visual inspections of deteriorated concrete structures is often inadequate to accurately assess the condition of the structure More useful information can be obtained by using more sophisticated concrete inspection methods. A wide range of in-situ inspection and laboratory testing techniques are available to the engineer for determining the quality and mechanical properties of concrete. These tests can either be destructive or nondestructive in nature, with the latter being preferred Destructive testing is usually undertaken to perform laboratory analyses on specimens retrieved from areas of the structure which are suspected of deterioration. Often a combination of both types of testing techniques is required to determine the primary cause or causes of deterioration. Prior to implementing any of these testing procedures, it is important for the investigator to be fully aware of the inherent limitations of the testing devices.

Once the inspection and evaluation of a structure is completed, the method chosen for repair is dictated by the cause and extent of deterioration, site logistics and cost. Repairing underwater concrete structures generally involves one or more of the following techniques: patch repair of spalled concrete, injection of cracks, and replacing large portions of the structure with new concrete. The success of a repair is contingent upon proper surface preparation of the damaged area, requiring a range of concrete removal methods and surface cleaning techniques. Numerous existing and recently developed techniques are available for placing concrete under water, using variations of the basic tremie and pumping methods. Recent developments in concrete admixtures to prevent cement washout has made pumping methods more preferable over the traditional tremie pipe. The degree of workability of the concrete needed for a repair depends primarily on the placement method.

Concrete piles supporting wharf structures and bridge pier footings are often damaged by corrosion of embedded reinforcing steel, abrasion, and scour. Several options are available for repairing the deteriorated concrete piles and generally involve the following: epoxy patching/injection, applying protective barrier coatings, encapsulation (or wrapping) and reinforced concrete jacketing. In extreme cases, partial or complete replacement is necessary. The state-of-the-art method for repairing concrete piles is polymer encapsulation. Through the use of special concrete additives and pumping techniques, this method is considered to be the most suitable for repairing concrete piles under water.

Scour damage to bridge pier footings has long been a problem for bridge managers and engineers. Several causes are responsible for pier scour, and in some cases, if it is not corrected immediately, it could lead to catastrophic failure of the bridge superstructure. Many techniques exist for repairing scour and generally involve replacing the scoured area with new material to restore the bearing surface to the pier New technologies and instrumentation systems are being developed to help detect and measure the extent of scour These systems are grouped into two categories: post-flood measurement systems, and real-time systems that monitor the streambed during floods.

A majority of concrete hydraulic structures, such as dams, spillways and stilling basins are reaching or have reached their design service life and need to be repaired. Many of these will be required to operate far beyond their design service life and must be upgraded to meet future demands. The four most common types of deterioration found in these structures are cracking, spalling, abrasion-erosion and seepage. The deterioration is primarily caused by the erosive action of high-velocity water and freeze-thaw cycles Repairing these structures usually requires dewatering the facility, and involves using a variety of concrete materials and techniques Polymer impregnation significantly improves concrete resistance to freeze-thaw cycles and chemical attack The addition of polymers and pozzolans such as silica fume have greatly improved the abrasion erosion resistance of concrete. The advent of antiwashout admixtures has made possible the placement of higher quality concrete under water, avoiding the high cost of dewatering

With continued research and development in the areas of degradation modelling, design, concrete materials, and concrete placement techniques, higher durability repairs can be achieved

APPENDIX A

CASE HISTORIES OF CONCRETE EXPOSED TO SEAWATER

This information is provided for reader interest and has been reproduced from the following reference:

Mehta, P.K , Concrete: Structures, Properties, and Materials, Prentice Hall, New Jersey, 1986.

History of Structures	Results of Examination
MILD CLIMATE	
Forty-centimeter mortar cubes made with different cements and three different cement contents, 300, 450 and 600 kg/m^3, were exposed to seawater at La Rochell, southern France, in 1904-1908 [a]	After 66 years of exposure to seawater, the cubes made with 600 kg/m^3 cement were in good condition even when they contained a high-C_3A (14 9%) portland cement. Those containing 300 kg/m^3 were destroyed, therefore, chemical resistance of the cement was of major importance for low cement content cubes In general, pozzolan and slag cements showed better resistance to seawater than portland cements Electron micrographic studies of deteriorated specimens showed the presence of aragonite, brucite, ettringite, magnesium silicate hydrate, and thaumasite
Eighteen 69 x 69 x 42 in. unreinforced concrete blocks, made with six different portland cements and three different concrete mixtures, partially submerged in seawater in Los Angeles in 1950. [b]	After 67 years of exposure, the dense concrete (1 2 4) blocks, some made with 14% C_3A portland cement, were still in excellent condition Lean concrete (1 3 6) blocks lost some material and were much softer (Fig.5-18a) X-ray diffraction analyses of the weakened concrete showed the presence of brucite, gypsum, ettringite, and hydrocalumite the cementing constituents, CSH gel and $Ca(OH)_2$, were not detected
Concrete structures of San Francisco Ferry Building, built in 1912. Type I portland cement with 14-17% C_3A was used. 1:5 concrete mixture contained 658 lb/yd^3 (395 kg/m^3) cement (a) Precast concrete cylinders jacket for Pier 17 (b) Cast-in-place concrete cylinders for Piers 30 and 39. (c) Cast-in-place concrete cylinders and transverse girders for Piers 26 and 28 [c]	After 46 years of service (a) was found in excellent condition, and 90% of piles in (b) were in good condition In (c) 20-30% of piles were attacked in tidal zone, and about 35% of the deep transverse girders had their underside and part of the vertical face cracked or spalled due to corrosion of reinforcement Presence of microcracks due to deflection under load might have exposed the reinforcing steel to corrosion by seawater Poor workmanship was held responsible for differences in behavior of concrete, which was of the same quality in all structures.

History of Structures	Results of Examination
COLD CLIMATE	
Many 20 to 50 year coastal structures were included in a 1953-55 survey of 431 concrete structures in Denmark.[d] Among the severely deteriorated structures were the following in Jutland.	Of the coastal structures, about 40% showed overall deterioration, and about 35% showed from severe surface damage to slight deterioration.
Oddesund Bridge, Pier 7. History of structure indicated initial cracking of caissons due to thermal stresses This permitted considerable percolation of water through the caisson walls and the interior mass concrete filing. General repairs commenced after 8 years of service.	Examination of deteriorated concrete from the Oddesund Bridge indicated decomposition of cement and loss of strength due to sulfate attack below low-tide level and cracking due to freezing and thawing as well as alkali-aggregate reaction above high-tide level. Reaction products from cement decomposition were aragonite, ettringite, gypsum, brucite, and alkali-silica gel.
Highway Bridge, North Jutland. Severe cracking and spalling of concrete at the mean water level provided a typical hourglass shape to the piers. Concrete in this area was very weak. Corrosion of reinforcement was everywhere and pronounced in longitudinal girders.	Examination of concrete piers of the highway bridge showed evidence of poor concrete quality (high w/c). Symptoms of general decomposition of cement and severe corrosion of the reinforcement were superimposed on the evidences for the primary deleterious agents, such as freezing-thawing and alkali-aggregate reaction.
Groin 71, north barrier, Lim, Fiord. Lean concrete blocks (370 lb/yd^3 cement) exposed to windy weather, repeated wetting and drying, high salinity, freezing and thawing, and severe impact of gravel and sand in the surf. Some blocks disappeared in the sea in the course of 20 years	Examination of the severely deteriorated concrete blocks from Groin 71 showed very weak, soapy matrix with loose aggregate pebbles. In addition t the alkali-silica gel, the presence of gypsum and brucite was confirmed.

History of Structures	Results of Examination
Along the Norwegian seaboard, 716 concrete structures were surveyed in 1962-64. About 60% of the structures were reinforced concrete wharves of the slender-pillar type containing tremie-poured under-water concrete. Most wharves had decks of the beam and slab type. At the time of survey, about two-thirds of the structures were 20-50 years old.[e]	Below the low-tide level and above the high-tide level, concrete pillars were generally in good condition In the splashing zone, about 50% of the surveyed pillars were in good condition, 14% had their cross-sectional area reduced by 30% or more, and 24% had 10-30% reduction in area of cross section. Deck slabs were generally in good condition but 20% deck beams needed repair work because of major damage due to corrosion of reinforcement. Deterioration of pillars in the tidal zone was ascribed mainly due to frost action on poor-quality concrete.

[a]M.Regourd, *Annales de L'Institute Technique du Bâtiment et des Travaux Publics*, No. 329, June 1975, and No. 358, February 1978.
[b]P.K. Mehta and H. Haynes, *J Struct Div ASCE*. Vol. 101, No. ST-8, August 1975.
[c]P.J. Fluss and S.S. Gorman, *J ACI, Proc* , Vol. 54, 1958.
[d]G.M. Idorn, "Durablity of Concrete Structures in Denmark," Ph.D. dissertation, Tech. Univ., Copenhagen, Denmark, 1967.
[e]O.E. Gjorv, Durability of Reinforced Concrete Wharves in Norwegian Harbors, The Norwegian Committee on Concrete in Sew Water, 1968.

APPENDIX B

SUMMARY OF DEFECTS OCCURRING DURING CONSTRUCTION

This information is provided for reader interest and has been reproduced from the following reference:

Allen, R.T.L.; Edwards, S.C.; Shaw, J.D.N., The Repair of Concrete Structures, 2nd Edition, Chapman and Hall, London, U.K., 1993, 212 p.

Summary of Defects Occurring During Construction			
Symptom	Cause	Prevention	Remedy
Cracks in horizontal surfaces, as concrete stiffens or very soon after.	Plastic shrinkage: rapid drying of surface.	Shelter during placing. Cover as early as possible Use air entrainment.	Seal by brushing in cement or low-viscosity polymer
Cracks above form ties, reinforcement etc., or at arrisses, especially in deep lifts.	Plastic settlement: concrete continues to settle after starting to stiffen.	Change mix design. Use air entrainment	Re-compact upper part of concrete while still plastic seal Seal cracks after concrete has hardened.
Cracks in thick sections, occurring as concrete cools.	Restrained thermal contraction.	Minimize restraint to contraction. Delay cooling until concrete has gained strength.	Seal cracks.
Blowholes in formed faces of concrete.	Air or water trapped against formwork. Inadequate compaction. Unsuitable release agent.	Improve vibration Change mix design or release agent. Use absorbent formwork.	Full with polymer-modified fine mortar.
Voids in concrete. Honeycombing	Inadequate compaction. Grout loss.	Improve compaction. Reduce maximum size of aggregate. Prevent leakage of grout.	Cut out and make good. Inject resin.
Erosion of vertical surfaces, in vertical streaky pattern.	Scouring: water moving upwards against form face.	Change mix design, to make more cohesive or reduce water content.	Rub in polymer-modified fine mortar.
Color variation.	Variations in mix propoitions, curing conditions, materials, characteristics form face, vibration, release agent. Leakage of water from formwork.	Ensure uniformity of all relevant factors. Prevent leakage from formwork.	Apply surface coating.

Summary of Defects Occurring During Construction			
Symptom	Cause	Prevention	Remedy
Powdery formed surfaces.	Surface retardation, caused by sugars in certain timbers.	Change form material. Seal surface of formwork. Apply lime-wash to form face before first few uses.	Generally none required.
Rust stains.	Pyrites in aggregates. Rain streaking from unprotected steel. Rubbish in formwork. Ends of wire ties turned out.	Avoid contaminated aggregates. Protect exposed steel. Clean forms thoroughly. Turn ends of ties inwards.	Clean with dilute acid or sodium citrate/sodium dithionite. Apply surface coating.
Plucked surface.	Insufficient release agent. Careless removal of formwork.	More care in application of release agent and removal of formwork.	Rub in fine mortar, or patch as for spalled concrete.
Lack of cover to reinforcement.	Reinforcement moved during placing of concrete, or badly fixed. Inadequate tolerance in detailing.	Provide better support for reinforcement. More accurate steel-fixing. Greater tolerances in detailing.	Apply polymer-modified cement and sand rendering. Apply protective coating.

APPENDIX C

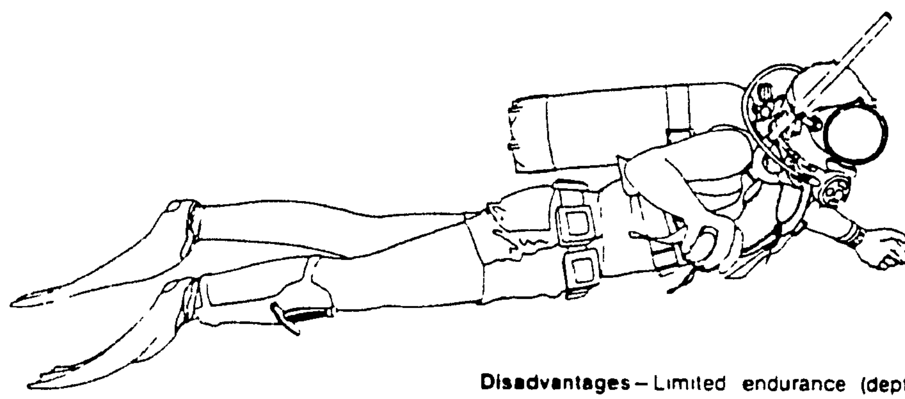
CHARACTERISTICS OF THE BASIC DIVING MODES

This information is provided for reader interest and has been reproduced from the following reference:

Lamberton et al., "Underwater Inspection and Repair of Bridge Substructures," NCHRP Report No. 88, Transportation Research Board, Washington, D.C., December 1981.

SCUBA

GENERAL CHARACTERISTICS



Minimum Equipment –
Open-circuit SCUBA
Life preserver
Weight belt
Knife
Face mask
Swim fins

Principle Applications –
Shallow water search
Inspection
Light repair and recovery
Clandestine operations

Advantages –
Rapid deployment
Portability
Minimum support
Excellent horizontal and vertical mobility
Minimum bottom disturbances

Disadvantages –
Limited endurance (depth and duration)
Breathing resistance
Limited physical protection
Influenced by current
Lack of voice communication

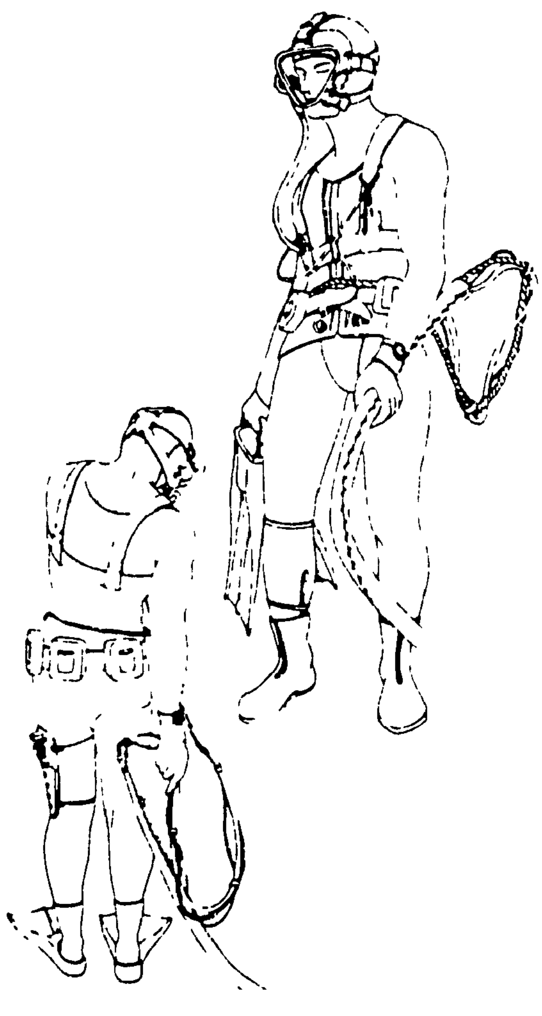
Restrictions –
Working limits –
Normal 60 feet/60 minutes
Maximum 130 feet/10 minutes
Current – 1 knot maximum
Diving team – minimum 4 men

Operational Considerations –
Buddy and standby diver required
Small boat required for diver recovery
Avoid use in areas of coral and jagged rock
Moderate to good visibility preferred
Ability to free ascend to surface required

OPERATIONS PLANNING

LIGHTWEIGHT DIVING

GENERAL CHARACTERISTICS



Minimum Equipment— Diver s Mask USN MK 1 or Jack Browne mask
Wet suit
Weight belt
Knife
Swimfins or shoes
Surface umbilical

Principle Applications— Shallow water search
Inspection and major ship repair
Light salvage

Advantages— Unlimited by air supply
Good horizontal mobility
Voice and/or line pull communications
Fast deployment

Disadvantages— Limited physical protection
Limited vertical mobility
Large support craft required

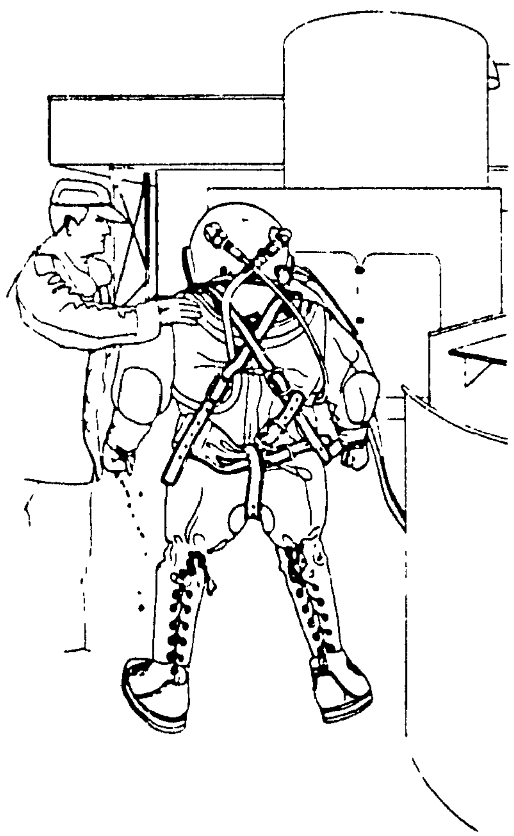
Restrictions— Work limits—Jack Browne —
Normal 60 feet/60 minutes
Maximum 90 feet/30 minutes
Work limit—MK 1 without come home bottle
Maximum 60 feet
Work limit—MK 1 without open bell
Maximum 130 feet/10 minutes
Work limit—MK 1 with open bell
Maximum 190 feet/60 minutes
Current—2 5 knots max

Operational Considerations— Ability to free ascend to surface required, with exception noted in Para 6 8 7 2
Adequate air supply system
Standby diver required

U S NAVY DIVING MANUAL

DEEP-SEA DIVING

GENERAL CHARACTERISTICS



Principle Applications –	Deep diving operations Heavy salvage and repair Underwater construction
Advantages –	Unlimited by air supply Maximum physical and thermal protection Voice and line pull communication Variable buoyancy
Disadvantages –	Slow deployment Poor mobility Large support craft and surface crew
Restrictions –	Work limits – Normal 190 feet/40 minutes Maximum 250 feet/90 minutes (Exceptional Exposure) Current – 2.5 knots Diving team – minimum 6 men
Minimum Equipment –	Helmet and breastplate Diving dress Thermal underwear Weight belt Weighted shoes Knife Rubber cuffs and/or gloves Surface umbilical
Operational Considerations –	Adequate air supply system Stand-by diver required Medical officer and recompression chamber required below 170 feet Exceptional exposures require approval of Commanding Officer or higher authority

OPERATIONS PLANNING

APPENDIX D

EXAMPLES OF ROVs

This information is provided for reader interest and has been reproduced from company information brochures of:

International Underwater Contractor's, Inc.

ROV Mantis:
2,300-Foot Remotely Operated or Manned Vehicle

General

Operating depth	2,300 feet (701 meters)
Forward speed	In excess of 1.0 knots
Length o.a.	8.8 feet (2.68 meters)
Breadth	7.3 feet (2.23 meters)
Height	4.5 feet (1.37 meters)
Weight	2,300 pounds (1,043 kilograms)
Life support	96 hours (manned intervention)
Surface supply	380/440 v 3 phase a.c.
Forward/Aft propulsion	6 a.c. thrusters (vectored), each giving 80 pounds (36.3 kilograms) thrust. 4 d.c. thrusters that can be fed from batteries in a power loss situation
Vertical propulsion	2 a.c. thrusters
Additional power	4 k.w. (available for tools, etc.)
Manoeuverability	360° in horizontal and vertical planes
Payload	440 pounds (199.6 kilograms)
Lift capability	50 pounds (22.7 kilograms)
Manipulators	Two 6 function manipulators
Ballast	Variable trim tanks
Certification	ABS and Lloyds

Surface Controls

Depth control	Auto/manual (± 0.5 feet)
Height control	Auto/manual (± 0.5 feet)
Heading control	Auto/manual (± 7 degrees)
Trim control	Remote pitch indicator
Ballast trim tanks	Manual or remote control of buoyancy with surface monitoring of air reservoirs
Life support sensors	Water intrusion, % oxygen, % carbon dioxide, and internal pressure
Multiplex system	Time division system standard
Diagnostic capability	Full software and hardware diagnostic program with test equipment allowing fault tracing to component level
Manipulators	Standard controls for each function

Work & Documentation

Viewport	■ 2.2 feet (0.67 meters) diameter clear acrylic hemisphere
Television cameras	■ Two cameras via direct co-axial connection. A third camera may share one channel by remote switching. Focus control can be by pilot or surface for any of the three cameras. Each TV channel has a monitor with video taping capability
Data display	■ In addition to client specific video annotation, the system can also display vehicle status and other pertinent information
Still camera	■ Provision for 35mm or 35mm stereo camera with strobe operated by pilot or remote
Scanning sonar	■ Ametek Model 250A scanning sonar or mesotechs new color sonar
Manipulators	■ Two 6 function sea water hydraulic manipulators standard HEA arm
Tools	■ Cable cutter, grinder, A X Ring tool, wirebrush, and other various tools

Communication & Navigation

Underwater	■ Hardware and acoustic system providing communication of 10 KHz, 27.5 KHz, and 37.5 KHz with directional pinger mode
Direction	■ Direction gyro and magnetic compass
Depth sensor	■ Digital depth system with indicator to pilot and surface
Altitude (bottom depth)	■ Digital 200 KHz echo sounder
Tracking	■ Standard Honeywell RS 7 beacon

System Size & Weight

Control Cabin	■ 8'H x 8'W x 10'L (2.44m x 2.44m x 3.0m), 3 tons (2,722 kilograms)
Umbilical Winch w/Mantis	■ 6'H x 8'W x 12'L (1.83m x 2.44m x 3.66m), 5 tons (4,536 kilograms), skid mounted
Launch/recovery system w/hydraulic power pack	■ 8'H x 8'W x 12'L (2.44m x 2.44m x 3.66m), 7 tons (6,350 kilograms), skid mounted

International Underwater Contractors, Inc.

Recon IV:
2.300-Foot Remotely Operated Vehicle

General

Operating depth	2 300 feet (701 meters)
Length o a	6 5 feet (1 98 meters)
Breadth	3 feet (9 meters)
Height	2 75 feet (84 meters)
Weight in air (gross)	900 pounds (410 kilograms)
Speed, forward	3 knots
Speed, lateral	2 knots
Payload (wet)	250 pounds (114 kilograms)
Depth control	Automatic or manual
Thrusters	Four 1 HP electric (80 pound thrust)
Portable consolette	50 feet (extended) reach with controls for thrust pan, tilt, camera focus, flying tether payout vehicle lighting and manipulator

Tether Management System

Operating depth	2 000 feet (610 meters)
Diameter	4 58 feet (1 4 meters)
Height	4 33 feet (1 3 meters)
Weight in air (gross)	1,650 pounds (748 42 kilograms)
Tether drive motor	1 HP electric (100 pounds pull)
Tether payout indicator	Digital surface meter

Tether (Vehicle to Cage)

Length	400 feet (121 9 meters)
Breaking strength	4 000 pounds (1,814 kilograms)
Strength member	Braided Kevlar
Weight in water	Neutrally buoyant

Main Umbilical (Winch to Cage)

Length	2 200 feet (670 56 meters)
Diameter	1 25 inches (3 18 centimeters)
Breaking strength	30,000 pounds (13,607 8 kilograms)
Armor	Contrahelically wound improved plough steel, two layers

Handling System

Type	Skid mounted U frame
Size	17' (L) × 8' (W) × 9' (H)
Power	Electric 220/440 v 3 phase
Line speed (full drum)	100 feet/min (30 48 meters/min)

Work & Documentation

Manipulator	One 4 function (optional second arm)
Tools	Grinder cable cutters water jet and other various tools
Television camera	Sub Sea, CM50 color or black & white (CM 8)
Video recorder	Two ½ inch cassette units
Video monitor	12 inch (30 48 centimeters) color
Video annotation	Date time depth heading and CP
Remote video monitor	Color or black & white at up to 50 feet (15 24 meters) away
Lighting	Four 250 watt incandescent (variable intensity)
Pan & tilt	270 degrees pan 180 degrees tilt
Pan & tilt (speed)	45 degrees per second
Still camera	35mm (standard or stereo)
NDT	CP Probe Harco Model HRP 803

Navigation

Sonar	Straza 250A
Compass	Digicourse Magnetic
Depth sensor	0 2300 feet (0 701 meters) ± 0 5% of full scale

International Underwater Contractors, Inc.

Recon IV:
1,000-Foot Remotely Operated Vehicle

General	
Operating depth	1 000 feet (305 meters)
Length o a	6 5 feet (1 98 meters)
Breadth	3 6 feet (1 1 meters)
Height	2 75 feet (84 meters)
Weight in air (gross)	900 pounds (410 kilograms)
Speed, forward	3 knots
Speed, lateral	1 knot
Payload (wet)	40 pounds (18 kilograms)
Depth control	Automatic or manual
Thrusters	Four 1 HP electric (160 pound forward thrust)
Portable consolette	50 feet (extended) reach with controls for thrust, pan, tilt, camera focus, flying tether payout, vehicle lighting and manipulator

Tether Management System	
Operating depth	1 000 feet (305 meters)
Diameter	4 58 feet (1 4 meters)
Height	4 33 feet (1 3 meters)
Weight in air (gross)	1 650 pounds (748 42 kilograms)
Tether drive motor	1 HP electric (100 pounds pull)
Tether payout indicator	Digital surface meter

Tether (Vehicle to Cage)	
Length	400 feet (121 9 meters)
Breaking strength	2,000 pounds (907 kilograms)
Weight in water	Neutrally buoyant

Main Umbilical (Winch to Cage)	
Length	1 200 feet (366 meters)
Diameter	1 25 inches (3 18 centimeters)
Breaking strength	30 000 pounds (13 607 8 kilograms)
Armor	Contrahelically wound improved plough steel two layers

Handling System	
Type	Skid mounted U frame
Size	14' (L) x 8' (W) x 8' (H)
Weight	18,000 lbs (with vehicle and TMS)

Documentation	
Television camera	Black and White (CM 8) or Sub Sea, CM50 color (Optional)
Video recorder	Two ½ inch cassette units
Video monitor	12 inch (30 48 centimeters)
Video annotation	Date time, depth and heading
Remote video monitor	Black and White at up to 50 feet (15 24 meters) away
Lighting	Two 250 watt incandescent (variable intensity)
Pan & Tilt	90 degrees pan, 80 degrees tilt

Navigation	
Compass	Magnetic
Depth sensor	0 1000 feet (0 305 meters) ± 0 5% of full scale

Control Cabin	
Size	16' (L) x 8' (W) x 8' (H)
Power	220/440V 3 Phase 60 amps
Weight	8,000 lbs

International Underwater Contractors, Inc

APPENDIX E

EXAMPLES OF MANNED SUBMERSIBLES

This information is provided for reader interest and has been reproduced from company information brochures of:

International Underwater Contractor's, Inc.

Mermaid II: 1,000-Foot Submersible

General

Operating depth	1,000 feet (305 meters)
Length o a	20 feet (6 09 meters)
Breadth	6 5 feet (1 98 meters)
Height	9 4 feet (2 86 meters)
Tonnage (gross)	6 3 tons
Crew	2
Life support	240 man hours
Speed (cruising)	1 5 knots
Speed (top),	4 knots
Power	28 kw
Propulsion	7 hp slewable mainthruster, 2 side mounted horizontal 90 degree slewable reversible thrusters of 2 hp each
Maneuverability	5 degrees and hovering, rotates 360 degrees at zero velocity
Payload	1,000 pounds (454 kilograms)
Lift capability	500 pounds (227 kilograms)
Certification	American Bureau of Shipping

Work & Documentation

Viewports	■ 8 including a 30 inch (0 76 meter) diameter bow window
Television	■ Pan and tilt color CCTV plus audio and video recording
External lighting	■ 4, 1000 watt and two 150 watt quartz iodide lamps
Manipulator	■ Two with 6 degrees of freedom, 75 pounds (34 kilograms) lift, fully extended ■ 35mm external still camera (250 exposure) with strobe

Navigation & Communications

Underwater	Mesotech Acoustic 9 and 27 khz
Surface	VHF FM
Direction	Sperry CL II directional gyro
Altitude/depth	Wesmar digital depth sounder
Obstacle avoidance	Wesmar SS 140 Sonar
Tracking	Honeywell RS 7
Bottom/position	ELA 20

Support Equipment

Equipment	■ 15 ton "A" frame lift system ■ Battery replacement trolleys with complete charging system

Work

Mermaid II has a history as a tough and reliable work boat It has completed contracts in the North and South Atlantic and the Gulf of Mexico, as well as numerous inland waters of the continental US Two jobs stand out as representative of this submersible s steadiness and versatility (1) A number of low profile environmental impact trays in the Baltimore Canyon waters to 400 ft were deemed all but lost by the client, who came to IUC when numerous attempts at locating the arrays had failed In the contracted 7 days at sea 5 arrays were located and retrieved with their critical bottom samples intact, (2) In the Gulf of Mexico, 17 bouys, 33 ft in length, spaced along a 2,000 ft section of pipeline, were cut loose in 320 ft of water in a total bottom time of 8 hours, on a job which had been determined to be too dangerous for divers In addition Mermaid has been used on numerous occasions to evaluate damaged and collapsed platforms in the aftermath of fire on the decks and underwater blowout

Beaver Mk IV:
2,000-Foot Submersible with Wet & Dry Lockout

General

Operating depth	2,000 feet (610 meters)
Lock out depth	1,000 feet (305 meters)
Length o a	24 feet (7 3 meters)
Breadth	9 5 feet (2 9 meters)
Height	8 5 feet (2 6 meters)
Tonnage (gross)	34,000 pounds (15 422 kilograms)
Crew	2 crew (and up to 3 observers/ divers)
Life support	360 man hours (minimum), with emergency battery and lighting system
Speed, cruising	2 5 knots
Speed, top	5 7 knots
Power	52 kw/hr, lead acid batteries
Propulsion	Three 7 hp reversible thrusters with 360-degree rotation
Payload	2,000 pounds (907 kilograms)
Lift capability	1,500 pounds (680 kilograms) at maximum depth
Certification	American Bureau of Shipping

Work & Documentation

Viewports	■ 6 including a 30 inch (0 76 meter) forward downward window
Television camera	■ Color, externally mounted on pan and tilt unit with videotape recorder
Still camera	■ External 35mm with strobe
Manipulators	■ 2 with 10° of freedom and reach of 6 feet (1 83 meters) lift at wrist of 50 pounds (22 7 kilograms) lift at shoulder of 250 pounds (113 kilograms), plus continuous wrist rotation with 83 foot pounds (11 5 meter kilograms) torque
Manipulator tools	■ Cable cutter wire brush impact wrench stud gun, power hammer and saw
Torque operations	■ High speed, low torque 0 5300 rpm @ 3 5 foot pounds (0 5 meter kilograms) maximum Low speed, high torque 50 1500 rpm @ 83 foot pounds (11 5 meter kilograms) maximum Impact torque 1 200 foot pounds (166 meter kilograms)

Navigation & Communications

Underwater	Mesotech 703 A
Direction	Sperry Mk 27 gyrocompass
Altitude/depth	EDO 326 downward and upward looking sonar
Obstacle avoidance	Wesmar 146
Tracking	Honeywell PS / acoustic position system
Bottom/position	ELA 20 bottom mounted trans ponder system

IUC Canada

Pisces VI:
8,300-Foot Submersible

General

Operating depth	6 600 feet (2 012 meters)
Design depth	8,300 feet (2 530 meters)
Length o a	19 25 feet (5 86 meters)
Breadth	10 feet (3 5 meters), can be reduced to 8 feet (2 44 meters) for airfreight
Tonnage (gross)	23,500 pounds (10,660 kilograms)
Crew	2
Life support	250 man hours (minimum)
Speed (cruising)	2 knots
Speed (top)	3 knots
Power	42 kw
Propulsion	2 side thrusters at 7 hp each, slewable 100 degrees
Payload	1,500 pounds (680 kilograms)
Lift capability	800 pounds (363 kilograms)
Certification	American Bureau of Shipping
Built	Vancouver, Canada

Work & Documentation

Viewports	■ Three 6 inch (0 15 meter) diameter acrylic windows
External lighting	■ Three 1 000 watt quartz iodide lamps
Television camera	■ Internal and external camera on pan and tilt unit
Still camera	■ Benthos 35mm camera with strobe ■ Photosea stereo 35mm camera with strobe
Manipulators	■ Heavy duty claw with 2 3 foot (0 70 meter) grip and 250 pound (113-kilogram) lift General purpose arm with six independent movements 7 inch (0 18 meter) grip with 150 pound (68 kilogram) lift
Cable cutters	■ Two

Navigation & Communications

Underwater	27 khz and 9 khz, 100 watts
Surface	VHF radio
Direction	Sperry MK 37 gyro
Altitude/depth	Depth gauge
Obstacle avoidance	Wesmar SS 140 Sonar
Tracking	Directional transducer 27 khz, Honeywell RS 7 system
Bottom/position	ELA 20 Doppler Navigation Sonar, Variable frequency pinger receiver

Work

Besides its world record dives to 4,876 feet off Newfoundland, in '79, Pisces VI has achieved a number of other offshore "firsts " In '82, a new record for deep water permanent guide base retrieval was established when the client oil company sought to extend its practice of restoring the sea bottom to pre drilling status in deeper water than had been previously attempted At a depth of 1,263 feet, Pisces VI placed a chemical explosives charge in the PGB funnel After successful detonation, the guide base was carefully retrieved to the deck of the drillship Later examination found it to have suffered no structural damage of any kind At one point in the retrieval manuever, the running tool was accidentally backed out and disconnected from the drill pipe Pisces VI, using its manipulator and heavy duty claw together, made it possible to reconnect the tool to the Guide Base and complete the deepest water PGB retrieval on record Mating the pipe threads of the running tool and pipe in such deepwater also is claimed as a submersible work manuever record

Life Support and Safety

Mechanical redundancy, together with personnel that adhere to sound procedures and practice, is the best assurance that life support and safety aboard a submersible will be sufficient to meet any eventuality On board Pisces VI, pilot and co pilot equipment for approximately 7½ days of total life support consists of dual scrubbers, 7 days of O² and sodasorb, 2 DUI sodasorb packets and chest pacs, MSA self-rescuer pacs, 2 Drager sodasorb pacs with bottles and 7 days of Lithium for scrubbers or pilot sphere dispersion

APPENDIX F

EXAMPLES OF SATURATION D'VING SYSTEMS

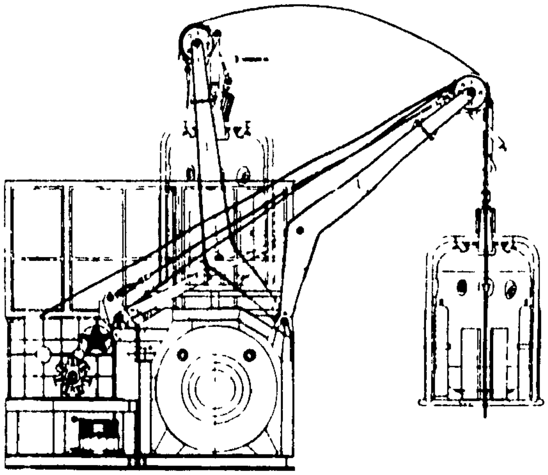
This information is provided for reader interest and has been reproduced from company information brochures of:

International Underwater Contractor's, Inc.

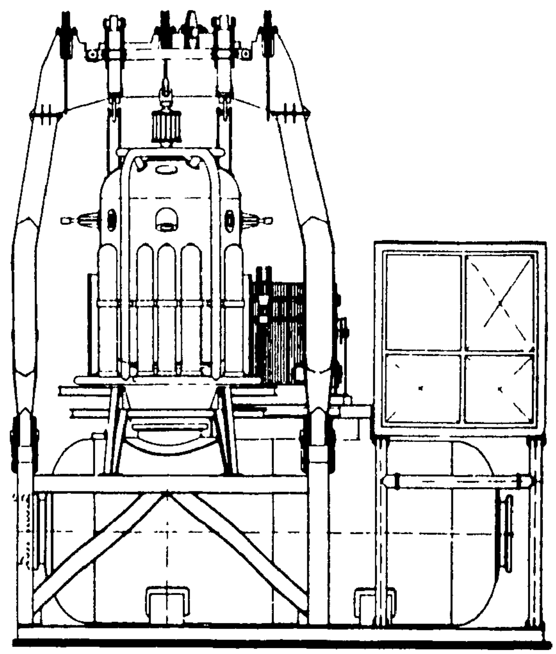
1,000-Foot Saturation System

General

Length	19.5 feet (5.9 meters)
Width	15.4 feet (4.7 meters)
Height (Control Van)	15.1 feet (4.6 meters)
Height (U-frame)	23.8 feet (7.25 meters)
Characteristics	■ 1,000-foot (305-meter) saturation diving system ■ 465 square feet (43.2 square meters) of deck space required ■ 35 feet (10.7 meters) of clearance required for U-frame ■ Total system weight 45 tons ■ Lloyd's certification ■ Special gas mixing equipment



Side View



Front (Sea) View

600-Foot Deep-Diving System

The Diving Bell

The standard bell is a 5.5 foot (1.68 meter) I.D. sphere with an operational depth capability of 600 feet (183 meters).

The bell consists of the following components:
- ■ Two through-hull coamings with double doors, one at the bottom for diver ingress and egress and one at the side for transfer under pressure to the skid mounted decompression chamber.
- ■ Three 12 inch (0.3 meter) side viewports and a total of six 6 inch top and bottom viewports arranged for omnidirectional visibility.
- ■ Backup life support gas storage.
- ■ A four wire telephone system with a redundant amplifier in the bell to take over in the event of topside unit failure.
- ■ A quick release mechanism to enable the divers in the bell to detach the 2,000 pound (907 kilogram) ballast for emergency ascent.
- ■ A dive control panel to deliver breathing gas to two divers, with backup system onboard that automatically sustains them in the event of umbilical pressure drop.

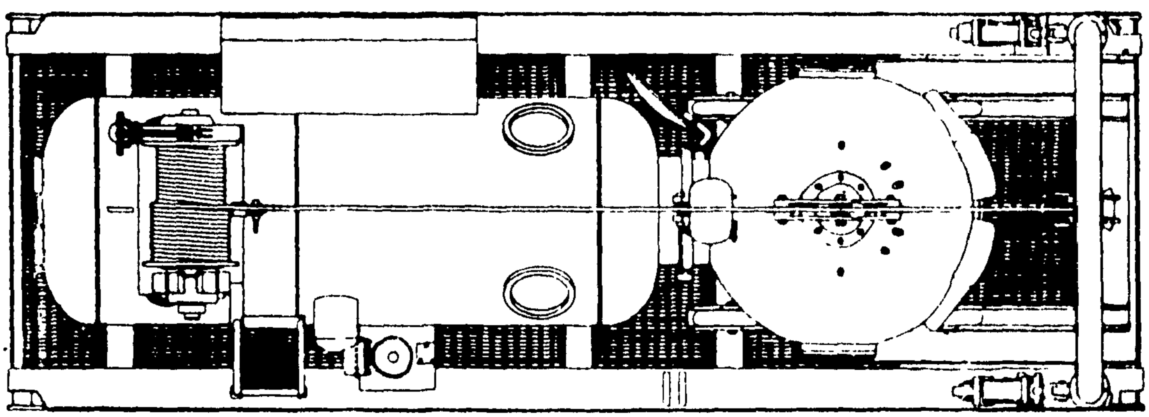
The Decompression Chamber

The 4.5 foot (1.4 meter) I.D. double lock decompression chamber is designed for transfer and decompression depths of 600 feet (183 meters). The main lock is 6 feet (1.83 meters) long and the entry lock is 4 feet (1.23 meters) long.

Chamber equipment includes the following components:
- ■ Communications in each lock.
- ■ Mixed gas and oxygen breathing facilities in each lock.
- ■ A medical lock.

Transportability

IUC's diving system (complete with support units weighing 15 tons) can be transported by truck, shipped by air and handled by most ship or platform cranes. Once on the job this system operates with its own winch and handling units. The system takes minimal deck space and needs only 220/440 3Φ 50KVA power and 150 psi service air 80 CFM.



The 600 Foot Deep Diving System (Top View). Basic Dimensions: Length 23.4 feet (7.1 meters). Width 8.0 feet (2.4 meters). Overall Height 13.2 feet (4.0 meters). Height at Umbilical Winch 9.7 feet (2.9 meters).

650-Foot Saturation System

The Diving Bell

The 450 ft diving bell has a 3'11" horizontal ID and 6'6" vertical It incorporates all standard safety equipment and many special IUC safety innovations as well It is equipped with

- CO scrubber, oxygen analysis meters, CO_2 analysis, metabolic oxygen makeup
- internal/external depth gauges
- temperature monitoring
- internal/external lighting
- emergency backup on board gas supply
- automatic secondary gas supply — in case of primary failure
- primary, secondary and auxiliary communication lines
- wireless emergency communications
- outside self powered beacon light

The Entry Lock Chamber

The entry lock chamber is 4'1" by 3'6" ID and is part of the deck decompression chamber It has an entry/exit hatch to the deck and deck decompression chamber making it possible to gain access to the divers at any stage in their decompression period The mating clamp is hydraulic with manual pins for safety backup

The Deck Decompression Chamber

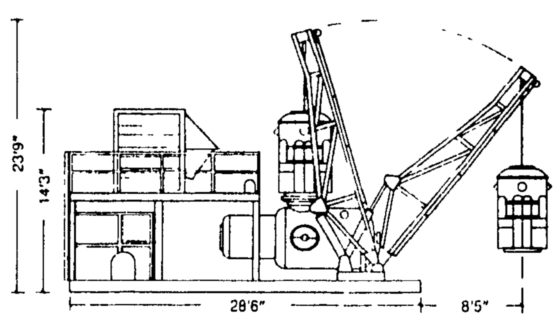
The main deck decompression chamber is 7'6" by 4'10" It accommodates two men comfortably for extended decompression periods It contains

- gas bibs, CO_2 scrubber and metabolic O_2 makeup
- O_2 and CO_2 analysis
- lighting and heater/cooler
- primary and secondary communications
- depth gauge

Topside support equipment includes

- hydraulic main winch
- hydraulic constant tension winch
- hydraulic A frame

IUC's 20 years experience in offshore diving have produced one of the best diver safety records in the world The 450 ft system is one of the reasons why Because of safety and other problems associated with the use of ambient (open bottom) diving bells in depths below 150 ft IUC minimizes their use in water this deep A full (closed bottom bell) dive system offers many advantages over an open bell system that are not always considered such as 1) the capability of bringing an engineer down as an observer for a firsthand view of the work area and job in progress, 2) increased bottom time for repairs, retrievals or other situations that require immediate action 3) an increased safety factor for the divers, including rapid ascent time made possible by the elimination of in water decompression stops Also, in strong current and rough seas launch and recovery manuevers are less dangerous with a closed bottom bell equipped with a constant tension guide wire system



System Width 11'10"

450-Foot Deep Diving System

The Diving Bell

The 650 ft diving bell has a 4'6" horizontal ID and 6'2" vertical It incorporates standard safety equipment and many special IUC safety innovations it is equipped with
- environmental hot water heater
- CO_2 scrubber, oxygen analysis meters, CO_2 analysis, metabolic oxygen makeup
- internal/external depth gauges
- temperature monitoring
- internal/external lighting
- emergency backup on board gas supply
- automatic secondary gas supply– in case of primary failure
- primary, secondary and auxiliary communication lines
- wireless emergency communications
- outside self powered beacon light

The Entry Lock Chamber

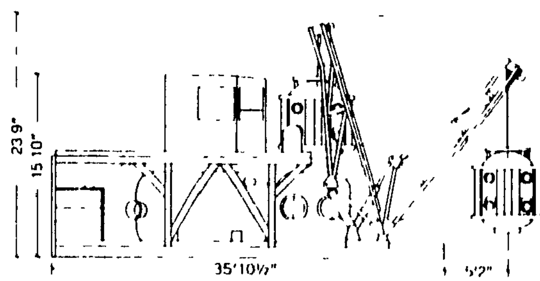
The entry lock chamber is 3'6" by 6'6" ID and is an integral part of the deck decompression chamber It has three exit/entry hatches to and from the deck, the deck decompression chamber, itself and the diving bell, making it possible to gain access to the divers at any stage in their decompression period The mating clamp from the bell is hydraulic with manual pins for added safety The entry lock chamber also contains the system s sanitation facilities, shower, sink and toilet

The Deck Decompression Chamber

The main deck decompression chamber is 6'6" by 9' ID It is fully insulated (including the entry lock chamber portion) Four men can be comfortably housed for extended decompression periods It contains
- gas bibs and emergency scrubbers (gas, electric or air powered)
- roomy medical lock for food, medicine and other needs
- main chamber lighting and special reading lamps
- observation ports
- work table and bunks for four men
- special environmental control unit for maintaining proper temperature, humidity and CO_2 scrubbing

Deck and topside support equipment includes a diesel hydraulic power unit that operates the bell A frame, main winch and bell mating clamp When the bell is launched or recovered it is connected to a constant tension system that maintains it in correct alignment throughout descent/ascent whatever the wind or current

All IUC dive teams include at least one emergency medical technician EMT diver All IUC divers are tenders, all tenders are divers, resulting in minimum crew size for safer efficient diving services



System Width 13 6"

APPENDIX G

TYPICAL DIVING SUPPORT VESSEL

This information is provided for reader interest and has been reproduced from company information brochures of:

International Underwater Contractor's, Inc.

The Aloha:
Submersible/RCV/Diving Support Vessel

General

Length o a	143 feet (43 6 meters)
Breadth	32 feet (9 8 meters)
Draft (loaded)	8 3 feet (2 5 meters)
Tonnage (gross)	165 tons (149 7 metric tons)
Tonnage (net)	119 tons (108 metric tons)
Crew	22 persons
Speed, cruising	12 knots
Range	6,000 nautical miles (11,114 kilo meters)
Propulsion	Twin screw with bow thruster
Main engines	Two Caterpillars (3412) 520 BHP each
Auxiliary power	Two 75kw generators
Fuel capacity	24,000 gallons (90,848 liters)
Fresh water	14,000 gallons (52,995 liters)
Maneuverability	Bow thruster (360° slewable)
Lift capability	Carrier 10 ton crane and 15 ton lift system
Certification	American Bureau of Shipping

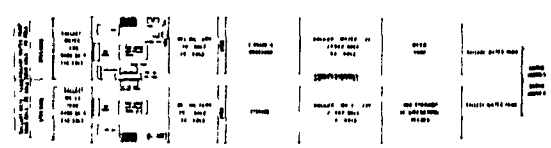
Navigation & Communications

Loran C	■ Micrologic ML 1,000 (primary) ■ Morrow LCM 950 (backup)
Gyrocompass	■ Sperry Mk 24
Autopilot	■ Decca Arkas 550G
Radar	■ Konel Furuno FR 116 (16 miles, 25 7 kilometers) ■ Konel Furuno FRN 64 (64 miles, 103 kilometers)
Radio	■ SSB CAI 35 MS/MK 11 synthesized 150 watts ■ SSB Intech Mariner 1 600 (12 channel 150 watts), unlimited range (primary) ■ VHF Konel KR-153V (10 channels, 100 watts) ■ Oki 1010 VHF (all channels, 25 watts)

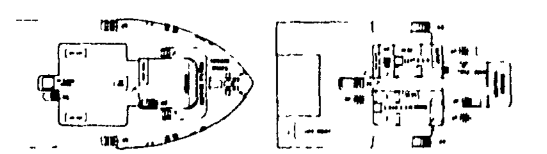
Additional Services Available

Systems	■ Complete diving system ■ Remote control tethered under water survey system ■ Two man 1 000 foot submersible ■ Submersible launch and recovery system designed and built to handle Mermaid II safely over the stern
Anchoring	■ 4 point mooring two double drum water fall type winches, with pneumatic controls centrally located 4 000 feet 1" (6 x 19) IWRC wire rope

Main Deck



Lower Deck



Pilot House

International Underwater Contractors, Inc.

APPENDIX H

PROCEDURES FOR INSPECTING UNDERWATER SEALS, FOOTINGS, AND PILES (NORTH CAROLINA, USA)

This information is provided for reader interest and has been reproduced from the following reference:

Lamberton et al., "Underwater Inspection and Repair Bridge Substructures," NCHRP Report No.8, Transportation Research Board, Washington, D.C., December 1981.

I. General Preparation and Safety Procedures

A. Preparation

1. Review plans (and specifications) of seals and footings, excavation sections, water conditions, and bridge and pile data.

2. Review previous inspection reports

3. Ensure that necessary inspection equipment is working property and is inventoried in vehicle.

B. Safety Procedures

1. Practice basic safety procedures as instructed by certifying agency

2. Have divers descend slowly in case of poor visibility and sharp objects along descent path or on bottom.

3. Take precautions in severe currents

a. Attach safety line on upstream side of bent or piles for divers' use underwater DO NOT use safety line when visibility is poor unless line is completely taut.

b. Stretch safety line across stream 45 m (150 ft.) downstream in rivers up to 91.5 m (300 ft.) wide.

c. In tidal areas attach 30 m (100 ft) of rope with ring buoy on stern of boat. Tie boat to bent or piles

d. Man a safety boat downstream of current.

e. Have divers return to bent or piles to protect themselves from boat traffic.

II. Inspection

A. Use masonry hammer, probing rod, rule, scraper, divers' tools, caliper, increment borer, marker.

B. Cofferdams

1. Inspect area at sheeting.

A minimum of two divers will perform inspection. Each diver inspects half the perimeter of the cofferdam sheeting at base of excavation. Each diver keeps in touch with sheeting with hands or feet and moves along the cofferdam while inspecting the base of excavation from the sheeting to as far as it is possible to reach.

The diver should look for any clay, mud, or silt buildup in all corners or between base of excavation and sheet piling throughout cofferdam. The base of each sheet piling should be examined to ensure that sheeting is driven all the way down (this is important in cases of shallow excavation depth). Check for any loose, large rocks that might be leaning against sheeting.

2. Inspect excavation base.

   Divers will look for mud, clay, silt, any loose rock, or other loose, hard foundation material. Surface should be clean of loose material. Divers will also describe geometric features and contour of surface, which can be specified as level, stepped, or serrated. A rock surface should be left rough.

   Surface should be inspected for material description such as sound rock, decomposed rock, or firm clay If surface is not sound rock, a sample of material is taken. Because specifications call for surface to be cut to firm surface, diver should be sure foundation material is what designer expected.

3. Inspecting corners and all corrugations of sheeting from natural ground to base of foundation for any earth inclusions must also be done.

4. Inspecting near center.

   When inspecting cofferdam up to 6 m (20 ft.) wide, at base of excavation near center, one diver keeps one hand on the sheeting while the other hand guides the other diver to near the center. The second diver inspects cofferdam while being guided and moved completely around it be the first diver.

   When inspecting cofferdams over 6 m wide, divers first use the method for those up to 6 m wide and then place a rope with weight on one end along the bottom to complete inspection. This is done as one diver carries the weighted rope and stations it near center of cofferdam wall, while other diver carries other end of rope and holds on bottom at opposite wall. First diver can then proceed on each side of rope, which is used as a guideline. Rope can be moved to a second location if inspection cannot be completed at first location.

C. Seals

1. Make layout of seal.

   Number with crayon the interior corrugations on each face of seal and record.

   Measure distance from interior corrugation to edge of footing on all seal sides at each corner of footing. This will establish the position of the footing onto seal.

Measure distance from interior corrugation to edge of footing on all seal sides at each corner of footing This will establish the position of the footing onto seal.

Measure height of seal from mudline at all four corners and record (report drawings are made from this information).

2. Inspect for condition of concrete

Inspect for soundness and appearance and take photographs when possible.

Inspect for spalls; measure their width, length, and height, and locate them on layout drawing.

Inspect for cracks and measure size, length, and depth Record crack location at designated corrugation number by recording the distance from top of seal to the crack within that corrugation If crack runs from on corrugation to another, record all new crack data to correspond with a different corrugation number

Use a surveyor's chain for probing to determine an approximate crack depth.

A final inspection will show crack sizes, lengths, depths, and locations in each corrugation number on all sides of seal. Scale crack depths on the plan view for the report to show relation to footing.

D. Footings

<u>Spread Footings</u>.

1. Inspect for scour near footing upstream or adjacent to footing, measure size of scour (width x length x depth), and document location.

2. Inspect for scour or soft material under footing.

Survey perimeter of footing. Use probing rod and rule. Station footing from upstream end to downstream and 50 mm (2 in) increments. At these stations measure water depth, height from bottom of footing to mudline, and depth of scour from edge of footing to point under footing where bearing is established. Take photographs of bottom of footing showing scour at each station when possible. Measure from top of footing to waterline on upstream and downstream ends. Measure size of footing.

3. Inspect for condition concrete.

   Measure size of spalls (width x length x depth) and locations and sizes of cracks.

   Inspect for any exposed reinforcing steel and for soundness and appearance

4. Inspect footings keyed into rock.

   Inspect for separation at base of footing and rock foundation. This condition could indicate foundation or substructure movement. Inspect for voids between footing and rock foundations that could have been formed by trapped clay, silt, loose rock, or mud in concrete.

5. Inspect footings on seals for any separation at base of footing and seal.

Pile Footings.

1 Measure width, length, and height of footing if unknown.

  Measure size, number and spacing of piles under footing if unknown.

2 Inspect for scour at piling and record approximate depth.

3. Inspect condition of concrete at sides, top, and bottom of footing. Measure size of spalls (width x length x depth) and crack size locations.

4. Inspect footing

   If originally designed to be embedded in steam, this requires measurements of each exposed pile from bottom of footing to mudline. Take photographs of bottom of footing showing exposed pile when possible.

5. Inspect piles for soundness and section loss.

6. Inspect for drift lodged between pilings.

E. Piles

1. Two divers inspecting same pile.

   When poor visibility requires mask and light close to pile, divers can inspect on opposite sides of pile. After descending to mudline, divers can rotate to uninspected sides and ascend.

2. Divers inspecting piles adjacent to each other.

   Divers can choose to inspect one pile each as long as piles are adjacent. This will be more desirable in strong currents, when piles have areas to be cleaned, or in times of good visibility. Divers are to concentrate on inspecting faces of each pile.

Concrete Piles

1. Inspect condition of concrete.

Take photographs when possible. Measure width, length, and height of spalls.

Inspect for any exposed reinforcing steel or cables and for soundness and appearance.

Inspect for cracks and measure size, length, and location for future inspections. If a crack is spalling on each edge, record actual size which is measured deeper than the spalled surface Crack length should be measured deeper than the spalled surface Crack length should be measured from waterline to end of crack underwater. If crack extends above waterline and has not been previously recorded, measure and record. If a crack does not start at waterline, locate crack with reference to it. Bent number, pile number, and face number are required when recording location of cracks Direction of numbering bents is from south to north or from east to west, numbering of piles is from left to right and of pile faces is counterclockwise Take close-up photographs of cracks when possible.

2. Inspect for scour at base of piling and record approximate depth.
3. Inspect concrete when marine growth covers pile. Clean random areas of pile from waterline to mudline The number of areas will depend on condition of concrete, visibility, water depth, and type of growth. This should be determined in the field. Inspect those areas that are already clean, which in most cases are located at the mudline.

III. Report

A. Drawings

1. Elevations showing dimensions and scour, cracks, unstable conditions, etc.
2. Sections showing degree of scour, spalling, etc., in terms of mudline and waterline.
3. Plans showing inspection area, inspected section, spacing of piles and footings, areas of damage

B. Summary Report from Inspection Data

1. Describe general overall condition
2. Indicate best and worst conditions found

APPENDIX I

IN-SITU TESTING AND INSPECTION TECHNIQUES FOR CONCRETE

This information is provided for reader interest and has been reproduced from the following reference:

Campbell-Allen, D.; Roper H., Concrete Structures: Materials, Maintenance and Repair, Longman, U.K., 1991, 369 p.

Test method	Principal reference	Application	Property assessed	Remarks
Structural integrity and layout				
Visual survey including (a) crack mapping (b) endoscope survey (c) photography - video (d) stereo pairs	Bridge Inspection Panel (1984) ACI (1968) Manning and Bye (1983) Cement and Concrete Association (1988)	All elements and structures	Condition and monitoring Condition and monitoring	Simplest but most important inspection method Include scales and color charts as appropriate close ups and general views
Hammer testing Chain drags, etc.	Moore et al (1973) Savage (1985) Manning and Bye (1983) Cantor (1984)	All elements and structures	Presence of cracks, spalls and delaminations	Hammer test or chain drag may only detect surface laminations to 75-100 mm deep
Covermeter surveys to locate reinforcement, prestressing ties, etc.	BSI (1988b) BSI (1986a)	All elements and structures	Location and concrete cover Bar sizes also determined	Influenced by magnetic aggregates and difficulties with lapped or closely spaced bars or layers of bars, modern instruments much more effective at locating steel upto 300 mm deep
Thermography	Manning (1985) BSI (1986a) Manning and Holt (1980) Kunz and Eales (1985)	Principally bridge decks (asphalt and plain concrete) and other elements	Presence of laminations	Influenced by water on deck and prevailing weather, rapid scan system
Radar	Manning (1985) Cantor (1984) BSI (1986a) Kunz and Eales (1985)	Principally bridge decks	Voidage etc on large scale including reinforcemnt, ducts etc	Bulky equipment, signals can be difficult to interpret
Acoustic emission	Mkalat et al (1984) Hendry and Royles (1985) Manning (1985) BSI (1986a)	All structures during load testing	Determination of initiation and origin of cracks	Specialist inspection equipment and interpretation
Dynamic response e.g. sonic echo, continuous vibration	Stain (1982) Manning (1985) BSI (1985)	Principally pile testing	Pile integrity	Specialist inspection equipment and interpretation

Test method	Principal reference	Application	Property assessed	Remarks
Load testing of structures	BSI (1986c) Menzies (1978) Jones and Oliver (1978) ACI (1985a)	Structures/elements	Deflection uner loads against structural analysis	Expensive but informative
Determination of concrete quality and composition				
Core and lump samples	Concrete Society (1987) BSI (1981) ASTM (1987a)	All elements and structures	Used in physical, chemical and petrographic analysis of quality	Required for almost all laboratory testing; restricted be access to certain members; can be expensive
Power drilled samples	Building Research Establishment (1977)	All elements and structures	Chloride, sulfate and moisture content of concrete	Simple but subject to errors of depth/cross contamination and sampling.
Partially non-destructive assessment of strength i e. Windsor probe, internal fracture, pull-out testing, Schmidt Hammer, etc	BSI (in preparation) BSI (1981) Keiller (1982)	All elements and structures	Strength, usually converted to equivalent compressive	Varying in simplicity; often wide margin of error (15%-30%) depending on method and availabilty of calibration. Information on cover concrete only. Often operator and equipment sensitive Can damage concrete.
Ultrasonic pulse velocity testing	BSI (1986c) BSI (in preparation) ASTM (1987c)	All elements and structures	Concrete quality uniformity, presence of cracks, voids, weak layers, etc.	Rapid scan technique thourgh thickness of concrete; can be correlated with strength
In-situ permeability test, e g ISAT, Figg test, Clam	Concrete Society (1988) Lawrence (1981) Montgomery and Adams (1985)	All elements and structures	Concrete quality and permeability, generally restricted to cover	Can be difficult to use in-situ and slow, may only give a permeability index measurement, not true intrinsic permeability, and can be influenced by moisture content of concrete, surface condition, etc.

Test method	Principal reference	Application	Property assessed	Remarks
Steel serviceability and condition				
Half-cell potential mapping	ASTM (1987f) Figg and Marsden (1985) ASTM (1983) Baker (1986)	All elements and structures	Likelihood of corrosion of reinforcement (and possible rate)	Single-cell and two-cell methods used with copper, silver reference electrodes Requires in-situ calibration
Resistivity of cover	Manning (1985) Vassie (1980) Figg and Marsden (1985) Wenner (1915)	All elements and structures	Electrical resistance of cover concrete	Four-probe method used with either emedded or surface-contact electrodes Measurement of circuit resistance may also be valued

APPENDIX J

COMMON LABORATORY TESTS FOR CONCRETE

This information is provided for reader interest and has been reproduced from the following reference:

Bell, G.R., "Civil Engineering Investigation," Forensic Engineering, Kenneth L. Carper editor, Elsevier, 1989, pp 190-232.

Test	Reference (where applicable)
Compound or mockup load tests	
Wood trusses	ASTM E1080
Wall, floor, and roof panels	ASTM E72
Shear resistance of framed walls	ASTM E564
Window/wall assemblies	ASTM E330
Data reporting	ASTM E575
Beam flexural strength (see also Table 8.5)	ASTM E529
Concrete materials	
Cylinder compressive strength	ASTM C873, C39
Modulus of elasticity	ASTM C469, C215
Thermal expansion	ASTM C531
Bond strength	ASTM C234
Tensile strength	ASTM C496
Flexural strength	ASTM C192, C42, C1018, C293, C78
Diagonal shear strength	
Fatigue strength	
Fracture characteristics	
Petrographic analysis	ASTM C295
Air content	ASTM C457, C138, C231, C173
Chemical analysis of cement	ASTM C114
Cement content	ASTM C85
Alkali reactivity	ASTM C289
Abrasion resistance	ASTM C779, C944, C418
Absorption	ASTM C642
Density	ASTM C1040
Metal materials	
Tensile tests	ASTM E8
Charpy impact	ASTM E23, A370, E812, A673
Hardness	
Compressive testing	ASTM E9
Ductility	ASTM E290
Acoustic emission	ASTM E1139
Metallography	ASTM E807, E7, E112, E2, E883
Chemical tests	ASTM E60, A751
Corrosion	ASTM E937
Elongation	ASTM E8
Fatigue	ASTM E647, E812, E468, E467, E466, E1150

Test	Reference (where applicable)
Masonry materials	
Compressive strength of units	ASTM E447, C67
Prism strength	ASTM C349, E447
Flexural strength	ASTM C1072, C348, C67
Bond strength	ASTM E518, C952
Shrinkage	ASTM C426
Mortar strength	
Shear strength	ASTM E519
Thermal expansion	ASTM C531
Tensile strength	ASTM C1006
Water absorption	ASTM C67
Efflorescence	ASTM C67
Freeze-thaw resistance	ASTM C67
Petrography	
Mortar air content	ASTM C1072
Chemical resistance	ASTM C279
Wood materials	
Compression strength	ASTM D2555, D143
Flexural strength	ASTM D1037, D198
Shear strength	ASTM D1037, D198
Tensile strength, modulus of rupture	ASTM D2555
Creep	
Shrinkage	
Moisture content	
Durability of adhesives	ASTM D3434
Weld inspection (see Table 8.5)	
Subsurface tests and nondestructive weld testing (see Table 8.5)	
Model tests	
Structural load tests	Schreiver (1980)
Boundary layer wind tunnel tests	
Water and air penetration	
Window/wall air leakage	ASTM E 283
Window/wall water leakage	ASTM E 331, E 547, E1105, AAMA 501.3
Scanning electron microscopic examination	

APPENDIX K

TYPICAL RANGE OF DESIRABLE EPOXY RESIN PROPERTIES

This information is provided for reader interest and has been reproduced from the following references:

Bean, D.L., "Epoxy-Resin Grouting of Cracks in Concrete," Miscellaneous Paper SL-85-18, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, December 1985, 36 p.

Mendis, P., "Commerical Applications and Property Requirements for Epoxies in Construction," Repairs of Concrete Structures - Assessments, Methods and Risks, SCM-21(89), American Concrete Institute, Detroit, Michigan, 1989.

Gel time, pot life:	25 min minimum at 73°F	ASTM D 2471-66T
Compressive strength, 28 days:	8,000 psi minimum	ASTM D 695-80
Tensile strength:	4,000 psi minimum	ASTM D 695-80
Bond strength, 14 days	1,500-psi minimum	ASTM C 882-78
Specific gravity:	Greater than 1	
Viscosity:	500 cps maximum without filler	ASTM D 2393-68
Modulus of elasticity:	340,000 minimum	
Final cure (75 percent ultimate strength):	2 days at 73°F	
Color:	One component is white; the other is black; mixture of components is gray	
Weight per epoxide:	160-278	ASTM D 1652-67
Container size:	1-pt or 1-gal units	
Solvents or diluents:	Must be 100 percent reactive, no nonreactive materials.	
Safety data:	Must supply safety information with materials.	
Their typical properties are:		
a) Tensile Strength		2000 psi - 8000 psi
b) Tensile Elongation		1% - 35%
c) Compressive Strength		6000 psi - 14,000 psi
d) Low Creep inches/inch		4.05×10^{4}
f) Good dimensional stability low shrinkage		0.05 in/in °F
g) Versatility in adjusting the rheological properties of the grout		Viscosities can vary from 200 cps to gel consistency
h) Rapid Strength Development		14,000 psi in 24 hrs.
i) Resistance to long term aging		
j) Resistance to the alkaline effect of cement and concrete		

APPENDIX L

CONCRETE ADMIXTURES BY CLASSIFICATION

Kosmatka, S.H., Panarese, W.C., Design and Control of Concrete Mixtures, Portland Cement Association, Skokie, Illinois, 13th edition, 1988, 205.

Type of admixture	Desired effect	Material
Accelerators (ASTM C 494 Type C)	Accelerate setting and early strength development	Calcium chloride (ASTM D 98) Triethanolamine sodium thiocyanate calcium formate calcium nitrite calcium nitrate
Air detrainers	Decrease air content	Tributyl phosphate dibutyl phthalate octyl alcohol water-insoluble esters of carbonic and boric acid, silicones
Air entraining admixtures (ASTM C 260)	Improve durability in environments of freeze thaw, deicers, sulfate and alkali reactivity Improve workability	Salts of wood resins (Vinsol resin) Some synthetic detergents Salts of sulfonated lignin Salts of petroleum acids Salts of proteinaceous material Fatty and resinous acids and their salts Alkylbenzene sulfonates Salts of sulfonated hydrocarbons
Alkali-reactivity reducers	Reduce alkali reactivity expansion	Pozzolans (fly ash silica fume) blast-furnace slag, salts of lithium and barium, air-entraining agents
Bonding admixtures	Increase bond strength	Rubber polyvinyl chloride polyvinyl acetate acrylics butadiene-styrene copolymers
Coloring agents	Colored concrete	Modified carbon black iron oxide, phthalocyanine umber, chromium oxide, titanium oxide cobalt blue (ASTM C 979)
Corrosion inhibitors	Reduce steel corrosion activity in a chloride environment	Calcium nitrite, sodium nitrite sodium benzoate, certain phosphates or fluosilicates fluoaluminates
Dampproofing admixtures	Retard moisture penetration into dry concrete	Soaps of calcium or ammonium stearate or oleate Butyl stearate Petroleum products
Finely divided mineral admixtures		
Cementitious	Hydraulic properties Partial cement replacement	Ground granulated blast-furnace slag (ASTM C 989) Natural cement Hydraulic hydrated lime (ASTM C 141)
Pozzolans	Pozzolanic activity Improve workability, plasticity, sulfate resistance, reduce alkali reactivity, permeability, heat of hydration Partial cement replacement Filler	Diatomaceous earth, opaline cherts, clays, shales, volcanic tuffs, pumicites (ASTM C 618, Class N), fly ash (ASTM C 618, Classes F and C), silica fume
Pozzolanic and cementitious	Same as cementitious and pozzolan categories	High calcium fly ash (ASTM C 618, Class C) Ground granulated blast furnace slag (ASTM C 989)
Nominally inert	Improve workability Filler	Marble, dolomite, quartz, granite
Fungicides, germicides, and insecticides	Inhibit or control bacterial and fungal growth	Polyhalogenated phenols Dieldrin emulsions Copper compounds
Gas formers	Cause expansion before setting	Aluminum powder Resin soap and vegetable or animal glue Saponin Hydrolized protein
Grouting agents	Adjust grout properties for specific applications	See Air-entraining admixtures, Accelerators, Retarders, Workability agents
Permeability reducers	Decrease permeability	Silica fume Fly ash (ASTM C 618) Ground slag (ASTM C989) Natural pozzolans Water reducers Latex

Type of admixture	Desired effect	Material
Pumping aids	Improve pumpability	Organic and synthetic polymers Organic flocculents Organic emulsions of paraffin coal tar asphalt acrylics Bentonite and pyrogenic silicas Natural pozzolans (ASTM C 618 Class N) Fly ash (ASTM C 618 Classes F and C) Hydrated lime (ASTM C 141)
Retarders (ASTM C 494, Type B)	Retard setting time	Lignin Borax Sugars Tartaric acid and salts
Superplasticizers* (ASTM C 1017, Type 1)	Flowing concrete Reduce water-cement ratio	Sulfonated melamine formaldehyde condensates Sulfonated naphthalene formaldehyde condensates Lignosulfonates
Superplasticizer* and retarder (ASTM C 1017, Type 2)	Flowing concrete with retarded set Reduce water	See Superplasticizers and also Water reducers
Water reducer (ASTM C 494, Type A)	Reduce water demand at least 5%	Lignosulfonates Hydroxylated carboxylic acids Carbohydrates (Also tend to retard set so accelerator is often added)
Water reducer and accelerator (ASTM C 494 Type E)	Reduce water (minimum 5%) and accelerate set	See Water reducer Type A (Accelerator is added)
Water reducer and retarder (ASTM C 494, Type D)	Reduce water (minimum 5%) and retard set	See Water reducer Type A
Water reducer—high range (ASTM C 494, Type F)	Reduce water demand (minimum 12%)	See Superplasticizers
Water reducer—high range—and retarder (ASTM C 494, Type G)	Reduce water demand (minimum 12%) and retard set	See Superplasticizers and also Water reducers
Workability agents	Improve workability	Air entraining admixtures Finely divided admixtures, except silica fume Water reducers

*Superplasticizers are also referred to as high range water reducers or plasticizers These admixtures often meet both ASTM C 494 and C 1017 specifications simultaneously

APPENDIX M

SPECIFICATIONS FOR PILE WRAPPING SYSTEM (RETROWRAP)

This information forms part of a pile encapsulation specification system developed by CATHODIC SYSTEMS, INC., and is provided for reader interest.

RETROWRAP PILE ENCAPSULATION SYSTEM

SPECIFICATIONS

Part 1 - Scope of Work

1.1 The contractor shall furnish all labor, materials, tools and equipment necessary to install a Retrowrap and Pile Encapsulation System on piles as indicated on the drawings and specifications.

Part 2 - Materials

2.1 All materials for the pile encapsulation system shall be Retrowrap© Pile Encapsulation System.

2.2 Retrowrap shall meet the following specifications

2.2.1 Outer Geo-Membrane

Finish weight	6.5 OZ/YD	ASTM 5041
Tongue tear	150/150/LBS	ASTM 5134
Adhesion (minimum)	12 LBS/IN	ASTM 5970
Adhesion (wet)	80 LBS/FT	ASTM DO751
Adhesion (dry)	107 LBS/FT	ASTM DO751
Strip tensile	380-400 LBS/IN	ASTM 5012
Breaking load	2554 LBS/FT	
Tear resistance	1008 LBS/FT	

2.2.2 Pultruded Stiffener

Tensile strength	120000 PSI
Tensile modulus	6.5 x 10^6 PSI
Flexural strength	12000 PSI
Flexural modulus	6.5 x 10^6 PSI
Compressive strength	70000
Izod impact strength	40 LBS/IN
Water absorbtion	.25%
Density	0.74 LBS/IN^3

2.2.3 Inner Sealing Flap

Nominal density	56-57 lbs.ft^3	ASTM 1505-60T
Tensile yield stress	4200 PSI	ASTM D638-61T
Flexural strength	4800 PSI	ASTM D790-59T
Flexural modulus	1.5 x 10^5 PSI	ASTM D790-59T
Izod import strength	7' lbs/in notch	ASTM D256-56T
Tolling weight impact strength	15 FT/LBS	2mm Sample
Hardness (Rockwell)	R-89	ASTM D785-62
Hardness (Shore D)	71	ASTM 1706-61

2.2.4 Inner Geo-textile Membrane

Weight	10 OZ/YD^2	ASTM D3776
Thickness	100 MILS	ASTM D1777
Tensile strength	100 LBS	ASTM D1682
Puncture strength	160 LBS	ASTM D751
Balloon burst strength	450 PSI	ASTM D751
Co-efficient of permeability	0.30 CON/SEC	ASTM D4491
Trapizoid tear strength	145 LBS	ASTM D4553

2.2.5 Bonding Agent

Total solids:	20 + 1%
Viscosity:	3500-4000 cps
Specific gravity:	0.86
Flash Point:	-17°C
Heat reactivation: (both surfaces)	85°C Surface temperature

2.2.6 Thixotropic Gel

Specific gravity (water =1)	1.0
Boiling point/range (c)	> 200
Freezing/melting point (c)	d.p. 86
Vapor density at 20 c	< 0.01 mm Hg

Miscibility with water	Immiscible	
Evaporation rate	(Butyl acetate = 1)	

2.2.7. ___specified___ Additive

2.2.8. Cable Ties

Class 1 type 1	Mils	23190E
Tensile strength	250 LBS	
Melting point	264°C	
Water absorbtion	1.3%	ASTM D570
Brittleness	-85°F	
Temperature	-85°F 2.5% water content	

2.3 Delivery, Storage and Handling

2.3.1. Deliver all materials to the job site in unopened packages bearing the manufacturer's name

2.3.2. All materials shall be stored in a protected area at the site until ready for use.

2.4 Submittals

2.4.1. Submit catalog cuts of all materials proposed to be furnished.

2.4.2. Submit to the Engineer six copies of manufacturer's printed scheduled start of work.

Part 3 - Surface Preparation

3.1 For steel piles, insure that the substrate has been prepared in accordance with the standards of the Steel Structures Painting Council (SSPC)

3.1.1. SSPC-SP2 Hand tool cleaning

3.1.2. SSPC-SP3 Power tool cleaning

3.2 For concrete piles, this cleaning specification merely calls for minimal surface preparation that removes loose material and marine growth to provide a sound surface.

Part 4 - Installation Procedure

4.1 Insure that the pile surface within the area to receive the Retrowrap has been properly cleaned.

4.2 Attach the polypropylene inner sealing flap to the pile insuring that it runs parallel to the axis of the pile and apply pressure such that the flap adheres to the surface.

4.3 With the cable ties in position around one stiffener, attach the wrap to the pile insuring that the stiffeners run parallel to the inner flap and that the central axis of the flap is equidistant from both stiffeners.

4.4 Feed the leading edge of all cable ties through the vacant holes of the adjacent stiffener and secure in the normal way.

4.5 Recheck the alignment of both wrap and flap to insure that they are in the desired position and correct orientation.

4.6 Place calipers across the stiffeners and sequentially in stages to insure that the stiffeners are brought together in parallel over the inner flap.

4.7 Tighten all calipers across the stiffeners and sequentially in stages to insure that the stiffeners are brought together in parallel over the inner flap.

4.7.1. While Retrowrap units are designed so that the specified tensions may be achieved by use of hand tools only, the installation of the units may be facilitated by use of air powered wrenches. Additional care should be taken when using air powered wrenches due to the speed at which they operate, to insure that no individual bolt is drawn too quickly and that the stiffeners are pulled together in parallel.

4.8 Continue to tighten calipers until maximum travel has been achieved. Pull all cable ties until tight and secure.

4.9 Remove all caliper units and cut off ends of cable ties.

Part 5 - Quality Assurance

5.1 The Contractor shall provide and pay for the services of a qualified technical representative of the manufacturer to supervise the installation of the pile wrapping system.

5.2 The representative shall be completely competent in all respects with the material and all equipment necessary to install it properly. The representative shall be responsible to:

5.2.1. Be present until such time that the contractor is knowledgeable and comfortable with all phases of the installation.

5.2.2. Advise the Engineer and the Contractor that the correct installation method is being followed.

5.2.3. Certify to the Engineer that all materials being used are in accordance with the company's requirements.

5.2.4. Train assigned personnel in the correct methods of installation.

5.2.5. Certify to the Engineer that the material has been installed correctly, after installation procedures have been completed.