

Engineering Report

SAER-5803

31 December 2001

Concrete Repair Manual

Document Responsibility: Consulting Services Dept./CEU

Saudi Aramco DeskTop Standards

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BACKGROUND OF THE PROBLEM

The Arabian Gulf region's climate, which is characterized by high temperature and humidity conditions and large fluctuations in the diurnal and seasonal temperature and humidity, adversely affects concrete durability in the region. The temperature can vary by as much as 20 °C during summer and the relative humidity ranges from 40 to 100% over 24 hours. These sudden and continuous variations in temperature and humidity initiate ever present cycles of expansion/contraction and hydration/dehydration which cause damage due to thermal and mechanical stresses. The damage to concrete due to these stresses is reflected by micro cracking and enhanced permeability, which results in a tremendous increase in the diffusion of aggressive species, such as chloride, oxygen, carbon dioxide and moisture, towards the steel-concrete interface. The changes in the diurnal and seasonal temperatures cause continuous thermal expansion and contraction cycles that may lead to the cracking of concrete. These expansion-contraction cycles become all the more damaging due to the thermal incompatibility of concrete components. The differential expansion and contraction movements of the aggregate material and hardened cement paste may set up tensile stresses far beyond the tensile capacity of concrete resulting in microcracking. Limestone, the predominantly used aggregate in this region, has a coefficient of thermal expansion of $1 \times 10^{-6}/^{\circ}\text{C}$. The coefficient of expansion for hardened cement paste is much higher (usually between 10×10^{-6} and $20 \times 10^{-6}/^{\circ}\text{C}$). With the fall in temperature, tensile and compressive stresses are set up in the cement paste and the aggregates, respectively. With the rise in temperature, the stresses are not exactly reversed but tensile stresses are set up at the aggregate-paste interface tending to cause interface bond failure and significant microcracking around the transition zone.

The other factor that contributes to the poor durability performance of concrete is the quality of local aggregates. Most of the aggregate available in the region is crushed limestone that is of marginal quality because it is porous, absorptive, relatively soft, and excessively dusty on crushing. These drawbacks are attributable to the source material, which comprises poor quality Tertiary age dolomitic limestone. The aeolian dune sands in the coastal areas form the main source of fine aggregate. These sands are essentially fine grained and have narrow grading. Nearly all the material passes No. 30 sieve and an appreciable portion, 10 to 20% passes # 100 sieve. The grains are not angular. Furthermore, the fine and the coarse aggregates are characterized by excessive dust content. Dust and excessive fines cause high water demand resulting in lower strength and greater shrinkage of concrete. Dust also forms a fine interstitial coating between the aggregate and the cement paste thereby weakening the bond at the aggregate-paste interface. This transition zone, being the weakest link of concrete composite, may further lower its strength and quality.

Concrete construction in the coastal areas of the Arabian Gulf is continually exposed to a ground and an atmosphere contaminated with salts. Aided by capillary action and high humidity conditions, the salt-contaminated groundwater and the salt-laden airborne moisture and dew find an easy ingress into the concrete matrix. Further, the salts also pollute the mix water and the aggregates thereby increasing the total salt content in the concrete. In this region, sulfates and chlorides occur at several horizons in the geological formations.

Reduction in the useful service-life of reinforced concrete construction is a major problem confronting the construction industry world wide, in general, and the Arabian Gulf, in particular. Deterioration of reinforced components is aggravated by the area's environmental conditions, high temperature and humidity. Saudi Aramco is faced with a similar problem as reinforced concrete structures in the industrial facilities exhibit signs of deterioration much earlier than their planned design life. In addition to environmental conditions, concrete structures in Saudi Aramco's industrial facilities are required to serve in aggressive environments, such as exposure to acid spillage, molten sulfur, etc.

Repair and rehabilitation of deteriorated concrete structures are essential not only to utilize them for their intended service-life but also to assure the safety and serviceability of the associated components. A good repair improves the function and performance of the structure, restores and increases its strength and stiffness, enhances the appearance of the concrete surface, provides water tightness, prevents ingress of the aggressive species to the steel interface, and improves its durability.

Several repair materials are marketed for this purpose. These repair materials are classified into different types, such as cement, epoxy resins, polyester resins, polymer latex, and polyvinyl acetate. Cement-based materials and polymer/epoxy resins are the most widely used among the repair materials. These materials mostly consist of a conventional cement mortar often incorporating special water-proofing admixtures. These admixtures are commonly impregnated with one or more additives, such as polymer, silica fume, fly ash or some other industrial by-products.

Polymer modified cement repair materials are used to overcome the problems associated with the cement-based repair materials, particularly the need for longer curing. Over the years, many different polymers have been used in a range of applications in the repair and maintenance of buildings and other structures. Such polymer mortars provide the same alkaline passivation protection to the steel, as do conventional cement materials. Polymers are usually used as admixtures; they are supplied as milky white dispersions in water and in that state are used either as a whole or as partial replacement of the mixing water. The polymer also serves as a water-reducing plasticizer, which produces a mortar with a good workability and lower shrinkage at lower water-to-cement ratios. Polyvinyl acetates (PVA), styrene butadiene rubber (SBR) and polyvinyl dichlorides (PVDC) are some of the polymers commonly used in the cement mortars. A recent development in the field of polymers are redispersible spray-dried polymer powders, which may be factory blended with graded sand, cement, and other additives to produce mortars and bonding coats simply by adding water on site.

While several repair materials, both cement- and polymer/resin-based, are used in the repair and rehabilitation of deteriorated concrete structures world-wide, their performance in the Arabian Gulf environment, extreme temperature and aridity, has not been thoroughly investigated.

Moreover, in the initial stages of casting a repair layer over a hardened concrete substrate, stresses resulting from restrained shrinkage commonly cause tensile cracking through the repair layer and/or delamination at the interface of the repair layer and the substrate. Loss of integrity in the early stages in the repair systems is primarily due to stresses resulting from restrained shrinkage.

A properly designed repair system may survive the initial onslaught of drying shrinkage, but would then be subjected to fluctuations in temperature, resulting in alternating cycles of expansion and contraction, which are known to induce micro-cracking at the interface of the aggregate and paste in a hardened concrete. At latter stages, the repair system is subjected to thermal cycling, resulting in alternating cycles of expansion and contraction. This may lead to internal microcracking in the repair layer due to differences in the coefficients of thermal expansion or to delamination at the interface of the repair layer and the substrate.

To minimize rehabilitation costs and increase service life of repaired structures, Saudi Aramco as part of its Technology Program conducted a study under item CSD-01/94-T at KFUPM. This concrete repair manual contains all the study results.

As part of the above study, tests were conducted on the individual repair components, such as repair mortars, bond coat materials, steel primers, and surface coatings, to evaluate their physical properties and durability characteristics. These results were compared with the manufacturer's data. This comparison indicates that the manufacturers provide very minimal data on either the physical properties or durability characteristics of repair components. Most of the data pertain to the strength characteristics of the repair mortars, and almost no data are provided on the properties of other repair components, such as bond coat materials, steel primers, and surface coatings. The meager data provided by the manufacturers corroborate well with the results of tests conducted in this study.

More than one repair material, under each category, was tested to generate data on the relative performance of repair components. This relative performance was utilized to select the repair components for full-scale evaluation as complete repair systems. Selection of these materials was based on their performance relative to the other material of similar generic type.

OBJECTIVES

This manual is intended to assist engineers in planning repair and rehabilitation of deteriorating concrete structures in Saudi Aramco's industrial facilities. The subject matter of the report is divided into seven chapters. In Chapter 1, commonly noted concrete deterioration problems are described along with the causes for such problems. Photographs showing the various forms of concrete deterioration are shown in Table 1.1. This will assist an engineer in identifying the nature of concrete deterioration.

The nature and extent of concrete deterioration should be assessed by conducting field and laboratory investigations. The procedures for planning field and laboratory investigations, to evaluate the cause and extent of concrete deterioration, are elucidated in Chapter 2.

After the cause and extent of concrete deterioration is known a strategy for repair of a deteriorated concrete structure is to be planned. Guidance on the selection of a suitable repair strategy is provided in Chapter 3.

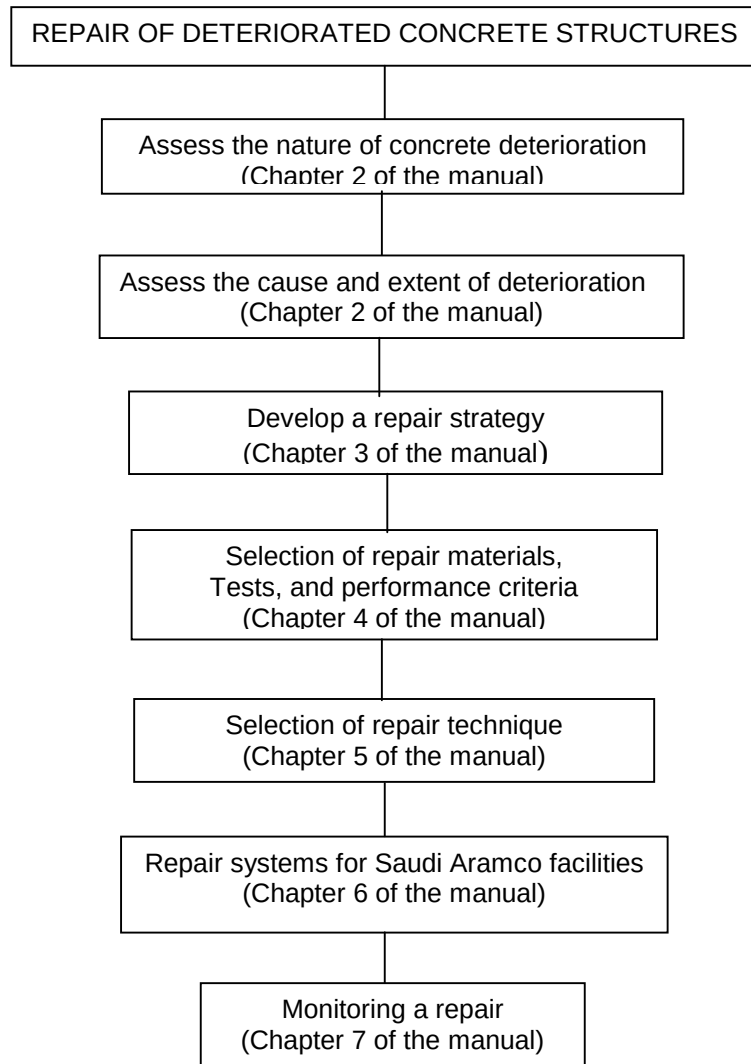
The materials that are commonly utilized for the repair of deteriorated concrete structures are described in Chapter 4. The tests to be conducted to evaluate the physical properties and durability characteristics of repair materials are also elucidated in this chapter. The performance criteria that the repair materials should conform to are also provided in this chapter.

The procedures for repairing deteriorated concrete structures are described in Chapter 5, while repair systems suitable for repairing concrete structures exposed to the industrial environment in Saudi Aramco are described in Chapter 6.

The procedures for monitoring the performance of a repair are described in Chapter 7.

The step-by-step procedure for using this repair manual is shown in the flow chart provided on the following page.

GUIDELINES FOR USING THE REPAIR MANUAL



SUMMARY

This repair manual consists of seven chapters dealing mainly with the concrete durability problem, methodology for assessment of the cause and extent of the deterioration, repair material, repair of the deteriorated concrete structures, and a strategy for monitoring the repaired structures. The topics covered under each chapter of this repair manual are discussed in the following paragraphs.

Chapter 1 details the commonly occurring concrete deterioration problems. The topics covered in this chapter comprise the following:

- i. Background to the problem of concrete durability.
- ii. Reinforcement corrosion.
- iii. Carbonation of concrete.
- iv. Sulfate attack.
- v. Salt weathering.
- vi. Alkali-aggregate reaction.
- vii. Cracking of concrete due to environmental factors and loading.
- viii. Acid attack.
- ix. Damage due to fire.
- x. Damage due to microbial organisms.

In Chapter 2, methodology for assessing the causes and extent of concrete deterioration has been presented. It is emphasized that a well-planned investigation is essential for planning an efficient repair. The topics covered under this chapter comprise the following:

- i. Evaluation program.
- ii. Preparation prior to preliminary inspection.
- iii. Preliminary investigation.
- iv. Detailed investigation.
- v. Commonly used test methods in assessing concrete deterioration.

Guidelines for selecting a repair strategy, after assessing the cause and extent of deterioration, are provided in Chapter 3. Since deterioration of reinforced concrete structures, in the Eastern Province of Saudi Arabia and many of Saudi Aramco facilities, is mainly due to reinforcement corrosion, a strategy for repair of these types of structures is also described in this chapter.

Chapter 4 describes the repair materials that are utilized for the repair of the deteriorated concrete structures. Procedures for evaluating the properties of the repair materials and

the performance criteria, suitable for use under local environmental conditions, are also presented for assessing the suitability of the selected repair materials. The topics covered under this chapter are the following:

- i. Repair mortars and concrete.
- ii. Injection grouts.
- iii. Bond coat materials.
- iv. Steel primers.
- v. Surface coatings.
- vi. Testing of repair materials.
- vii. Performance criteria.

The procedures for repairing deteriorated reinforced concrete structures are described in Chapter 5. The topics covered under this chapter comprise the following:

- i. Repair of cracked and deteriorated concrete.
- ii. Repair of concrete exposed to chemicals.
- iii. Repair of spalled concrete.
- iv. Other repair procedures.

In Chapter 6, repair systems most appropriate for repairing concrete structures commonly exposed to the environmental conditions in Saudi Armco's facilities are described. The repair systems for the following exposure conditions are presented:

- i. Marine.
- ii. Below ground.
- iii. Sulfur fumes.
- iv. Acid.
- v. Water under pressure and subject to thermal variations.
- vi. Fire damage.

Procedures for repairing concrete structures exposed to the above conditions are also presented.

In Chapter 7 methodology for monitoring the performance of repairs is described.

CHAPTER 1
A GUIDE TO FORMS OF CONCRETE DETERIORATION

1.1 FORMS OF CONCRETE DETERIORATION

Concrete deterioration is associated with the reaction of the concrete ingredients, cement in particular with the exposure conditions. While the deterioration of highway structures in North America and Europe is attributed to the use of deicer salts, the deterioration of concrete structures in the Arabian Gulf is caused by (i) severe climatic and geomorphic conditions, (ii) incorrect materials specifications, and (iii) defective construction practices.

The main causal factors for concrete deterioration, in decreasing order of importance are the following:

- (i) Corrosion of reinforcement,
- (ii) Sulfate attack and salt weathering, and
- (iii) Cracking due to environmental factors.

Concrete deterioration due to other factors, namely alkali-silica reactivity and carbonation, may occur, but due to the predominant nature of the distresses due to the first two factors, reinforcement corrosion in particular, these deterioration go largely unnoticed.

In Saudi Aramco industrial facilities concrete deterioration is also noticed due to acid spillage, sulfur exposure, fire and microorganisms.

The salient features of the concrete deterioration due to the aforesaid causes will be discussed in the following sections.

1.2 CONCRETE DETERIORATION DUE TO REINFORCEMENT CORROSION

Portland cement concrete provides both chemical and physical protection to the reinforcing steel. The chemical protection is provided by the highly alkaline nature of the pore solution ($\text{pH} > 13$). At this pH, steel is passivated in the presence of oxygen, presumably due to the formation of a submicroscopically thin $\gamma\text{-Fe}_2\text{O}_3$ film. This layer is thought to screen most of the surface of the steel from direct access of aggressive ions and to act as an alkaline buffer to pH reductions. The physical protection to steel is provided by the dense and impermeable structure of concrete that reduces the diffusion of aggressive species, such as chlorides, carbon dioxide, oxygen, and moisture, to the steel surface.

Depassivation of steel occurs by the reduction of the pore solution pH, due to carbonation, or by diffusion of chloride ions to the steel surface.

The flow of electrons depends on the potential difference in concrete that may be provided by differences in the metallurgical properties of steel, variation in air, moisture and ionic concentration in concrete or variations in the properties of concrete.

Given sufficient oxygen, this product can be further oxidized to form insoluble hydrated red rust. Depending on the availability of the reactants various oxides of iron are formed. The volume of the rust product may be 2 to 14 times that of the parent iron from which it is formed. Due to an increase in the volume, the corrosion product exerts tensile stresses

of the order of 4000 psi, which is 10 times the tensile strength of concrete. This excessive pressure causes the concrete cover to crack and may eventually spall off at an advanced stage of the corrosion process leading to a reduction in the cross-section of a structural member.

Due to the importance of chloride ions on reinforcement corrosion, almost all the standards lay down limits on the allowable chloride concentration. For example, ACI 318 allows a water soluble chloride ion concentration of 0.15% while BS 8110 allows a total chloride concentration of 0.4% by weight of cement.

Temperature and the presence of sulfate ions are the other factors that affect the rate of reinforcement corrosion.

1.3 CARBONATION

Carbonation of concrete normally involves a chemical reaction between atmospheric carbon dioxide and the products of cement hydration. This reaction results in a significant reduction in the pH of the pore solution due to removal of hydroxyl ions. Once the pH of the pore solution is reduced, due to carbonation, the reinforcing steel is depassivated leading to its corrosion if moisture and oxygen are available. Factors influencing carbonation of concrete include, concrete mix design, curing, moisture condition, and temperature.

Though deterioration of concrete structures in the Arabian Gulf is primarily attributed to chloride-induced reinforcement corrosion, carbonation of concrete to an advanced stage, sometimes to the rebar level, has been noted. This is particularly true as the environmental conditions in the Arabian Gulf are marked by elevated ambient temperatures (40 °C and above) and relative humidity (50 to 60%). These temperature and humidity regimes are particularly suitable to accelerate the carbonation process. Presence of chloride and sulfate salts also accelerates carbonation of concrete. Therefore, it is advisable to minimize the chloride and sulfate contamination to avoid carbonation of concrete. Another method of preventing carbonation of concrete, particularly in the industrial environments, is to coat the structures with an anti-carbonation coating.

1.4 SULFATE ATTACK

Deterioration of concrete due to the chemical reaction between the hydrated Portland cement and sulfate ions is known to occur in two forms depending on the concentration and the source of sulfate ions (the associated cation) and the composition of cement paste in concrete. Sulfate attack normally manifests in the form of expansion of concrete leading to its cracking. It can also take the form of a progressive loss of strength and mass due to deterioration in the cohesiveness of the cement hydration products.

Among the hydration products, calcium hydroxide and alumina-bearing phases are more vulnerable to attack by sulfate ions. On hydration, Portland cements with more than 5% tricalcium aluminate (C_3A) will contain most of the alumina in the form of monosulfate hydrate, $C_3A.C\bar{S}.H_{18}$. If the C_3A content of the cement is more than 8%, the hydration products will also contain $C_3A.CH.H_{18}$. In the presence of calcium hydroxide both the

alumina-containing hydrates are converted to ettringite ($C_3A.3\bar{C}\bar{S}.H_{32}$), when the cement paste comes in contact with sulfate ions,

The formation of ettringite generates excessive expansion in concrete leading to its cracking.

Depending on the cation type present in the sulfate solution (i.e., Na^+ or Mg^{++}) calcium hydroxide and calcium silicate hydrate (C-S-H) may be converted to gypsum by sulfate attack.

In the presence of sodium sulfate, formation of sodium hydroxide as a by-product of reaction ensures the continuation of high alkalinity in the system, which is essential for stability of the main cementitious phase (C-S-H). However, in the event of magnesium sulfate attack, conversion of calcium hydroxide to gypsum is accompanied by the formation of relatively insoluble and poorly alkaline magnesium hydroxide; thus the stability of the C-S-H in the system is reduced and it is also attacked by the sulfate solution. The magnesium sulfate attack is, therefore, more severe on concrete compared to that of sodium sulfate.

Further, gypsum can react with calcium carbonate, a product of carbonation in cement, to form thaumasite ($CaCO_3.CaSiO_3.CaSO_4.15H_2O$). The formation of thaumasite results in a very severe damaging effect that is able to transform hardened concrete into a pulpy mass. Thaumasite formation is favored by high relative humidities (more than 90%) and temperature in the range of 0 to 10 °C.

Preventive measures to mitigate sulfate attack include the following (i) minimizing the sulfate contamination of the constituent materials, (ii) using a dense and impermeable concrete through the use of low water-cement ratio, (iii) use of a cement type compatible with the service environment, and (iv) coating the below ground components with a epoxy-based coating.

1.5 SALT WEATHERING

This type of deterioration of concrete is more of a physical nature than a chemical reaction. Deterioration of concrete due to salt weathering is evident on the components just above the grade level and also in situations where the salt is deposited from the environment on the exposed surface. It is characterized by progressive crumbling or scaling that erodes the surface of concrete leaving the aggregates exposed. Permeable concrete in contact with salt bearing soil or groundwater absorbs groundwater containing soluble salts. If water can evaporate from any surface of the concrete, additional ground water will be drawn into the concrete by capillary action. At the evaporation face, water will be lost, but the salts will remain on the concrete surface. Highly permeable concrete allows more water through the pores; and therefore, a rapid build-up of salts on the concrete surface. Once the salt solution reaches the saturated or supersaturated level, then salt crystals will be precipitated on the evaporating face. Repeated crystallization cycles, caused primarily by the night and day thermal changes, and secondarily by relative humidity changes produce the destructive action on the concrete. The damage is concentrated at the evaporation face where a thin layer of concrete is crumbled or scaled. The eroding process continues as long as the environmental changes of temperature and humidity cause cycles of salt crystallization. Two sodium salts are known to cause salt

crystallization damage to concrete; they are sodium sulfate and sodium carbonate. Both salts are commonly found in soils and are highly soluble in ground water.

Salt weathering may also be a major problem in the marine environments, particularly in the tidal zones, where concrete deterioration is aided by both the deposition of salts and their dissolution due to the cyclic action of wetting and drying. This phenomenon may also be attributed to the material properties. For example, concretes incorporating pozzolanic materials and low water cement ratio generate a very fine pore structure that cannot accommodate the salt crystals. These salt crystals exert considerable pressure, resulting in greater expansion and deterioration of concrete. Deterioration of concrete skin, scaling, is also more prominently noted in the silica fume and blast furnace slag cement concretes exposed to concentrated salt environments.

1.6 ALKALI AGGREGATE REACTION

This type of concrete deterioration is mainly attributed to the reaction of certain minerals in aggregates with alkalis in the concrete. These types of reactions are attributed to a swelling pressure developing as a result of the reactivity within the fabric of the concrete, which is sufficient to produce and propagate micro-fractures. Such deterioration is mainly attributed to alkali-carbonate reaction, alkali-silicate reaction, and alkali-silica reaction.

The alkali-carbonate reaction manifests in the following three forms:

1. The reaction between the carbonate fraction in the calcitic limestone and the alkaline material in the cement is manifested in the form of dark reaction rims that develop within the margin of the limestone aggregate particles. These rims are more soluble in hydrochloric acid than the interior of the particle.
2. Reactions involving dolomitic limestone aggregates are characterized by distinct reaction rims within the aggregate. Etching with hydrochloric acid shows that both rim zones and the interior of the particles dissolve at the same rate.
3. The reaction of fine-grained dolomitic limestone aggregate with alkalis produces a distinct dedolomitized rim. This type of reaction appears to be the only type that produces a significant expansion. The cause is not properly understood at present, but it is suggested that the dedolomitization of the crystals in the aggregate particles open channels, allowing moisture to be absorbed on previously dry clay surfaces. The swelling caused by this absorption causes irreversible expansion of the rock and subsequent expansion and cracking of the concrete. The reaction process is essentially one of dedolomitization, together with the production of brucite ($\text{Mg}(\text{OH})_2$), and regeneration of alkali hydroxide.

A second group of reactions reported in concrete is referred to as alkali-silicate reaction. These reactions appear to occur in alkali-rich concretes that contain argillite and greywack minerals in the aggregate. The reaction of these minerals with alkalis is generally slow and is not completely understood.

Silica mineral constituents in the aggregates appear to expand and cause disruption of concrete. The expansion of individual rock particles suggests absorption of water on previously dry aluminosilicate surfaces.

The third and the most common reaction between alkali hydroxides and siliceous material in the concrete are usually referred to as alkali-silica reaction (ASR). The alkaline concrete pore solution reacts with silica-containing aggregates leading to a destructive expansion. Visible damage due to this phenomenon manifests in the form of small surface cracks in an irregular pattern (map cracking) followed eventually by complete disintegration. General expansion develops in the direction of least resistance, giving parallel surface crack patterns developing inward from the surface (for slabs), or cracking parallel to compression trajectories for compressed members (for columns or prestressed members). Other typical manifestations are pop-outs and weeping of glassy pearls. However, this reaction progresses slowly so that it is usually some years before expansion and damage to the structure becomes apparent.

1.7 CRACKING OF CONCRETE

Concrete undergoes cracking when the tensile stress generated in the concrete due to various physical and chemical processes exceeds its tensile stress capacity.

The cracks resulting from stresses induced in concrete, due to physical deformations, and surpassing the tensile strength capacity of concrete can be grouped into following types:

Type I: Cracks resulting from physical actions in young concrete.

- (a) Plastic shrinkage
- (b) Plastic settlement
- (c) Subgrade deformation
- (d) Formwork movement

Type II: Cracks resulting from physical processes in hardened concrete.

- (a) Drying shrinkage
- (b) Creep

Type III: Cracks resulting from environmental actions on hardened concrete.

- (a) Thermal cycles
- (b) Freeze/Thaw cycles

Type IV: Cracks resulting from chemical processes in concrete.

- (a) Dissipation of heat of hydration
- (b) Alkali-aggregate reaction

- (c) Carbonation
- (d) Sulfate attack
- (e) Chemical shrinkage

Type V: Cracks resulting from chemical processes in embedded material.

- (a) Corrosion of reinforcing steel

Type VI: Cracks resulting from externally applied actions.

- (a) Design loads
- (b) Accidental overloads
- (c) Differential settlement of foundations.

The causes of cracks stated above are described in the following subsections.

1.7.1 Plastic Shrinkage and Settlement Cracks

Some cracks may develop in concrete structures when the concrete is in a state of transition from the green concrete state to young concrete. During this period, concrete has a very low tensile strength. Cracks in this state can result from plastic shrinkage, plastic settlement, subgrade deformation, and formwork movement.

PLASTIC SHRINKAGE CRACKS

Plastic shrinkage cracks result from moisture transport within a short time of concrete placement. They are associated with the bleeding of the concrete and are caused by capillary tension in pore water. It occurs most commonly in slabs and structures, which have high surface areas. The time of appearance of these cracks is within the first six hours after placement of concrete.

The plastic shrinkage cracks on slabs occur typically near the corner. They are oriented at angle of 45° and are parallel, the crack spacing being irregular. Map cracking can also result due to plastic shrinkage of concrete. The plastic shrinkage cracks are of the order of 2 to 3 mm on the surface and in some cases they extend through the depth of the slab.

PLASTIC SETTLEMENT CRACKS

After pouring, the concrete tends to settle in the formwork due to gravitational forces. The mix water simultaneously moves towards the surface. If the movement of concrete is restrained by reinforcement, plastic settlement cracks develop at the locations where such movements are restrained.

Plastic settlement cracks occur in deep beams, thick slabs, foundations and mat slabs. These cracks are either longitudinal cracks following the direction of the main

reinforcement or transverse reinforcement such as stirrups in columns and shear reinforcements in beams.

1.7.2 Drying Shrinkage and Creep Cracks

These cracks are associated with long-term physical processes occurring in hardened concrete and are principally associated with the movement of the moisture in concrete. They appear in structures after several weeks or months after the casting of concrete. Drying shrinkage is a load independent long-term deformation of concrete that occurs due to transport of moisture from the body of the concrete to the surface and into the ambience. Creep on the other hand is a load dependent long-term deformation that occurs under sustained stresses on the structure.

1.7.3 Cracks Due to Thermal Cycles

Temperature differences within a reinforced concrete member can result in cracking of the structure. The diurnal non-linear variation in the environmental temperature can result in temperature variation in concrete elements like bridge decks and pavements. This results in changing the length of the element and causing its bending.

1.7.4 Cracks due to Chemical Processes in Concrete

Several chemical processes occur in young and hardened concrete that may lead to cracking of concrete. At early ages, hydration of cement results in generation of heat in the concrete. The cooling of concrete members results in cracking of concrete. The depletion of moisture due to hydration reaction results in chemical shrinkage of concrete element. Some long-term chemical processes, such as alkali-aggregate reaction and carbonation, also result in cracking in concrete.

CRACKS DUE TO HEAT DISSIPATION

The heat of hydration of cement is generated during the setting and hardening of concrete. In massive concrete elements, the heat generated remains entrapped resulting in a temperature gradient from the surface of the concrete to the core which is at a higher temperature. The temperature gradient results in tensile stresses on the surface and compressive stresses in the core. A map cracking results if the tensile stresses exceed the still low tensile strength of the hardening concrete. These cracks are formed within the first two or three weeks after casting of the concrete. They are usually a few millimeter or centimeter in depth and usually close up when the temperature differences vanish. These cracks are, however, permanent and are visible once the surface is wetted.

CRACKS DUE TO EXTERNAL LOADS AND DEFORMATIONS

Application of loads results in the development of flexural, shear, compressive, torsional, bond and tensile stresses in a structural member. If the computations for the ultimate limit-state are erroneous and the members are under designed, load-induced cracks develop in the structural member. Cracking may result from overstressing of the concrete locally. Common cases of cracking are excessive bond stresses leading to cracking along

the line of the bar and cracking due to concentrated loads, such as beneath anchorage of pre-stressing tendons, leading to cracks parallel to the direction of applied compression.

The settlement of subgrade and foundations also results in the development of cracks in concrete. A differential settlement of a foundation causes cracks in structural members. These cracks are similar to the load induced cracks. The settlement of subgrade generally results in cracking in non-structural elements like partitions, in-fill panels, windows, and doors.

1.8 CONCRETE DETERIORATION DUE TO ACID ATTACK

Concrete is an alkaline composite material composed of coarse and fine aggregates embedded in hydrated cement paste. Therefore, it is very susceptible to attack by acidic materials. The mechanism of acid attack on concrete involves the reaction between the acidic solution and the calcium hydroxide of the hydrated Portland cement producing either water-soluble or insoluble calcium compounds.

In case of high concentrations of acidic solution, calcium silicate hydrate (C-S-H) may also be attacked forming silica gel. Depending on the type of the anion associated with the hydrogen ion, the reaction product between the acidic solution and the concrete will be either soluble or insoluble calcium salt.

The formation of soluble calcium salts due to acidic solution is frequently encountered in industrial environments. For example, hydrochloric, sulfuric, or nitric acids may be present in the chemical industry. Acetic, formic and lactic acids are found in many food processing industries. Carbonic acid is present in soft drinks. Water with highly dissolved free carbon dioxide can be harmful to concrete. Some mineral waters with high concentrations of either carbon dioxide or hydrogen sulfide, or both, can seriously damage the concrete.

All the acids mentioned above form soluble calcium salts that severely damage the structure of concrete. Other acids that form insoluble and non-expansive calcium salts when they come in contact with concrete include oxalic, phosphoric and hydrofluoric acid.

1.9 CONCRETE DAMAGE DUE TO FIRE

Fire introduces high temperature gradients. As a result of these high temperature gradients, hot surface layers tend to separate and spall from the cooler interior of the concrete body. Cracks, then, tend to form at joints, in poorly consolidated parts of the concrete, or in planes of reinforcing steel bars. Once the reinforcement has become exposed, it conducts heat and accelerates the action of heat.

Factors that tend to promote spalling are high moisture content, restraint to expansion (e.g., panels within a frame), low porosity and low permeability, closely spaced reinforcement, and rapid temperature rise. Spalling can also result from differential expansion of the mix constituents. Another common cause of spalling is the rapid quenching of hot fires by fire hoses. Rapid quenching of fire can cause serious structural damage.

A key factor in the amount of damage that is caused to concrete is the duration of the fire. Because of the low thermal conductivity of concrete, it takes a considerable time for the interior of concrete to reach damaging temperatures. For instance, damage commonly does not extend to more than about 10 to 30 mm below the surface of the concrete.

1.10 EFFECT OF MICROORGANISMS ON CONCRETE DETERIORATION

When anaerobic conditions occur in soils, water, and sewage in the presence of sulfate, sulfur-reducing bacteria, *Desulfovibrio* and related bacteria, will produce hydrogen sulfide. The durability of concrete structures can thus be adversely affected in such environments. As hydrogen sulfide is released, various populations of sulfur-oxidizing bacteria, known as the Thiobacilli, will proliferate. The proliferation of these organisms results in a decrease in the pH due to the production of sulfuric acid. Different Thiobacilli will be present depending on the pH of the environment. The actual events leading to sulfur-based microbial deterioration of concrete structures involve several groups of microorganisms operating in a cascade. The initial phase of the aerobic oxidation of sulfur in the environment near the concrete structure involves organisms that grow at neutral pH and slowly lower the pH as more sulfur is oxidized. These organisms are *Thiobacillus neopolitanus*. As the pH is lowered, a second group of organisms, *Thiobacillus thiooxidans* becomes active. These organisms are vigorous sulfur oxidizers and are capable of lowering the pH of an environment to 2.0 or below. The presence of these organisms indicates that significant sulfur oxidation and acid production has occurred. A third group of organism that may be present is *Thiobacillus ferrooxidans*. This organism has the unique ability to oxidize either sulfur or ferrous iron at low pH. This organism, in addition to oxidizing sulfur, can contribute to the destruction of exposed reinforcing steel in concrete structures.

Analysis of samples from regions of deteriorated and non-deteriorated concrete would indicate the presence of microorganisms that could cause microbially induced concrete deterioration. The degree of concrete deterioration could be correlated with the number and type of Thiobacilli present. Extensive deterioration may be noted at the locations where the most acidophilic group of Thiobacilli would be present in elevated numbers. Areas of lesser deterioration would be somewhat acidic, with a combination of different sulfur-oxidizing Thiobacilli present. Areas with less deterioration would be populated with the least acidophilic group of sulfur-oxidizing Thiobacilli.

Concrete may be also be damaged by live organisms, such as plants, sponges, boring shells, or marine borers. Mosses and lichens, which are plants of a higher order, cause insignificant damage to concrete. These plants secrete weak acids in the fine hair roots. The roots enable mosses and lichens to adhere to the concrete. The acids that are secreted from mosses and lichens will attack the cement paste and cause the concrete to disintegrate and scale. In some cases carbonic acids are produced from plants, such as mosses and lichens, when substances from these plants decompose. The carbonic acid that is produced will attack the concrete.

Rotting seaweed has been known to produce sulfur. Sulfur produces sulfuric acid. The presence of sulfuric acid on concrete leads to concrete disintegration. The growth of seaweed on concrete may also create a problem if the seaweed is exposed at low tide. When the seaweed is exposed at low tide, the seawater that is retained by the seaweed

becomes more concentrated by evaporation. The effect of seawater on concrete increases as the concentration of seawater increases.

Rock boring mollusks and sponges, which are common in reefs or areas where the seabed is composed of limestone, may invade underwater concrete structures and piles containing limestone aggregate.

The pattern of infestation greatly differs between organisms. When mollusks attack concrete, their pattern of infestation is widespread and relatively deep. The holes that mollusks bore extend through both the aggregate and cement paste. Boreholes created by mollusks are located perpendicular to the outer surface of the concrete and can measure up to 10 mm in diameter. Although the depths of boreholes from mollusks vary, growth measurements indicate a rate of bore hole penetration of about 10 mm per year. Boreholes serve solely as protective enclosures for the mollusks.

The pattern of infestation created by boring sponges are shallow, closely spaced, small diameter holes that average 1 mm in diameter. The boreholes created by boring sponges are often interconnected. The attack of boring sponges on concrete is generally concentrated in small areas. As the degree of honeycomb in the concrete increases, the surface material of the concrete crumbles.

Marine borers, such as mollusks and sponges bore holes into underwater concrete structures. Marine borers reduce the concrete's load-carrying capacity as well as expose the reinforcing steel to the corrosive seawater. Boring sponges produce interconnected bore holes. The surface material of the concrete crumbles as the degree of interconnection increases.

Rock boring mollusks and sponges will also chemically bore holes into concrete containing calcareous substances.

Table 1.1 summarizes the commonly occurring deterioration of concrete. The causes for such deterioration phenomena and the appearance of concrete are also shown in this table.

Table 1.1. Commonly occurring concrete deterioration problems.

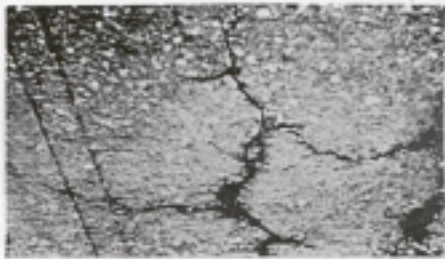
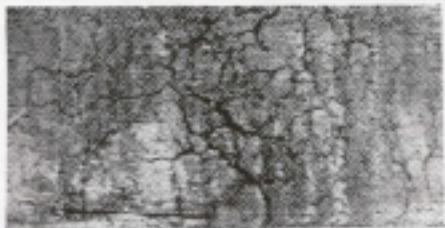
Damage	Description	Typical Picture
Pattern cracking (alkali-silica reaction)	Fine openings on concrete surface in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material near the surface, or both.	
Pattern cracking (alkali-carbonate reaction)	Fine openings on concrete surface in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material near the surface, or both.	
Plastic cracking	Cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic.	
Shrinkage cracking	Cracking of a structure or member due to failure in tension caused by external or internal restraints as reduction in moisture content develops, or as carbonation occurs, or both.	
Abrasion damage	Wearing away of a surface by rubbing and friction.	
Blistering	The irregular raising of a thin layer, frequently 25 to 300 mm in diameter, at the surface of placed mortar or concrete during or soon after completion of the finishing operation; blistering is usually attributed to early closing of the surface and may be aggravated by cool temperatures. Blisters also occur in pipe after spinning or in finish plastic coat in plastering as it separates and draws away from the best coat.	

Table 1.1 (Contd.). Commonly occurring concrete deterioration problems.

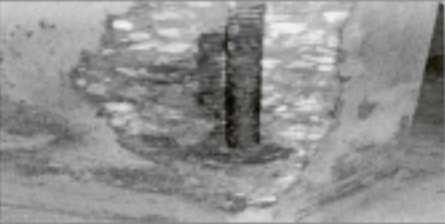
Damage	Description	Typical Picture
Cavitation damage	Pitting of concrete caused by implosion, i.e., the collapse of vapor bubbles in following water which form in areas of low pressure and collapse as they enter areas of higher pressure.	
Corrosion damage	Destruction of metal by chemical, electrochemical, or electrolytic reaction with its environment.	
Delaminations	A separation along a plane parallel to a surface as in the separation of coating from a substrate or layers of a coating from each other, or in the case of a concrete slab, a horizontal splitting, cracking or separation of a slab in a plane roughly parallel to, and generally near, the upper surface; found frequently in bridge decks and other types of elevated reinforced-concrete slabs and may be caused by the corrosion of reinforcing steel; also found in slabs on grade caused by development, during the finishing operation, of a plane of weakness below the densified surface; or caused by freezing and thawing, similar to spalling, scaling, or peeling except that delamination affects large areas and can often be detected by tapping.	
Dusting	The development of a powdered material at the surface of hardened concrete.	

Table 1.1 (Contd.). Commonly occurring concrete deterioration problems.

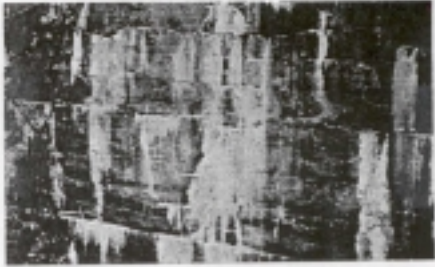
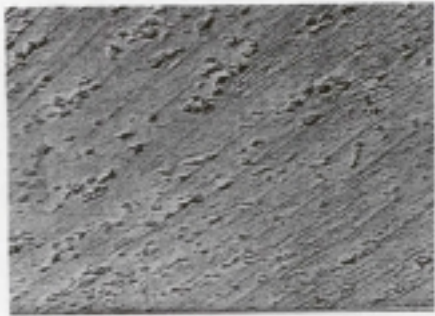
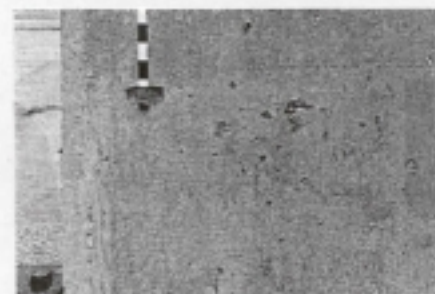
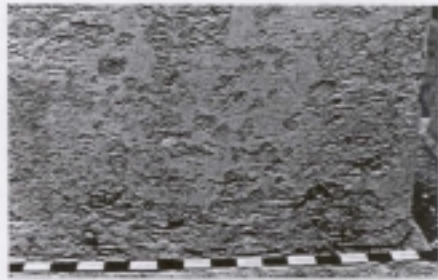
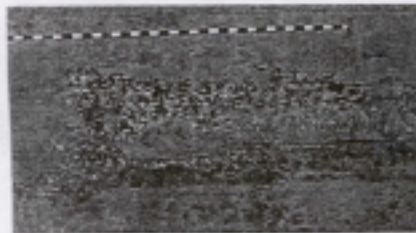
Damage	Description	Typical Picture
Efflorescence	A deposit of salts, usually white, formed on a surface, the substance having emerged in solution from within either concrete or masonry and subsequently been precipitated by evaporation.	
Joint spall	A spall adjacent to a joint.	
Pitting	Development of relatively small cavities in a surface; in concrete, localized disintegration, such as a popout; in steel, localized corrosion evident as minute to forms as forms are removed.	
Peeling (close-up)	A process in which thin flakes of mortar are broken away from a concrete surface, such as deterioration or by adherence of surface mortar to forms as forms are removed.	
Popout	The breaking away of small portions of a concrete surface due to localized internal pressure which leaves a shallow, typical conical, depression. Popout leaving holes up to 10 mm in diameter, or the equivalent.	

Table 1.1 (Contd.). Commonly occurring concrete deterioration problems.

Damage	Description	Typical Picture
Scaling (very severe)	Local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also of a layer from metal.	
Scaling (medium)	Local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also of a layer from metal. Loss of surface mortar 5 to 10 mm in depth and exposure of coarse aggregate.	
Honeycomb	Voids left in concrete due to failure of the mortar to effectively fill the spaces among coarse aggregate particles.	

CHAPTER 2

ASSESSMENT OF CONCRETE DETERIORATION

2.1 INTRODUCTION

Before a repair or rehabilitation work can be proposed, it is often necessary to conduct an inspection and evaluation of the deteriorated concrete structures so as to identify the nature and extent of the existing problem and its probable causes. This prior knowledge is an essential prerequisite, as it enables engineers to seek a long lasting and functionally effective remedial work.

The inspection and assessment work is generally performed for one or several of the following purposes:

1. To determine the causal factors of deterioration or damage, which may result from loading, exposure conditions, inadequate design, or poor construction.
2. To assess the structural adequacy and safety and to rate its residual capacity, if required.
3. To determine the feasibility of retrofitting or repairing a distressed structure to restore its strength and serviceability, conforming to codes of practice.
4. To check if the strength and quality of the concrete conform to the prescribed specification, or if they are acceptable for carrying out the planned repair.

As an inspection is carried out on existing structures, the inspection and assessment program should be followed in a manner that minimizes further damage to the structure. This requires the use of nondestructive test methods.

For a damaged or deteriorated structure, an effective repair or restoration work can be designed only after an inspection and assessment of the structural condition and identification of the probable causes, as the objective of repair is to restore serviceability and safety on a long-term basis. Even for a routine repair, the selection of repair materials should conform to the findings of a diagnostic evaluation.

2.2 EVALUATION PROGRAM

For a reliable assessment of concrete deterioration or damage, the investigative work is carried out systematically in two phases: (a) preliminary investigation and (b) detailed investigation. The preliminary investigation, which is essentially a visual inspection and appraisal, is a must for an evaluation program. A well-trained eye at a site visit can pick up valuable information with regard to the problems related to deterioration, damage, structural integrity, safety, serviceability and cracking. This initial progress, which leads to an *initial optional* on the nature of the problem, sets the stage for the detailed investigation, if required. Based on the results of the preliminary survey, the detailed inspection is planned, identifying the tests and the number of test samples required for the evaluation.

A detailed investigation will often necessitate some testing of concrete, either in-situ or in the laboratory (or both) to determine the material properties, their composition, and characteristics. In planning a test program, three factors should be collectively considered: (a) objectives of the program, (b) cost and time, and (c) degree of accuracy and reliability.

An evaluation program that may require a detailed investigation can be synthesized into the following major components.

- Preparation (preparatory deskwork).
- Preliminary investigation (visual inspection and appraisal).
- Detailed investigation (data collection, field measurements and condition survey, sample collection, testing of samples, analysis, and evaluation).
- Conclusions and recommendations.

A full report of the investigation, which is normally required in a project, must be written with full details of all work performed, documenting the extent of deterioration or damage and identifying the causes of the problem. The report should address the feasibility of repair and recommend the remedial work needed to restore serviceability and strength.

2.3 PREPARATION PRIOR TO PRELIMINARY INSPECTION

Prior to inspection, some preparatory deskwork needs to be undertaken. This includes collection and review of the following:

- a. Design Information
 - i. Plans/drawings of the structure, as-built drawings, specifications and other applicable information or data.
 - ii. Location of the structure and topology and accessibility of parts of the structure for inspection.
 - iii. Structural drawings to know the orientation of the main reinforcing steel.
- b. Service History

2.4 PRELIMINARY INVESTIGATION

The goals of the preliminary investigation, which is essentially a thorough visual inspection, are to obtain initial information regarding the condition of the structure, the nature of the problems affecting it, the feasibility of undertaking the proposed rehabilitation work and, importantly, the need and scope of a subsequent detailed investigation. This investigation in most cases reveals sufficient information so as to form an initial opinion on the problem. It helps in establishing the following:

- i. Structural condition, extent of cracking, damage/deterioration;
- ii. The apparent safety of the structure and the need for temporary safety measures;
- iii. The need for commissioning a detailed inspection;
- iv. Accessories needed for detailed inspection: boat (if water involved), ladder, formwork for access, lighting, and other equipment;

- v. Traffic control requirements, and
- vi. Any unusual problem facing the structure.

During a site visit for the preliminary inspection, simple equipment such as hammer, tape, cover meter, crack width microscope, camera etc. should be carried along to assist the inspection and simple *in-situ* measurements.

A condition survey through visual inspection should be recorded with sufficient photographic documentation of the extent and severity of any damage and deterioration that could affect the serviceability or load-carrying capacity of the structure. Previously repaired area should also be examined. The inspection should be supplemented with sketches. General crack mapping with significant crack widths should be recorded.

Visual inspection should also include other areas such as the examination of bearings, expansion joints, drainage, seals, etc. Any visual impairment of the functional capacity of a component should be recorded. Exploratory removal can be used when there is substantial evidence of serious deterioration or distress, when hidden defects are suspected, or where such action will enlighten or reinforce the feeling of the problem.

The preliminary inspection should also aim to find if there is an imminent danger of safety so as to close the structure from users, and if temporary strengthening or supporting is needed. The preliminary inspection results should be summarized in a report focusing on the actions required. If no further testing or evaluation is needed, the report should specify the necessary repair or remedial work to be performed.

2.5 DETAILED INVESTIGATION

A detailed investigation should only be undertaken after the preliminary investigation has been completed and the goals and objectives are properly identified.

A detailed investigation, in general, consists of the following six major tasks:

- (i) Data collection (documentation),
- (ii) Field measurements and condition survey,
- (iii) Sample collection,
- (iv) Testing of field samples,
- (v) Analysis and evaluation, and
- (vi) Final report containing conclusions and recommendations.

The findings of the detailed investigation directly influence the final outcome of the evaluation process and the choice of the repair/rehabilitation method and the materials.

2.5.1 Data Collection (Documentation)

Apart from the data and information collected earlier in preparation, additional data, drawings, and documents related to the structure should be obtained.

2.5.2 Field Measurements and Condition Survey

The scope and degree of involvement depends upon the findings of the preliminary investigation. There is no need of this task when the available information and the findings of the preliminary condition survey are considered sufficient to complete an evaluation with confidence.

If a detailed investigation is necessary, it is required to make an assessment of what specific information is needed, which translates into the type of tests required. In choosing the test methods, a compromise must be made among the three important aspects of in-situ testing, namely, the objective, cost, and reliability, as test methods range widely in cost, reliability and complexity.

Generally, *in-situ* testing should involve commonly used nondestructive test (NDT) methods and should cover a sufficient number of locations that are determined from a compromise of cost, accuracy, and effort.

The following are included under this:

- (a) With regard to structure
 - (i) Verification of as-built construction
 - as-built dimensions of members at critical locations with regard to spans and cross sections
 - determination of voids, honeycombing in suspected locations using NDT and by hammer-sounding.
 - (ii) Construction anomalies
 - spacing of reinforcing steel, size, and concrete cover to reinforcement at a sufficient number of locations or at locations of interest using NDT
 - estimation of in-situ concrete strength for the purpose of verification and use in analysis and evaluation using NDT
 - (iii) Environment

The actual loading and load combination and the prevailing environmental conditions may be different from those assumed in the design. Any addition of load (equipment or machinery) or a variation from the intended use of the structure should be recorded along with the prevailing environmental data.

It is recommended that the provisions for the condition assessment, as stipulated in ACI 201.1R should be followed.

- (b) With regard to deterioration and distress:
 - (i) The crack pattern should be mapped, indicating the width, length, and location of significant cracks and the type of crack (structural or

nonstructural). An attempt should be made to identify the structural cracks as flexure, shear or direct tension, if possible.

- (ii) Spalling, scaling, efflorescence and other surface defects should be measured and photographed for documentation.
- (iii) Unusual or excessive deformations, misalignments, and visible constructional anomalies should be measured, recorded, and photographed.
- (iv) Defective bearings for bridges, connections for precast elements, and architectural elements, joint seals, etc. should be noted.
- (v) Water leakage, drainage problems, ponding areas, and other indications of water related problems should be noted.
- (vi) Evidence of chemical attack and the extent of damage, if any, due to sulfate attack, corrosion of reinforcing steel, salt-weathering, and alkali-aggregate reactivity should be noted and documented. For corrosion damage, the loss of reinforcement area shall be measured, if possible.
- (vii) For steel corrosion activity assessment, measurements or the mapping of potentials and electrical resistivity of concrete can be carried out according to ASTM C 876. Corrosion rate in the in-situ concrete can be measured using the linear polarization resistance method.
- (viii) Problems related to foundations (e.g. settlement, tilting of the structure and erosion of soil) should be noted.

2.5.3 Sample Collection

Generally, the field samples needed for testing consist of (a) cores, (b) broken pieces of concrete, (c) drilled powdered samples and (d) pieces of steel reinforcement (if needed). Sampling should be conducted in a manner that would yield a good representative sample.

Cores are the widely used test samples, as they can be engaged in a host of tests, including the measurement of compressive strength. Unlike cores, the broken concrete pieces and the powdered samples, which are, extracted by drilling into the concrete body, have limited applications. The former type can only be used for physical examination and chemical analysis and the latter type (powdered samples) is usable only for chemical analysis. For the evaluation of steel reinforcement, pieces of steel from the representative areas have to be extracted by chipping or by breaking the concrete, and they are needed only if information about the type of steel (e.g. grade and strength) is required.

Cores provide a good deal of information about the quality and strength of concrete in a very reliable manner. The following data/information can be extracted from the core samples:

- (a) Concrete strength: by testing 70 to 75 mm or higher diameter cores.

- (b) Crack depth measurement: by extracting cores across a significant crack, the depth of the crack through the thickness and the orientation of crack can be observed.
- (c) Chemical analysis: provides the chloride and sulfate content and the chloride profile through the thickness needed in evaluating the corrosion potential and activity. Chemical analysis is required for identification of a possible chemical attack. From the chemical analysis, an estimate of the cement content can also be made.
- (d) Concrete permeability: large core samples can be used in water permeability tests to determine the permeability of the in-situ concrete. Chloride permeability can be determined utilizing 70 to 75 mm diameter cores according to ASTM C 1202.
- (e) Aggregate gradation: from large core samples, the aggregate grading and the original water/cement ratio can also be determined according to BS 1881 or ASTM C 85.
- (f) Aggregate type: from the core samples, the aggregate type can be examined visually, physically, chemically or petrographically. A petrographic study can reveal reactive aggregates (alkali-aggregate reactivity).

Broken pieces of concrete can be used for items (c), (e) and (f) above, and powdered samples extracted by drilling can only be used for item (c).

2.5.4 Testing of Field Samples

Samples collected from the field are tested in the laboratory to derive the data that are being sought. Laboratory tests can reveal the following data:

- (i) Concrete strength
- (ii) Cement content
- (iii) Chloride and sulfate content
- (iv) Carbonation
- (v) Aggregate gradation
- (vi) Original water content (water/cement ratio)
- (vii) Type of aggregates
- (viii) Alkali-aggregate reactivity
- (ix) Permeability
- (x) Density
- (xi) Air voids, etc.

Several of these methods are described in detail in ASTM and BS specifications. Therefore, testing should be carried out in strict compliance with these specifications.

2.5.5 Analysis and Evaluation

ANALYSIS

Three phases are involved in the analysis of in-situ results:

- (a) Computation and correction of results,
- (b) Calibration, and
- (c) Data presentation.

Computation and Correction of Results: In-situ measurements or the data collected from samples tested in the laboratory should be converted to an appropriate parameter in accordance with well-defined procedures. For example, the core strength determined using core samples taken from the field must be corrected for the height/diameter ratio and embedded reinforcement to yield the equivalent cylinder strength. To determine the cube strength, the cylinder strength must again be converted with an appropriate factor. Pulse velocity is calculated, making proper allowance for cracks and reinforcements.

All test results are compiled to determine the mean, maximum, and minimum strengths with the standard deviation. In making such calculations, occasionally, the data that appear to be suspicious (unusually high or low) should be disregarded. The poor results may be due to poor samples and a possible error in testing.

Pulse velocity and rebound numbers are converted to concrete strengths by using appropriate calibration models, which are either available or have been developed for similar concrete. The degree of accuracy of the predicted strengths would depend upon the accuracy of the calibration models. In many cases, some correction factors may have to be applied due to the variation in the concrete quality from that of the model concrete.

Results of the surface hardness measurement, pullout test, Lok-test, etc. should be averaged to determine a mean value. Chemical or similar tests must be carried out appropriately and the results computed to determine the appropriate parameters, such as cement content, chloride content, mix proportions, etc. Load tests conducted for behavior and the rating of structure will involve a considerable amount of calculations involving deflection, stress, and moments.

Calibration: In-situ measurements for strength by various nondestructive test methods are converted to predict the concrete strength by means of appropriate calibration curves, developed earlier or modeled exclusively for a particular case. In general, it is not always feasible to develop an independent calibration curve for each test. If a calibration curve is available from the supplier or manufacturer of the equipment, or it has been developed earlier in another test program, this curve can be used by taking into account the variation of the significant parameters between the in-situ and the model concrete. Most of the time, calibration curves are predetermined from a set of tests on laboratory specimens. The concrete quality and mix proportions may not be identical for the laboratory and the in-situ samples. If such a disparity exists, some correction factors must be introduced to the calibration curves to suitably modify them for application.

Data Presentation: When numerous results are available over different areas of the structure, a study of variability (or check for uniformity) can define the areas of differing quality (or strength). This study can be done in two ways:

- (a) Graphical presentation, and
- (b) Numerical presentation.

Graphical presentation is a more visible documentation of the variability. Contours can be plotted, using either the actual readings (e.g. pulse velocity, rebound number, half-cell potential readings) or the converted strength values. Contour plots become meaningful, if a large number of readings are available throughout the surface, with some degree of variations in the readings.

Contour maps can also be plotted for the half-cell potentials for corrosion activity. From the contour plots, the areas of active corrosion and no corrosion can be marked to clearly indicate these zones.

Concrete variability can also be expressed as histograms when a large number of results are available. For a good concrete construction, the spread of the histogram (tail) will be smaller, and for a poor construction, the spread will be longer, indicating a significant variation in the concrete strength.

The variation in strength or any other property can also be expressed by statistical parameters (numerical methods). The standard deviation and variance of the results can be calculated to indicate the degree of variability. Traditionally, the coefficient of variation (calculated as standard deviation $\times$ 100/mean value) is used as an indicator of variability.

EVALUATION

Final results by themselves will provide the answer to the problem. Occasionally, however, the results should be interpreted in the light of the figures and the limitations. Each in-situ test has its limitation. Furthermore, likely test and sampling errors, which are unpredictable, do influence the outcome. Hence, a careful review of the final results should be made to finally conclude the assessment.

The evaluation of concrete in a structure may involve one or more of the three components: (i) load-carrying capacity, (ii) protective component (durability), and (iii) deterioration.

For concrete to function as a *load-carrying member*, the following three coincidental characteristics are required:

- (i) Adequate strength,
- (ii) Adequate cross-sectional area of both the concrete and the reinforcing steel, and
- (iii) Adequate bond between the steel and the concrete.

The adequacy of these three components must be proven to declare the member as structurally adequate.

For concrete to function as a *protective cover* for steel and to provide durability, it must be dense, with low permeability and diffusivity, free from a harmful level of aggressive salts (chlorides and sulfates), and must contain good quality non-reactive aggregates.

With regard to the *assessment of deterioration*, standards specify some guidelines regarding the maximum limits of the influencing parameters. Whenever such values or data are available, they can be used as the basis for interpreting the final results. Some of the frequently used permissible values are provided below:

Chlorides and Sulfate Limit in Hardened Concrete:

According to BS 8110 (1985):

$$\begin{aligned}\text{max. chloride content} &= 0.4\% \text{ by weight of cement for reinforced} \\ &\text{concrete,} \\ &= 0.1\% \text{ by weight of cement for prestressed} \\ &\text{concrete, and}\end{aligned}$$

sulfate content should not exceed 4% by weight of cement.

The maximum chloride ion content recommended by ACI 318 is shown in Table 2.1 below.

Table 2.1. Chloride concentration limits according to ACI 318

Type of member	Maximum water soluble chloride ion concentration in concrete, percent by weight of cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

Permeability: DIN 1048 specification provides some guidelines on water permeability.

The depth of water penetration under the DIN 1048 permeability test should not exceed 50 mm for concrete with a 'weak' chemical attack and 30 mm for concrete with a "strong" chemical attack. For the rapid determination of chloride permeability, the AASHTO T 277 test and its assessment criteria can be utilized.

Crack Width: Generally, cracks of width > 0.3 mm are considered significant. The measurement should indicate crack widths greater than 0.3 mm.

Carbonation: Carbonation can be easily detected by treating the concrete surface with phenolphthalein. A pink coloration indicates no carbonation and no coloration (acidic area) shows carbonation.

Strength Calculation: As part of the inspection, many times, the strength calculation will be required to assess the residual strength of the elements or structure as a whole. For such a calculation, measured in-situ values of material properties and dimensions would be necessary. The accuracy of the strength prediction will vary according to the method utilized. Furthermore, it is recognized that it is always very difficult to model properly a partially damaged/deteriorated structure for analytical solutions, regardless of how accurately relevant the material properties have been determined. BS 6089 specifies a minimum factor of safety of 1.2 for general assessment. If the material properties obtained are true representative of the critical locations, the engineer must adopt a higher safety factor using his judgment.

Data recorded from the in-situ load or strength test must be used in the analysis in the usual way to compute stresses and deformation. The load rating computation should yield a recommended safe load.

EVALUATION OF STEEL REINFORCEMENT

The function of the steel reinforcement in a concrete structure is to carry tensile and compressive forces. Thus, the properties of the steel reinforcement in terms of strength and reduced cross-sectional area must be determined in addition to the existing bond to evaluate the load carrying ability. The degradation of the bond in extensively cracked and corroded zones should be examined.

2.5.6 Final Report

Based on the analysis of data and damage, the final evaluations are made with a set of conclusions, which should indicate the causal factors for deterioration and damage. Recommendations are then prescribed as corrective measures for repair or rehabilitation work, which would be functionally effective.

A final report on the inspection and assessment is often required, and it should be submitted covering the entire inspection and assessment work undertaken, and summarizing the findings of the investigation. The report should cover the following:

- (a) The aim and scope of the investigation,
- (b) Data collection/documentation,
- (c) Field measurements and condition survey,
- (d) Sample collection and laboratory testing of samples,
- (e) Analysis and evaluation, and
- (f) Conclusions and recommendations

The report should include, whenever required, a discussion on the feasibility of the repair. The recommendation must address the following topics if repair/rehabilitation is part of the investigation action plan. Outlining the proposed remedial work, cost estimates for repair and the limitations and constraints, if any.

If the report recommends that no action is required for the present, it should indicate when a re-inspection is needed for further evaluation and what temporary measures shall be undertaken till then. The findings of the inspection report should be archived to allow any future change in the condition of the structure to be identified after a later inspection.

2.6 COMMONLY USED TEST METHODS

Over the past few decades, the nondestructive testing of concrete has received increasing acceptance for the evaluation of the strength, properties, and uniformity of in-situ concrete. Various techniques and measurement procedures have been proposed and are being used with a varying degree of accuracy, reliability, and complexity.

The available wide-ranging test methods can be grouped into three distinct categories: (a) methods for estimating the concrete strength, (b) methods for evaluating properties other than strength, and (c) methods for evaluating the physical condition of concrete structures.

(a) *Methods for Estimating the In-Situ Concrete Strength*

The following test methods to determine the strength of concrete have been proposed and used with a varying degree of accuracy:

	<u>Tests</u>	<u>Equipment Type</u>
1.	Core tests	Mechanical
2.	Surface hardness method	Mechanical
3.	Ultrasonic pulse velocity	Electronic
4.	Break-off and Pull-off	Mechanical
5.	Penetration tests (Windsor Probe)	Mechanical
6.	Pull-out test	Mechanical
7.	Lok test and Capo test	Mechanical

Of the above mentioned tests, only the surface hardness method (popularly known as the Schmidt hammer test) and the ultrasonic pulse velocity method are truly nondestructive, as the testing does not inflict any surface damage to the concrete. All other methods will cause some local damage to the structures. Core tests, if undertaken properly, are the most reliable tests among all the methods used in determining the in-situ concrete strength.

(b) *Methods for Determining Properties other than Strength*

There are a wide variety of tests available to determine the various properties pertaining to quality, composition, and durability characteristics. The latter group includes deterioration due to corrosion of bars, sulfate attack on cement, alkali-aggregate reactivity, carbonation, and salt weathering. The commonly used or needed tests are to determine the following:

- i. Alkali-aggregate reactions (petrographic analysis)
- ii. Chemical analysis (cement content, chloride content and sulfate content)

- iii. Corrosion activity (half-cell potentials, electric resistivity, and linear polarization resistance measurements)
- iv. Permeability
- v. Composition (w/c ratio, aggregate gradation)
- vi. Quality of aggregate (soundness, reactivity)

(c) *Methods for the Physical Evaluation of Structures*

The tests in this group are concerned with the physical condition in terms of cracking, delamination/voids, honeycombing, scaling, spalling, chemical deterioration, and uniformity of concrete and structural performance/rating. Again, various test methods are available, including in-situ load tests, to assess the structural performance and load rating.

Figure 2.1 elaborates the essential steps involved in the evaluation of causes and extent of concrete deterioration. Test methods available for item (a) and (b) are shown collectively in Table 2.1. Test methods for item (c) are shown in Table 2.2 and those applicable for reinforcing steel are shown in Table 2.3.

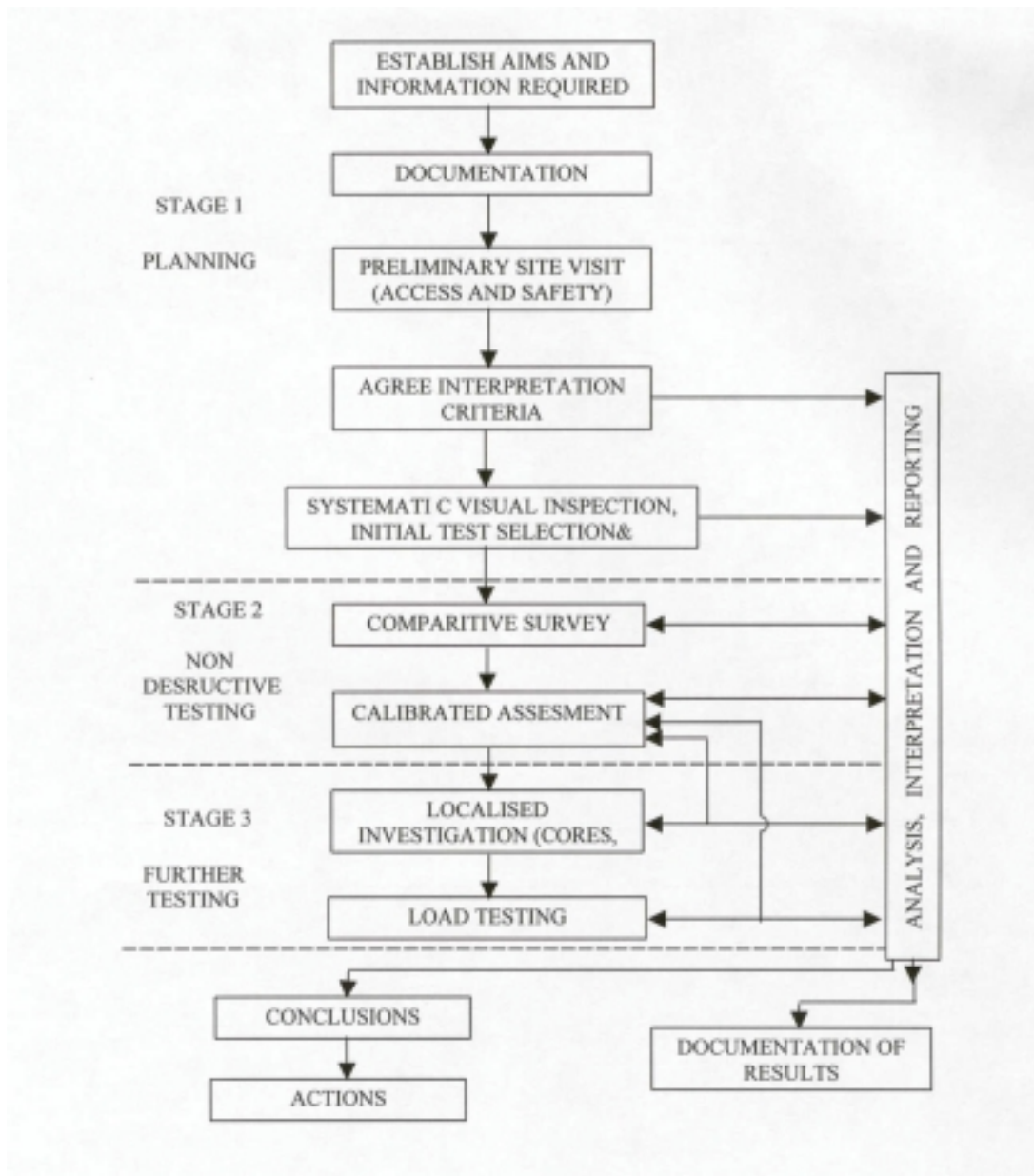


Figure 2.1. Stages of investigations to assess the cause and extent of deterioration in a concrete structure.

Stage 1- By Operating Organization

Stage 2- By Operating Organization / Inspection Department / Vendors

Stage 3- By Operating Organization / Vendors / Consult CSD

Table 2.1. Recommended tests for evaluation of concrete properties.

Evaluation Procedure Chemical and Physical Properties	Acoustic Impact	Air Content Test (ASTM C457)	Cement Content Test (ASTM C1084)	Chemical Tests	Core Testing (ASTM C42)	Electrical Potential Measurements	Electrical Resistance Measurements	Flexural Tests (ASTM C42)	Freeze-Thaw Test (ASTM C666)	Gamma Radiography	Nuclear Moisture Meter	Permeability Test (CRD C48)	Petrographic Analysis (ASTM C856)	Pullout Testing (ASTM C900)	Rebound Hammer (ASTM C805)	Ultrasonic Pulse (ASTM C597)	Windsor Probe (ASTM C803)
Acidity				♦									♦				
Air Content		♦											♦				
Alkali-Carbonate Reaction													♦				
Alkali-Silica Reaction													♦				
Cement Content			♦	♦									♦				
Chemical Composition				♦									♦				
Chloride Content				♦	♦								♦				
Compressive Strength					♦									♦	♦	♦	♦
Contaminated Aggregate				♦									♦				
Contaminated Mixing Water				♦									♦				
Corrosion Environment				♦		♦											
Creep					♦												
Density					♦					♦							
Elongation					♦												
Frozen Components													♦				
Modulus of Elasticity					♦											♦	
Modulus of Rupture					♦			♦									
Moisture Content					♦		♦				♦						
Permeability												♦	♦				
Pull out Strength														♦			
Quality of Aggregate													♦				
Resistance to Freezing and Thawing					♦				♦				♦				
Soundness					♦					♦			♦				
Splitting Tensile Strength					♦												
Sulfate Resistance				♦									♦				
Tensile Strength					♦			♦									
Uniformity	♦												♦		♦		♦
Water-Cement Ratio													♦				

Table 2.2. Recommended tests for evaluation of the physical condition of concrete.

Evaluation Procedure Physical Condition	Acoustic Emissions	Acoustic Impact	Chemical Tests	Core Testing (ASTM C42)	Fiber Optics	Gamma Radiography	Infra Red Thermography	Load Testing (ACI 437R)	Petrographic Analysis (ASTM C856)	Physical Measurement	Radar	Rebound Hammer (ASTM C805)	Ultrasonic Pulse (ASTM C597)	Ultrasonic Pulse – Echo (ASTM C597)	Visual Examination (ACI 201.1R, ASTM C823)	Windsor Probe (ASTM C803)
Bleeding Channels									◆						◆	
Chemical Deterioration			◆						◆						◆	
Corrosion of Steel			◆	◆					◆						◆	
Cracking	◆	◆		◆	◆		◆		◆	◆	◆		◆	◆	◆	
Cross-Sect. Properties and Thickness				◆		◆				◆			◆			
Delamination		◆		◆	◆	◆	◆		◆		◆		◆	◆	◆	
Discoloration			◆						◆						◆	
Disintegration				◆		◆	◆		◆				◆		◆	
Distortion															◆	
Efflorescence			◆						◆						◆	
Erosion									◆						◆	
Freeze-Thaw Damage									◆						◆	
Honeycomb				◆	◆	◆	◆		◆				◆		◆	
Popouts															◆	
Scaling															◆	
Spalling				◆		◆	◆								◆	
Stratification		◆			◆									◆	◆	
Structural Performance	◆							◆							◆	
Uniformity of Concrete						◆			◆			◆	◆		◆	◆

Table 2.3. Recommended tests for evaluation of the properties of reinforcing steel.

Evaluation Procedure Properties and Conditions	Acoustic Impact	Chemical Analysis (ASTM A751)	Coating Tests (ASTM A775, G12, 14, 20)	Cover Meter Pachometer	Electrical Potential Measurements	Gamma Radiography	Physical Measurements	Radar	Tension Test	Ultrasonic Pulse-Echo (ASTM C597)	Visual Inspection
Adhesion of Epoxy Coating			◆								
Anchorage							◆				
Bend Test							◆				
Breaking Strength									◆		
Carbon Content		◆									
Chemical Composition		◆	◆								
Coating Properties		◆									
Concrete Cover				◆		◆	◆	◆			
Continuity of Epoxy Coating											
Corrosion					◆		◆				◆
Cross-Sectional Properties and Thickness							◆				
Deformations							◆				◆
Elongation									◆		
Exposure											◆
Rebar Location	◆			◆		◆	◆	◆			
Reduction of Area									◆		
Shape							◆				
Strength of Connections							◆				
Tensile Strength									◆		
Thickness of Epoxy Coating			◆								
Weld Shear Strength							◆				
Yield Strength									◆		

CHAPTER 3
STRATEGY FOR REPAIR OF DETERIORATED CONCRETE
STRUCTURES

3.1 INTRODUCTION

Repair and rehabilitation of deteriorated concrete structures are essential steps not only to utilize them for their intended service-life but also to ensure the safety and serviceability of the associated components. A good repair should improve the function and performance of the structure, restore and increase its strength and stiffness, enhance the appearance of the concrete surface, provide water tightness, prevent ingress of the aggressive species to the reinforcing steel surface, and improve its durability. In order to achieve these attributes it is essential that the causes and extent of damage should be understood. For this purpose, a systematic investigation is necessary. A thorough assessment of the condition of the structure should include:

- (i) Cause of damage or loss of protection,
- (ii) Degree and amount of damage,
- (iii) Expected progress of damage with time, and
- (iv) Effect of damage on structural behavior and serviceability.

The methodology for assessing the causes and extent of deterioration has been presented in Chapter 2 of this manual.

Once the cause and extent of damage is established, a strategy for repair of structures has to be established. The strategy for repairing a structure depends on several factors, some of which are listed below:

- (i) The cause and extent of damage,
- (ii) The consequences of the damage, i.e. does the damage influence the structural safety of the structure or just its appearance?
- (iii) The appropriate time for intervention, i.e. is the rate of damage so low that repairs could be postponed or not attempted at all?
- (iv) Economic aspect, i.e., the cost effectiveness of the repair method adopted, and
- (v) Operational constraints.

It should, however, be realized that a repair strategy is dictated by the individual situation. Based on the situation, the repair strategy may vary from doing nothing to making repairs of varying intensity or partial to total replacement of the structure. A detailed description of the conditions under which one of these repair strategies needs to be considered is discussed in the following subsections.

Following the decision on the repair strategy, the repair work needs to be designed in detail and appropriate materials selected.

Since deterioration of concrete due to reinforcement corrosion is widely noticed in the Eastern Province of Saudi Arabia and many of Saudi Aramco's facilities, repair strategies for structures affected by this cause are discussed in Section 3.5.

3.2 NO REPAIR

This option should be considered when the extent of damage is very minimal or has occurred accidentally and is not expected to be repeated. Examples of such deterioration may be an accidental overload and an acid spillage or exposure to other aggressive agents that are not expected to occur in the remaining life of the structure. Another fact that may encourage this decision is that the rate of deterioration is very low and that the calculated useful service-life of the structure is more than the original designed life. Reinforcement corrosion progressing at a very low rate is a typical case. However, it should be confirmed, through a very well planned investigation, that the structural integrity, serviceability and short-term or long-term durability of the structure are not affected by delaying the repair. The structure should be continuously monitored to ascertain that the rate of deterioration is within the expected margin. The frequency of monitoring should be planned depending on the rate of deterioration. One of the drawbacks of this option is that one has to tolerate the aesthetic appearance, for example a buckled column, sagging beam, surface scaling etc., of a structure.

The "do nothing" option of repair does not totally preclude the option of repair. Depending on the extent of the deterioration, the repair may be postponed to a later time. This option has the advantage that repairs can be prioritized.

Another important aspect that should influence the choice of not repairing the structure is whether the cause of existing deterioration of the structure is minimized. As stated earlier, unexpected loads should not be repeated or in the case of durability related deterioration, the environment should be changed such that the rate of deterioration is less than the present rate of deterioration.

3.3 REPAIR

This option should be considered when avoiding the repair, as discussed in Section 3.2, is not feasible. A structural component needs to be repaired either to amend its structural failure or improve its durability. On many occasions a durability failure leads to a compromise of the structural integrity of the structure. For example, excessive reinforcement corrosion in a beam or column may contribute to a drastic reduction in the load-carrying capacity of a structure.

Based on the extent of deterioration repair may be classified as follows:

- (i) Cosmetic,
- (ii) Partial, and
- (iii) Total.

The factors that influence the selection of one of the above options are discussed in the following subsections.

3.3.1. Cosmetic Repair

This type of repair is aimed at improving the appearance of a deteriorated component. It normally addresses the deterioration of concrete in non-structural concrete or deterioration caused due to exposure to aggressive agents. Concrete deterioration caused by acid spillage may be one of the types repaired for cosmetic reasons. Another case where cosmetic repair, for example by the application of a surface coating, could be utilized is a situation where concrete distress due to reinforcement corrosion has been noticed. In such situations, application of a properly selected surface coating can decrease the diffusion of oxygen and moisture to the steel surface thereby decreasing the rate of reinforcement corrosion.

3.3.2 Partial Repair

This type of repair is carried out with the intention of rectifying the apparent defects without really addressing the cause of the damage. This is also a sort of deferred repair. Patch repair of the areas where spalling and delamination has been detected and leaving the other areas for repair at a later date, as and when deterioration will be apparent, is an example of a short-term repair.

However, and as emphasized in Section 3.1, the safety of the structure and the associated components should always be the prime criteria for selecting the repair strategy. Also, monitoring programs should be planned to assess the performance of repairs and damage at other locations.

3.3.3 Total Repair

The total or the so-called permanent repair option is better than the two repair options considered above. A total repair should address all aspects of concrete deterioration, viz., structural and durability, and should also include remedial actions to minimize future deterioration.

The considerations supporting a total repair are the following:

- (i) Structural safety of the components is compromised due to an advanced stage of deterioration, and
- (ii) Operational constraints do not permit the complete replacement of the structural component.

As elaborated above, a total repair needs not only to address the prevailing deterioration but also to design methodologies for avoiding future occurrence of the problem. In certain cases, for example offshore and intake structures, in addition to repairing the deteriorated concrete structures, it will be necessary to prevent future reinforcement corrosion probably by the application of a cathodic protection system.

3.4 PARTIAL OR TOTAL REPLACEMENT

The total or partial replacement of a structure should be considered when investigations have shown that the deterioration is at an advanced stage or the structure is structurally unsafe. A decision towards total replacement of a structure should be considered only after technical or economic considerations rule out the possibility of either partial or total repairs. The economic considerations should not only include the cost of actual repair but also the downtime involved during the repair. Sometimes, the cost of stopping the industrial operations may be much more than the actual cost of the repair itself.

Figure 3.1 details the factors influencing the selection of the repair strategies discussed in Sections 3.2 through 3.4.

As stated earlier, the cost of repair also controls the choice of a repair strategy. An engineer has therefore to calculate the cost of the proposed repair. The cost calculation assists in the selection of an appropriate repair methodology. A few examples of calculating the cost of repairing deteriorated concrete structures are provided in Chapter 6 of this report.

3.5 STRATEGY FOR REPAIRING A STRUCTURE WITH REINFORCEMENT CORROSION

Since deterioration of concrete due to reinforcement corrosion is widely noticed in the eastern province of Saudi Arabia and in many of Saudi Aramco's facilities, the repair strategies, as elucidated in Sections 3.2 through 3.4, are illustrated in the context of this situation. Figure 3.2 shows the essential steps in considering the strategy for repair of structures with varying stages of reinforcement corrosion.

3.5.1 “Do Nothing” Option with Corroded Bars

This option should be considered when the field data do not show corrosion activation and the chloride concentration at the rebar level is also very low. The rate of reinforcement corrosion is expected to be very low in this situation.

Potential measurements indicate active corrosion, but the loss of section, determined either by linear polarization resistance measurements or gravimetric weight loss measurements, on samples of rebars retrieved from cores indicate a low corrosion rate, i.e. the expected service life, at the present rate of deterioration, is more than the designed service life.

3.5.2 Cosmetic Repair of Structures with Corroded Bars

This option of repair should be considered when the field data indicate active corrosion, but deterioration is insufficient to significantly reduce structural capacity either at the time of investigation or in the foreseeable future. Cosmetic repair by the application of a surface coating will reduce the ingress of oxygen and moisture to the steel surface and decreases the rate of reinforcement corrosion within the acceptable limits.

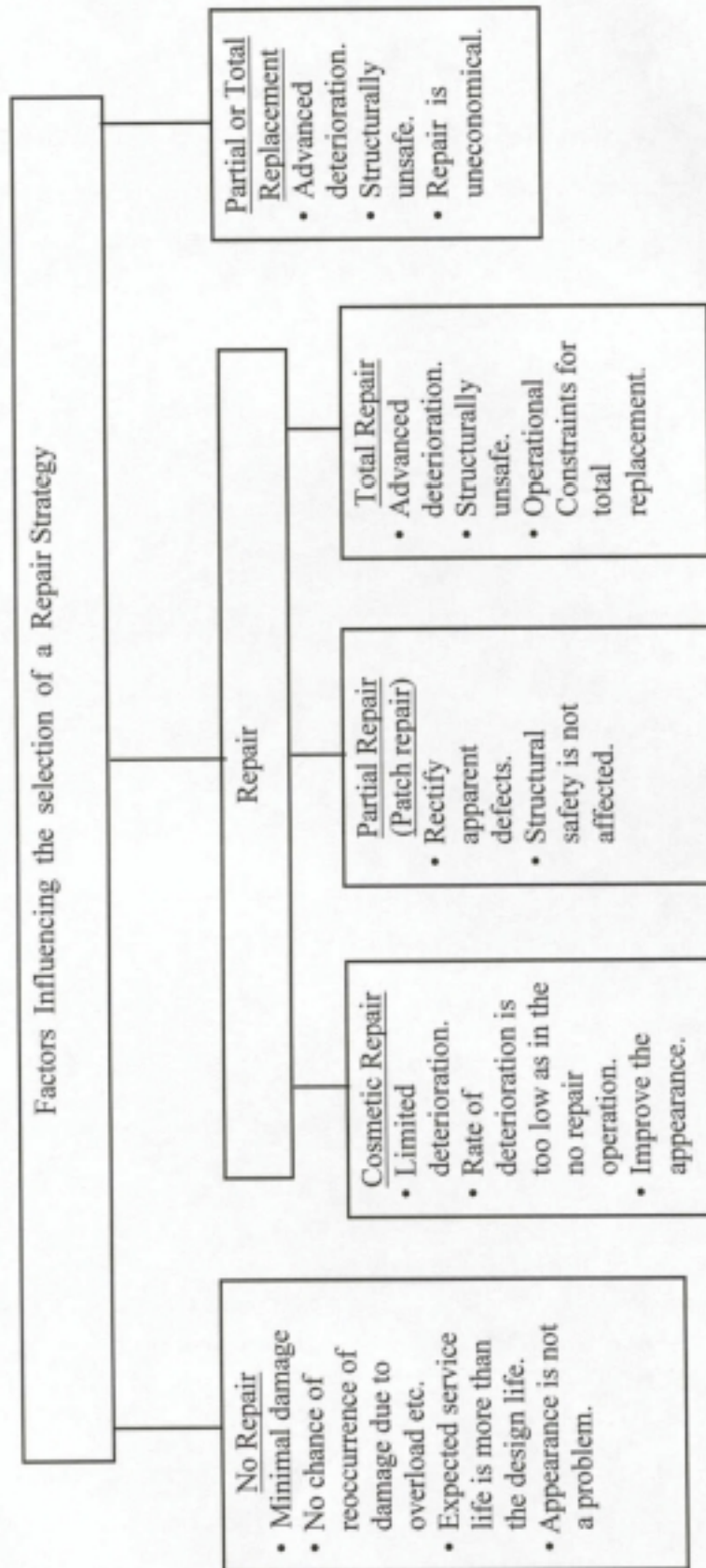


Figure 3.1. Factors influencing the selection of a repair strategy.

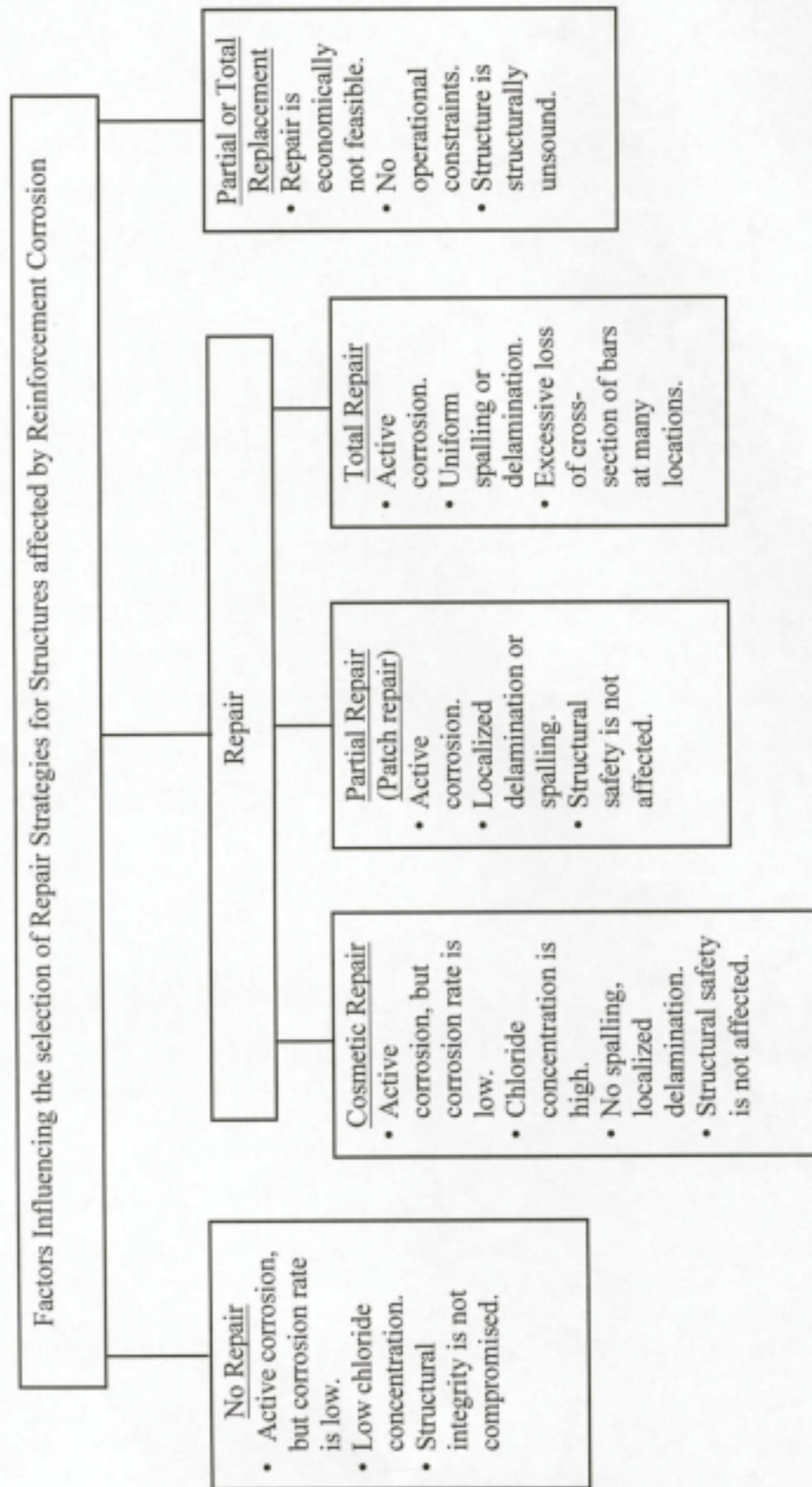


Figure 3.2. Factors influencing the selection of a repair strategy for structures with reinforcement corrosion.

3.5.3 Patch Repair of Structures with Corroded Bars

This option should be considered when the field data indicates active corrosion and deterioration in the form of localized spalling of concrete. The load carrying capacity of the structure should meet that required by the applicable codes of practice. Patch repair of concrete followed by application of coatings will reduce the rate of reinforcement corrosion in the repaired areas. However, the deterioration is not totally treated, as the corrosion will still be active in the un-repaired areas.

3.5.4 Total Repair of Structures with Corroded Bars

This repair option should be considered when the field measurements indicate active corrosion and sufficient loss of cross section of the rebars at many locations. Structural calculations should show that the structural capacity of the structure is below the design limits. The option for such a situation is to replace the bars and build the section to obtain a good bond between the old concrete and the repair material.

3.5.5 Partial or Total Replacement of Structures with Corroded Bars

Partial or total replacement is the other option that should be considered provided there are no operational constraints, such as excessive downtime. Whatever the mode of repair is, it should always be complemented by a well-planned methodology for improved durability of the structure. At the time of partial or total replacement the possibility of changing the environment around the structure or using materials resistant to the prevailing environment should be considered.

CHAPTER 4

REPAIR MATERIALS AND THEIR EVALUATION

4.1 INTRODUCTION

Repair and rehabilitation of deteriorating structures is essential not only to utilize them for their intended service-life but also to ensure the safety and serviceability of the associated components. A good repair improves the function and performance of the structure, restores and increases the strength and stiffness, enhances the appearance of the concrete surface, provides water tightness, prevents ingress of the aggressive species to the reinforcing steel surface and improves concrete durability.

Repair of a deteriorated concrete structure requires selection of an appropriate repair strategy. Once the repair strategy has been selected, the appropriate repair materials and techniques need to be selected. The selected repair materials have to conform to certain physical and durability properties.

This Chapter details the types of repair materials that are commonly utilized in the repair of deteriorated concrete structures. The procedures for their evaluation are also elucidated along with the recommended performance criteria.

4.2 REPAIR MATERIALS

A repair system essentially consists of four components, namely repair mortar, bond coat, steel primer and surface coating. Each of these components influences the properties of the total repair system. Incompatibility, either, physical, thermal or chemical, between the repair components or the repair system and the base concrete, leads to the failure of the complete system. It is therefore necessary to have knowledge of the generic systems for each component of the repair system so that a durable repair system may be planned. Since repair of a deteriorated structure is sometimes costlier than the original cost of the structure it is essential to make certain that multiple repairs are avoided.

4.2.1 Repair Mortars

There are many repair mortars available. Some important characteristics of a repair mortar are: good bond to substrate, movement characteristics compatible with substrate, low permeability, alkaline passivation of rebar, structural strength, durability to freeze-thaw and weathering, and easy application.

Several types of repair mortars, such as cementitious mortars, resin-based repair mortars, and polymer modified cementitious mortars are available for repair work, particularly patch work. Each of the repair mortars has advantages and disadvantages and is suitable for unique situation.

CEMENTITIOUS REPAIR MORTARS

In virtually all cases of concrete deterioration, the problem is associated with corrosion of steel reinforcement. It is well established that steel reinforcement well embedded in good quality concrete is protected from corrosion by a highly alkaline cement matrix. Therefore, whenever possible, it is desirable for both technical and economic considerations that deteriorated reinforced concrete should be repaired with impermeable alkaline cement-based materials closely matched in properties to the parent concrete. Cementitious mortars are lower in cost than the resin mortars, and they have thermal

expansion and movement characteristics more compatible with the concrete substrate. However, the resin-based mortars are preferred in areas subject to chemical attack, where thin sections have to be applied or a rapid strength gain is required. Sometimes, cement mortars are modified by the addition of additives, such as silica fume or polymers, to improve their properties.

POLYMER MODIFIED CEMENTITIOUS MORTARS

Polymer latexes are often added to cement mortars. Such mortars afford the same alkaline passivation protection of the steel as conventional cementitious materials and can readily be placed in a single application at 12 to 15 mm thickness that gives adequate protective cover. The polymer latex acts in several ways:

- (a) It functions as a water reducing plasticizer, producing a mortar with good workability and lower shrinkage at lower water/cement ratios.
- (b) It improves the bond between the repair mortar and the concrete being repaired, providing, of course, that they are applied and used properly.
- (c) It reduces the permeability of the repair mortar to water, carbon dioxide, and oils; and it also increases its resistance to some chemicals.
- (d). It acts, to some degree, as an integral curing aid; but in drying conditions, very careful curing is still essential.

Styrene-Butadiene Based Polymer Portland Cement Concrete (PCC) has been used as a repair mortar for the last four decades. The positive track records prove styrene/butadiene to be an excellent copolymer for PCCs.

A typical PCC formulation (wt. %) consists of:

Portland cement	50
Sand	125
Aggregate	100
Polymer dispersion	15.5
Water	9
Entrained air (%)	6 max.
Polymer to cement ratio	0.15
Water to cement ratio	0.30 to 0.35

RESIN-BASED REPAIR MORTARS

Unlike cement-based repair mortars, whose high alkalinity helps prevent reinforcement corrosion by passivation, resin mortars provide protection by encapsulating the steel reinforcement with an impermeable 'macro' coating which exhibits excellent adhesion to both the steel and the concrete substrate. This protective mortar/coating will give good long-term protection to the steel at a thickness far less than is possible with the cementitious repair materials. Epoxy resin mortars are most widely used in concrete repairs. Polyester resin and acrylic resin-based mortars are also used generally for patch repairs where very rapid development of strength is required. Polyester and acrylic

mortars with high elastic modulus and rigidity are not suitable for larger repairs because of dangers of shrinkage and subsequent cracking or debonding. However, those with low elastic modulus are capable of absorbing these stresses and have been used to a limited extent in rapid setting repair mortars. Epoxy resin mortars are also available which can give excellent handling and curing characteristics at high ambient temperatures. Mortar systems based on lightweight fillers are available which can be applied up to 30 mm thickness in a single layer on soffits and vertical faces without problems.

SPRAYED CONCRETE

Sprayed concrete is a mixture of cement, aggregate and water projected at high velocity from a nozzle into place against an existing structure or formwork where it is consolidated by its velocity to produce a dense homogenous mass. Sprayed concrete is placed either by the dry or wet process. Guniting and shotcrete are variously used to refer, respectively, to sprayed concrete where the aggregate is less than 10 mm, and 10 mm and upwards maximum size. They also apply respectively to dry process and wet process sprayed concrete.

Where relatively large areas (and in some instances relatively small areas) require repairing, particularly on arches, soffits, etc., repair by sprayed concrete techniques are probably the most cost effective. In this technique, the sand and cement are pre-blended and blown by air pressure into a nozzle where the gauging water is introduced under pressure. The gauged material is then pressure sprayed onto the formwork. Some times fibers are incorporated in the shotcrete. The use of sprayed concrete incorporating steel fibers eliminates the need for prior pinning of a steel mesh onto the prepared work.

INJECTION GROUT

Cracks in reinforced concrete greater than approximately 0.3 mm require sealing/injection to prevent ingress of moisture, oxygen, carbon dioxide, chloride and sulfate and other corrosive chemicals, or to protect the integrity of the structure. Cracks wider than about 1 mm in the upper surfaces of slabs etc. can often be sealed by brushing dry cement followed, if necessary, by light spraying with water. For cracks wider than about 2 mm it may be preferable to use a cement and water grout.

Low-viscosity liquid polymers can be used in a similar way to cement grout. Some of these materials will penetrate cracks down to about 0.1 mm width but, in general, the repair will not be structural. It will seldom be necessary to seal cracks narrower than this.

When it is necessary to ensure, as far as possible, that the sealant penetrates to the full depth of a crack, injection of polymer grout under pressure is the method most commonly used. For relatively wide cracks that are unlikely to be blocked by debris, it may be enough to use a gravity head of a few hundred millimeters, but in other cases hand-operated or mechanical pumps or pressure pots are used.

FLOWING MICROCONCRETES

In some situations, however, a more wholesale replacement of the facing concrete is required and a more efficient method than hand placing becomes an economic and logistic necessity. The congestion of reinforcement can also be another factor precluding

effective repair by hand placement. In these situations, the use of fluid cement grouts bulked out with small aggregate proves to be useful.

To base such a mix on simple cement grout would be to invite severe shrinkage. Superplasticizers must be used to achieve fluidity whilst keeping the water to cement ratio down to 0.4 or less. The two-stage shrinkage compensation can be built into the formulation: gas expansion in the fluid stage, and further slight expansion after the repair has hardened can prevent distress over the ensuing weeks as full drying occurs.

The quantity of aggregate that can be incorporated may be limited by the method of placing. If a simple pouring technique is used, 10 mm aggregate may form 50% of the total dry materials, but if pumping is preferred the aggregate size and quantity will need to be reduced to suit the limitations of the pump.

4.2.2 Bond Coat Materials

When applying conventional concrete, sprayed concrete or sand/cement repair mortars, establishing a reliable bond between the parent concrete and the repair mortar is often a problem. In particular, where the repairs are to be carried out at high ambient temperatures, water loss at the interface between the repair material and the prepared concrete may prevent proper hydration of the cement matrix at this interface. The use of an epoxy resin or polymer latex bonding aid can assist in achieving a reliable bond. With an epoxy bonding system, specifically formulated for bonding green uncured concrete to cured concrete, a bond is achieved which is significantly greater than the shear strength of good quality concrete or mortar. Under the severe drying conditions often encountered in the Arabian Gulf, the “open time” for polymer latex bonding coats can be too short to be a practical method of ensuring a good bond between the repair mortar and the parent concrete. For this reason, epoxy resin bonding aids with adequate pot life and open time for the application conditions are more widely used in concrete repairs in this area. As an alternative to polymer latex bond slurry coats, there are now available factory blended polymer modified cementitious bonding aids based on special spray dried copolymer powders blended with cement, fine sand, and other special additives. They are simply gauged with water on site and applied to the prepared parent concrete to give a “stipple” finish. Even when allowed to set overnight, this type of bonding aid gives a good “key” for the repair mortar. It also prevents rapid loss of water from the repair mortar, which may result in inadequate hydration and thus poor bonding of the repair mortar. However, application of the repair mortar, while this key coat is still tacky, is recommended wherever practicable. In some instances, the epoxy bonding aid is required to function as an impermeable barrier between the repair mortar and the parent concrete. In these cases, two coats of the bonding aid are applied and, while still tacky, are dressed with clean sharp sand. This ensures an excellent mechanical key between the two coats and the repair mortar.

The requirements for a concrete bonding aid include: compatibility with cement, adequate bond, usable working life, tolerance to wet conditions in use, able to be applied under strong drying conditions, tolerance to misuse, and ease of use. Many types, such as water, slurry coat, polymer emulsions, polymer emulsion slurries, and epoxies are available.

There are many types of polymer emulsions available. Polyvinyl acetate (PVAC), styrene butadiene rubber (SBR), and acrylic emulsions are commonly used. PVAC is cheap and gives the best overall performance in dry conditions. However, it should not be used under wet conditions or externally. SBR gives excellent results if used properly; however, failures have been experienced on site if the polymer is allowed to form film prior to the application of the repair mortar. This is because the polymer film is completely stable under wet conditions, and once formed can act as a slip plane.

Acrylic emulsions provide the most foolproof and effective bonding agents for concrete repair on site. Acrylic emulsions are more expensive than PVAC or SBR, but their properties seem to combine the benefits of the two. They are practically stable under wet conditions but will soften sufficiently to provide an excellent bond to a subsequent repair several hours after application.

Polymer emulsion slurries with cement or cement mortars can provide better results than with the emulsions used alone. However, they are very sensitive to site abuse, as widely differing mixes can be used. In addition, slurries tend to dry out more rapidly than most neat emulsions and are difficult to use under extreme drying conditions.

Epoxy bonding aids provide the best bond of all when assessed by the pull off test. They perform well on site, particularly the slow setting versions that give a long open time. Additionally they can provide a waterproof membrane between a substrate and a repair. There is a possibility that they may also electrically isolate the repair zone from the surrounding concrete, which will help prevent the steel in the substrate from corroding.

In situations where the saturating of the substrate is undesirable, the alternative is to use an epoxy resin bond coat. These are two-part systems requiring the correct proportioning of resin base and hardener (usually supplied in pre-weighed packs) which must be mixed thoroughly and then applied to the prepared concrete substrate. The fresh cementitious mortar or concrete should then be applied whilst the epoxy resin is still in a tacky condition. Suitably formulated epoxy bond coats usually have an 'open time', during which the cementitious material can be placed, of up to 24 hours. In fact, the most successful systems allow the concrete to harden before the epoxy. The suitability of a formulation should in any case be checked by carrying out a slant shear bond test.

Resin bond coats can also be of advantage in situations where the substrate concrete is likely to remain permanently saturated, when a polymer-dispersion based bond coat might never develop full strength.

The main disadvantages of epoxy bonding agents are their high cost, precise mixing, and the need to clean up with solvents. However, for certain applications, e.g., for the bond of a cementitious mortar on to a steel beam, they provide the most satisfactory service.

4.2.3 Steel Primers

The ideal requirements for a rebar primer are that it must protect the steel, not be subject to undercutting (i.e. progressive rust creep under the primer), have a good bond to the steel and subsequent repair, have no adverse effect on the adjacent steel, and be easy to use.

There are many types of rebar primers in use today. The following provides a list of possible alternative primers:

- Cement mortar slurry,
- Polymer modified cement slurry,
- Non passivating epoxy,
- Passivating epoxy, and
- Zinc rich epoxy (one or two parts).

4.2.4 Surface Coatings

After a patch repair, it is probable that the surrounding concrete has deteriorated to a certain extent. In order to improve the durability of the whole structure, it is beneficial to seal the concrete against further carbonation and chloride ion ingress. The requirements for such coatings are that they must penetrate and seal the surface for many years against penetration of oxygen, CO₂, ingress of chloride ions, sulfate ions, and water. At the same time, water vapor should be allowed to escape to the outside, enabling the concrete to “breathe”.

The range of choices for the protective coating system is quite wide; various compositions have been used to coat concrete, including bituminous coatings, chlorinated rubber, polyvinyl copolymers and terpolymers, acrylics (reactive, solvent based and water based), polyurethane and epoxy resins. Such coatings, if free from defects (cracks, pinholes, etc.), prevent the passage of water or aqueous salts in liquid or mist form and have a low permeability to water vapor, carbon dioxide, and oxygen. Long-term durability depends upon a number of factors, including chemical composition of the binder, precise formulation of the coating, total film thickness, and application techniques. In many instances, it is desirable that the appearance of the concrete substrate is unchanged and the concrete surface is treated to reduce its permeability. Such systems can significantly reduce the permeability of the concrete to water and aqueous salt. But without the build up of a finite film on the surface of the concrete, the permeability of the concrete to carbon dioxide is generally not reduced sufficiently for long-term service. In some service conditions, it is possible that the rate of carbonation may in fact be increased. This is because optimum rates of carbonation occur when the relative humidity in the pores within the concrete is on the order of 60 to 70%.

Penetrating sealers that reduce chloride ingress include acrylic resin solutions, water repellent silicone resins, and certain types of silane resins, epoxies, and polyurethane. Providing the materials have filled the pores within the surface of all of the concrete as intended, they should give good long-term durability. However, conventional silicone resin types which function purely by making the pore water repellent seldom last more than a few years. The alkyl silanes function in the same manner, however, they are more durable than silicone resins. The molecular size of resin or silane penetrants is important as it significantly influences the depth of penetration into the surface of the concrete. Some of the silane treatments, based for example on low molecular weight isobutyl trimethoxysilane, penetrate well into concrete; and, under still laboratory conditions they

have proved to be very effective water repellents. However, such silanes are extremely volatile, evaporating at similar evaporation rates to the solvents in conventional gloss paints; and when applied to concrete at high ambient temperatures, much of the material may disappear by evaporation. Their efficacy as water repellents may therefore be much reduced. There are now available blends of less volatile silanes of similar molecular size blended with oligomeric siloxanes derived from these silanes. These are more cost-effective water repellents when applied under typical site conditions. Aqueous solutions of highly alkaline silicate and silico-fluorides have also been used to seal and harden surfaces of concrete.

Unlike traditional silicone materials, the silane reacts with OH ions and water to form a hydrophobic layer that is repellent to liquid water but permeable to water vapor. After a minimum drying time of two hours, the special acrylic resin-based topcoat is applied. This appears to have a synergistic effect with the silane to provide excellent resistance against penetration by aggressive chemicals and yet still allows the concrete to breathe. The coatings can be applied by brush, spray or roller, and typical coverage rates are 0.4 l/m² for the primer and 0.2 l/m² for the top coat.

In addition to the ingress of gases that lead to a lower pH within the concrete matrix, concrete structures are occasionally subject to abnormally acidic environments. In such situations, the coatings need to arrest the process of acid attack may be different from those normally specified as anti-carbonation coatings. They have to withstand more aggressive conditions and in some cases a fairly high degree of chemical resistance may be necessary to afford protection to the substrate. Under most circumstances, two-part polyurethane coatings will cope well with the relatively dilute acids and still allow passage of water vapor through the film. There are situations, though, where the surface will have become quite badly etched and there will be difficulty in achieving a continuous film. This is particularly relevant to the materials described above, because they are normally applied in relatively thin coats. In these circumstances it may be preferable to use a high-build epoxy paint to achieve the necessary protection, or to apply a suitable leveling coating prior to applying the specified coating.

USES OF COATINGS

Anti-carbonation Coatings: Carbonation occurs because carbon dioxide diffuses into the concrete and dissolves in the pore water. This produces carbonic acid that reacts with the free lime to form calcium carbonate. Although this acts as a partial barrier to further carbonation, the process is progressive, except in very dense concrete, and it leads to a gradual fall in pH. Once carbonation reaches the reinforcement, depassivation of the steel results in corrosion and spalling when water and oxygen are present.

Coatings may be applied to concrete to arrest the carbonation process. These are known as anti-carbonation coatings and are normally based on chlorinated rubber, polyurethane resins or acrylic emulsions. Although they are principally designed to prevent diffusion of carbon dioxide and oxygen into the concrete, the coatings will also limit or prevent penetration of chlorides in solution.

In most cases, anti-carbonation coatings allow free passage of water vapor. This is so that vapor pressure does not build up behind the paint film and cause it to blister. It has been argued, though, that this is unnecessary and that adequate surface preparation will allow a

relatively impervious film to be applied without subsequent loss of adhesion. However, there are numerous examples of such coatings which have been successfully applied to concrete, it is not necessary to use impervious materials as anti-carbonation coatings. It should also be emphasized that adequate surface preparation is required prior to the application of all coatings where a long and effective service life is expected.

Anti-carbonation coatings may be effectively used to resist carbonation and general atmospheric deterioration of the concrete. In cases where carbonation has occurred but has not reached the reinforcing steel, application of a coating system will limit the ingress of oxygen, carbon dioxide and moisture and will reduce the rate of deterioration of the concrete. It should be emphasized that concrete should be coated only after suitable pore filler or leveling coat has been applied.

Where corrosion and spalling are widespread, anti-carbonation coatings are used as the final part of the repair specification. They are applied after the patch repairs have been carried out, in order to prevent further carbonation of the original concrete.

Acid-Resistant Coatings: In addition to the constant ingress of gases which may lead to a lower pH within the concrete matrix, concrete structures are occasionally subject to abnormally acidic environments. This is most often due to combustion of fossil fuels, or of elemental sulfur that occurs at chemical plants. This releases sulfur dioxide into the atmosphere that readily dissolves in rainwater and forms sulphurous acid. Limited amounts of sulfur trioxide are also present in flue gases, so some sulfuric acid may be produced as well.

In the situations described above, the coatings needed to arrest the process of acid attack may be different from those normally specified as anti-carbonation coatings. They have to withstand more aggressive conditions, and in some cases a fairly high degree of chemical resistance may be necessary to afford protection to the substrate. Under most circumstances, two-part polyurethane coatings will cope well with the relatively dilute acids and still allow passage of water vapor through the film. There are situations, though, where the surface will have become quite badly etched and there will be difficulty in achieving a continuous film. This is particularly relevant to the materials described above, because they are normally applied in relatively thin coats. In these circumstances it may be preferable to use a high-build epoxy paint to achieve the necessary protection, or to apply a suitable leveling coat prior to applying the specified coating.

Coatings to protect cracked concrete: Coatings are quite often applied locally over cracks to prevent ingress of water and carbon dioxide. Cracks that are protected in this way are normally very fine and not considered having any structural significance. Repair of such cracks by resin injection is not recommended in structures where movement is anticipated, because new cracks are likely to form. It is therefore necessary for coatings used in this type of repair to have flexibility as well as the ability to bridge cracks. High-build polyurethane and epoxy-polyurethane formulations have both been successfully used in this application. The method has been deployed to cover cracks in some offshore structures that would otherwise have been subjected to chloride attack. Coatings have also been successfully used to protect concrete affected by alkali silica reaction. The main requirement of such a coating is that it should be able to bridge cracks and also be flexible enough to accommodate further movement.

4.3 TESTING OF REPAIR MATERIALS

Performance testing of repair materials should be based on measurements of dimensional stability and strength, and protection provided to the reinforcement and durability of the components and the repair system as a whole. However, different elements of the repair system require different properties, depending on their functions in the system.

In order to develop a comprehensive approach to the testing of concrete repairs, individual components and the systems, as a whole should be examined where possible.

4.4 TESTS METHODS FOR CEMENT- AND POLYMER-BASED REPAIR MATERIALS

The standard test methods that are normally utilized to determine the properties of cement- and polymer-based repair mortars are detailed in Table 4.1.

4.5 TEST METHODS FOR RESIN-BASED REPAIR MORTARS

The standard test methods that are utilized used to determine the properties of resin-based repair mortars are detailed in Table 4.2

Table 4.1. Details of specimens and test methods to determine the properties of cement- and polymer-based repair mortars.

Property	Test method	Minimum number of specimens to be tested
Flow	ASTM C 190	3
Stiffening time	BS 4551	2
Bleed	Non-standard	2
Compressive strength	ASTM C 109	6
Tensile strength	BS 6319	3
Flexural strength	BS 6319 Part 3	6
Elastic modulus	BS 6319	6
Shrinkage	ASTM C 157	3
Thermal expansion	ASTM C 531	3
Adhesion	BS 6319 part 4	3
Chloride permeability	ASTM C 1202	3
Carbonation	Non-standard	3
Electrical resistivity	Non-standard	3

4.6 TEST METHODS FOR BOND COAT MATERIALS

The standard techniques that are normally utilized to determine the properties of bond coat materials are detailed in Table 4.3.

Bond strength is normally determined using a three crossed-prism test specimen, as specified in ASTM C 321.

Table 4.2. Details of specimens and test methods to determine the properties of resin-based repair mortars.

Property	Test method	Minimum number of specimens to be tested
Pot life	ASTM C 308	3
Rate of cure	ASTM C 884	3
Adhesion	BS 6319	3
Compressive strength	ASTM C 579 Method A	6
Tensile strength	ASTM C 307	6
Flexural strength	ASTM C 580	6
Elastic modulus	ASTM C 580	6
Shrinkage	ASTM C 531	6
Thermal expansion	ASTM C 531	6
Chloride	ASTM C 1202	3

permeability		
Chemical resistance	ASTM C 267	3
Peak exotherm	BS 6319	3

The chloride ion permeability of the bond coat materials can be determined according to ASTM C 1202. The bond coat material is coated on both sides of a concrete specimen and chloride permeability is determined, as per the procedure described in ASTM C 1202.

Cylindrical concrete specimens, measuring 75 mm in diameter and 150 mm in height, are normally utilized for determining the electrical resistivity of the bond coat materials. After curing for 28 days, the concrete specimens are coated with the selected bond coat materials. The coverage rate and the number of coats should be that specified by the manufacturer.

Table 4.3. Details of specimens and test methods to determine the properties of bond coat materials.

Property	Test method	No. of specimens to be tested per component
Improvement in bond	BS 6319	6
Chloride permeability	ASTM C 1202	3
Carbonation	Non-standard	3
Electrical resistivity	Non-standard	3

4.7 TESTING OF STEEL PRIMERS

The steel primers should be applied on steel specimens according to the manufacturer's recommendations. The number of specimens and test methods to determine the properties of steel primers are detailed in Table 4.4.

Table 4.4. Details of specimens and test methods to determine the properties of steel primers.

Property	Test method	No. of specimens to be tested per component
Adhesion to steel	ASTM D 4541	3
Sensitivity to steel cleaning	Non-standard	3
Resistance to	ASTM D 1654	3

salt exposure		
Crevice attack	ASTM G 78	3
Resistivity	Non-standard	3

4.8 TESTING OF SURFACE COATINGS

The surface coatings may be applied on cement mortar/concrete substrate and tested to determine their properties. Table 4.6 details the number of specimens and the test methods that can be utilized to determine the properties of the selected surface coatings.

Table 4.5. Details of specimens and test methods to determine the properties of surface coatings.

Property	Test method	Specimen size	No. of specimens to be tested per component
Adhesion	ASTM D 4541	62 x 100 x 300 mm (concrete)	3
Crack bridging	Non-standard	25 x 25 x 250 mm (mortar)	3
Chloride diffusion	Non-standard	75 mm dia and 50 mm high (concrete)	3
Moisture resistance	Non-standard	50 mm dia and 72 mm high (mortar)	3
Water permeability	DIN 1048	150 x 150 x 150 mm (concrete)	3
Carbonation resistance	Non-standard	50 mm dia and 72 mm high (mortar)	3
Chemical resistance	ASTM C 267	25 x 25 x 25 mm (mortar)	3

4.10 PERFORMANCE CRITERIA

The performance criteria for the repair materials are detailed in Tables 4.6 through 4.10.

Table 4.6. Performance criteria for polymer- and cement-based repair mortars.

Property	Performance criteria
Bleeding	1% maximum (applicable only for non-flowing mortars)
Compressive strength (ASTM C 109)	Min. 40 MPa after 28 days
Flexural strength (ASTM C 580)	Min. 4.0 MPa after 28 days
Tensile strength (BS 6319)	Min. 2.5 MPa after 28 days
Modulus of elasticity in compression (BS 6319)	25 to 35 GPa after 28 days
Coefficient of thermal expansion (ASTM C 531)	$7.5 \text{ to } 10 \times 10^{-6}/^{\circ}\text{C}$
Shrinkage, measured at 25 °C ASTM C 157)	Max. 500 μ after 7 days
Chloride permeability (ASTM C 1202)	Low
Electrical resistivity	More than 200 Ohm.m in saturated surface dry condition.

Table 4.7. Performance criteria for resin-based repair mortars.

Property	Performance criteria
Peak exotherm (BS 6319)	Low
Compressive strength (ASTM C 109)	More than 35 MPa
Flexural strength (ASTM C 580)	More than 20 MPa
Modulus of elasticity (BS 6319)	Min. 4.5 MPa
Chloride permeability (ASTM C 1202)	Negligible
Chemical resistance (ASTM C 267)	Good (Maximum strength reduction 10% when exposed to 3% sulfuric acid for 60 days).

Table 4.8. Performance criteria for selecting bond coat materials.

Property	Performance criteria
Bond strength (BS 6319)	More than 1 MPa.
Chloride permeability (ASTM C 1202)	Less than 1000 coulombs
Electrical resistivity (saturated condition)	Not less than 200 Ohm.m

Table 4.9. Performance criteria for selection of steel primers.

Property	Performance criteria
Adhesion to the steel surface (ASTM D 4541)	More than 1 MPa
Sensitivity to cleaning (reduction in adhesion)	Not more than 10%
Resistance to salt exposure (ASTM D 1654 procedures A and B)	Rating of 9 and above.

Table 4.10. Performance criteria for selection of surface coatings.

Property	Performance criteria
Adhesion	Not less than 1.0 Mpa*
Depth of water penetration (DIN 1048)	No penetration for epoxy-based coatings Less than 4.5 cm for cement-based coatings
Chloride permeability (ASTM C 1202)	Negligible for epoxy-based coatings Low for other generic types
Crack bridging capability	Not less than 0.2 mm
Chloride diffusion	Not more than $1.75 \times 10^{-7} \text{ cm}^2/\text{s}$

CHAPTER 5
REPAIR PROCEDURES

5.1 INTRODUCTION

Damage to reinforced concrete structures can be in the form of cracking, spalling, blowholes, voids, honeycombs, inadequate cover, etc. These damages are caused either during construction or during service.

Lack of adequate consolidation will cause various surface defects, including voids, blowholes, and honeycombs. Rapid drying of fresh concrete under hot weather conditions will give rise to the formation of shrinkage cracks on concrete surface. Settlement of the structure with time will cause settlement cracks. Thermal variation between the external and internal parts of the structure will cause thermal cracking in the structure. Corrosion of reinforcement steel bars will eventually end up with cracking and spalling of concrete in the vicinity of corroding steel bar. Sulfate attack and salt weathering will not only destroy the integrity of concrete mass but also inflict cracking and spalling of concrete. Finally, the change in dynamic loads or undue loading of structures, due to natural disasters, such as earthquakes, floods etc., can induce a wide range of damages from cracking to complete collapse.

In this chapter, the commonly utilized procedures for repair of damaged reinforced concrete structures are elucidated.

5.2 REPAIR OF CRACKED AND DETERIORATED CONCRETE

5.2.1 Repair of Shrinkage Cracks

Plastic shrinkage cracks (PSCs) are caused due to hot weather conditions during casting and hardening of concrete. Hot weather causes rapid drying of the concrete surface due to evaporation and results in cracking of concrete when it is still plastic. These usually appear as random straight cracks – sometimes shallow but may be as deep as 100 mm or more - are often concentrated in the center of flatwork. They form rapidly after the water sheen disappears from a slab under construction.

Repair of concrete after the plastic shrinkage cracks have formed usually consists of sealing them against ingress of water by brushing in cement or low viscosity polymers.

5.2.2 Repair of Settlement Cracks

These cracks form due to settlement of the concrete, especially in deep sections, after it has started to stiffen. Anything that obstructs the movement of concrete during settlement, such as reinforcement or formwork tie-bolts, may act as a wedge so that a crack forms immediately over the obstruction. Cracks may also form in vertical surfaces when friction against the formwork hinders settlement of the concrete.

Remedial measures after the concrete has hardened consist of sealing the cracks in order to protect the reinforcement. However, consolidation of concrete immediately after the cracks start to form is by far the best course of action.

5.2.3 Repair of Thermal Cracks

Thermal cracks are formed due to the temperature variation between the external environment and the interior of the concrete in regions where day- and night-time temperatures show large variation.

These types of cracks should be sealed to protect the reinforcement by brushing in cement or low viscosity polymers.

5.2.4 Repair of Dormant or Dead Cracks

Dead cracks generally result from an unexpected event, such as accidental overload, and they may usually be locked in such a way as to restore the structure as nearly as possible to its original uncracked state.

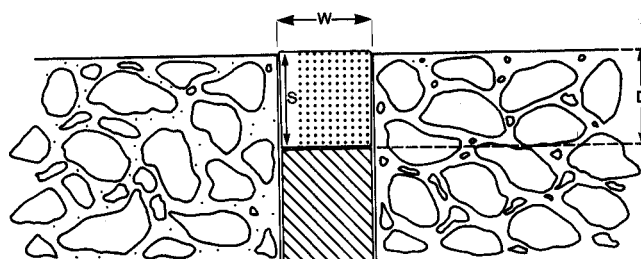
Cracks wider than about 1.0 mm on the horizontal surfaces can usually be sealed by filling them with cement grout. This is commonly done by brushing in dry cement followed, if necessary, by light spraying with water. This treatment often seals the upper part of a crack against ingress of moisture and other aggressive species. For cracks wider than about 2 mm, it may be preferable to use a cement and water grout. Alternatively, cracks can be widened to a width of 5 to 10 mm and pointed up with cement and mortar. Such a procedure will be more costly because of the additional labor required, but it ensures that the seal penetrates to a deeper portion than only when the crack was sealed, without widening, by grouting. Low-viscosity liquid polymers can be used in a similar way to cement grout. It may be possible to obtain an adequate seal by brush application or, on level surfaces, temporary bunds can be formed with clay, plasticene, or similar material to surround a crack so that it can be flooded with polymer. When no further liquid will penetrate the crack, the surplus material and the bunds are removed. Some of these materials will penetrate cracks down to about 0.1 mm in width. Such a repair, however, will not be structural, as the possibility of not sealing the cracks completely is a possibility.

Fine cracks in the soffits or vertical surfaces may be sealed by injecting a polymer. Epoxy resins are most frequently used when the repair is being carried out in order to restore the structural integrity, or when moisture is present. Cheaper polymers, such as polyester resin, can be utilized when the purpose of repair is to protect reinforcement from corrosion. In both the cases, the resin may be injected under gravity or positive pressure; better penetration can be achieved, however, by using vacuum-assisted injection.

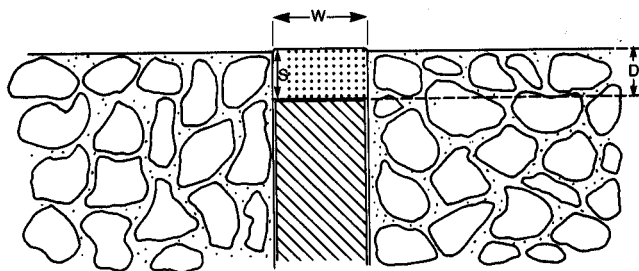
5.2.5 Repair of Live Cracks

If there are signs of continuing movement at a crack with time, it is usually necessary to make a provision for it to continue after repair. This type of crack can be regarded as an unplanned movement joint and, if it is locked solid, another crack will commonly form nearby. One way of achieving this is to cut a chase along the line of the crack. The sealant must then be adhered to the sides of the chase but debonded from the bottom so that the movement is spread over the full width of the chase, as shown in the figure below. A debonding strip of a material, such as smooth plastic is laid in the bottom before the sealant is applied. The depth of the sealant D is equal to S which is the surface

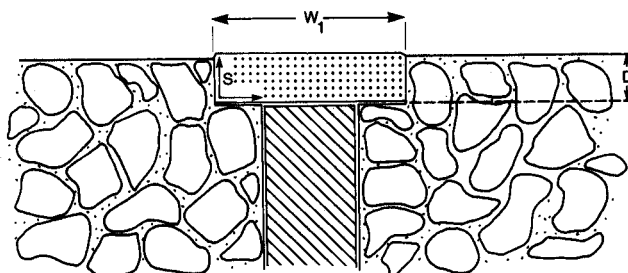
available for adhesion on either side of the joint. W , the width of the joint is equal to D , so any movement which places the sealant in shear or tension will exert considerable stress on the adhesive interface with the concrete. If movement is excessive, the seal will probably fail. The second diagram shows a better situation where, although D is still equal to S , the width of the sealant is twice that value. This means that for any one situation, the force exerted will be considerably reduced. In the third diagram, S has been doubled, and the top surface of the sealant W_1 is twice the value of W . The depth of the seal is half the width of the joint, half of the area available for adhesion and a quarter of the top surface measurement. In this situation the face seal will cope with extensive movement without exerting excessive stress on the adhesive surfaces. Alternatively, the sealant may consist of a surface bandage of elastic material which adheres to the concrete at its edges only. Alternatively, the sealant may consist of a surface bandage of elastic material that adheres to the concrete at its edges only. This may be a pre-formed strip, or it may consist of several layers of a high-build coating material with suitable elastic properties. In either case, it is applied over a slightly narrower strip of debonding material.



a.



b.



c.

Expansion joint detail

5.2.6 Repair by Vacuum Impregnation

In cases where a large number of cracks occur over an area, the affected part of the structure is enclosed within an airtight plastic cover by fixing it to the edges of the structures. A vacuum is then applied so that air is exhausted from all cracks and crevices in the concrete within the cover. Resin grout is then admitted and atmospheric pressure forces it into cracks and pores in the concrete surface. In order to ensure that the resin can flow into the whole of the area under the cover, a layer of net material is first fixed so as to provide a slight space between the cover and the concrete surface. On completion of impregnation, the cover and the net are removed before the resin hardens.

5.2.7 Resin Injection

Cracks in reinforced concrete greater than approximately 0.3 mm may require sealing/injecting to prevent ingress of moisture, oxygen, salt solutions and corrosive chemicals, or to protect the integrity of the structure. Before deciding the most appropriate methods/materials for repairing/sealing cracks, it is imperative to establish the cause of the cracking. It is possible to restore the structure to the original tensile/shear strength by injection with low viscosity epoxy resins specifically developed for repairing cracks, provided the bonding surfaces of the concrete at the crack interface are clean and sound. Cracking is caused by tensile stresses and, if these stresses re-occur after crack repair, the concrete may crack again. If it is not possible to establish and rectify the cause of the original cracking, it is recommended to cut out along the surface of the crack and treat it as a normal movement joint or, alternatively, to cut out a normal straight movement joint adjacent to the crack and then repair the crack by resin injection. This will involve filling with a flexible joint sealant using recommended procedures. In general, a bond breaker should be introduced at the base of the cut out so that a three-sided joint is avoided. The cut out joint should be sufficiently wide so that the predicted movement should not exceed approximately 20% of the minimum joint width. The use of a very low modulus system to fill the crack, as a cheaper alternative, is not recommended for filling fine cracks liable to movement, since the filling material is required to exhibit virtually infinite elongation over a very short width, which is for all practical purposes, impossible. Low viscosity epoxy resin systems are generally used for the structural repairs of cracks. Low viscosity acrylic or polyester resins are also used, but in general, give lower bond strengths and do not bond as reliably under damp conditions as epoxy resin injection systems specifically developed for use under wet conditions. In cases where it is required to fill/repair a network of cracks with 'dead ends' or honeycombed concrete, a combination of vacuum to remove the majority of air in the cracks/voids and pressure injection has proved most effective in some instances.

When it is necessary to ensure that a sealer penetrates to the full depth of a crack, injection of resins under pressure is the method commonly used. Epoxy resins are most frequently used when pressure injection is necessary or when the repair is done to restore structural integrity. However, other resins may be used when crack injection is done solely for purposes of sealing the structure to prevent moisture ingress and protect reinforcement. Epoxy injection is an effective method for repairing cracks in structural members such as walls, piers, floors, ceilings, etc. The process can restore the structure to its original monolithic condition, and to a large measure, structural strength is regained. It will not however, remove the causes of cracking and this should be eliminated in order to effectively repair the crack.

Figure 5.2.1 is an illustration of epoxy injection in cracked concrete.

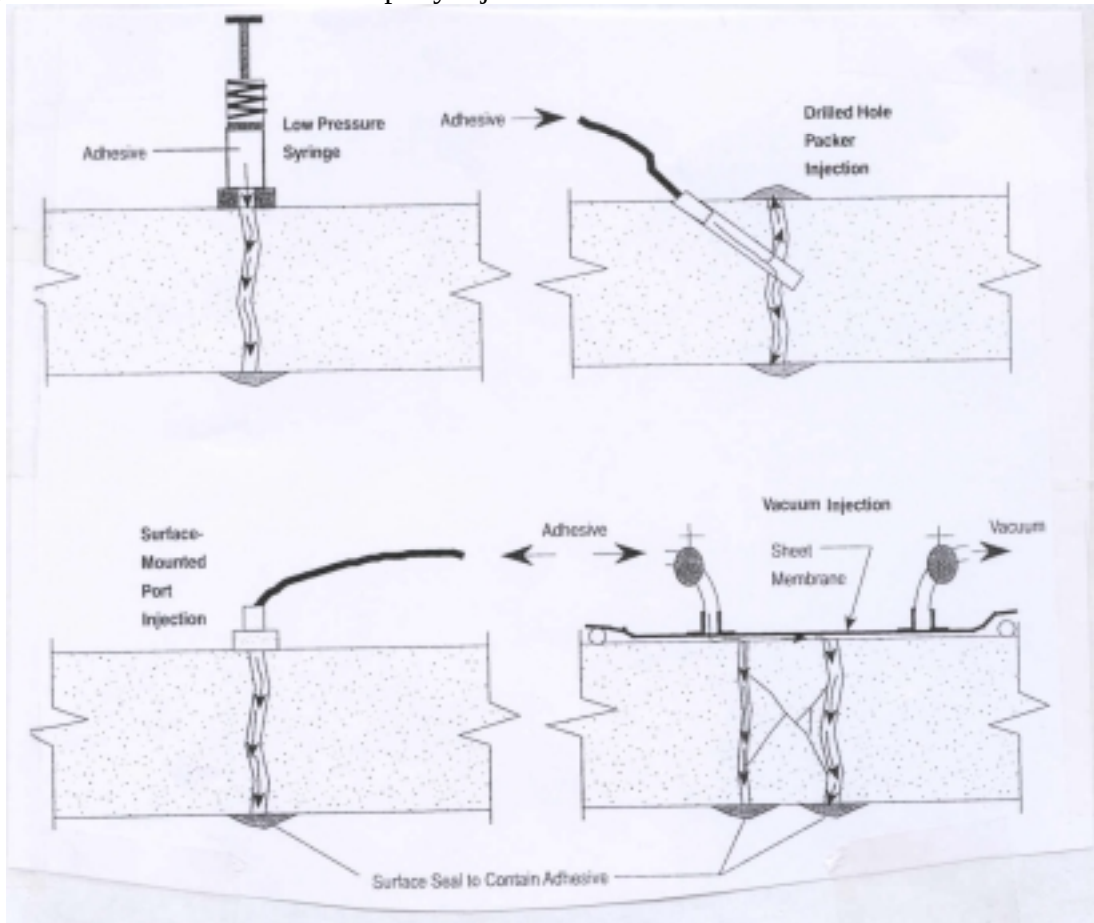


Figure 5.2.1. Repair of cracks by epoxy injection.

5.2.8 Repair of Surface Defects

Blowholes, voids, honeycombs, and cold joints usually form due to inadequate consolidation. When the bubbles of air or water are trapped against the face of the formwork, blowholes are formed. Voids and honeycombs in the concrete are a sign of either inadequate consolidation or loss of grout through joints in the formwork or between formwork and previously cast concrete.

Following procedure may be adopted for repair of concrete with voids and honeycombs:

1. Cut out the affected concrete and replace it with new concrete. If complete cutting out of the defective concrete is not possible, a seal can be formed by injecting a low-viscosity resin into the concrete.
2. Resin injection alone, without any cutting out, may be adequate if the prevention of leakage or protection of reinforcement is all that is required.
3. However, a combination of both methods may be necessary for structural reasons or for the sake of appearance.

4. Cement mortar prepared with fine sand passing either 300 or 600 μm or crushed limestone fines may be used. Mix proportions can be 1:1 or 1:2 and incorporate a polymer admixture. The w/c ratio may be between 0.4 to 0.45.
5. The mortar should be applied over the whole area of the concrete with a rubber-faced float, and finally it can be rubbed down with a smooth stone or mortar block for a smooth finish.

5.2.9 Repair of Inadequate Cover

The following procedure may be adopted to repair concrete with inadequate cover:

1. The concrete cover can be increased by increasing the face of the concrete with a rendering and, if a polymer-modified cement and sand mix is used, it may be possible to provide adequate protection to the reinforcement with a slightly reduced thickness. But it will be necessary to ascertain that there is an adequate key.
2. If the face of the concrete is weak, it should be scabbed or grit-blasted to provide a sound, roughened surface. If it is sound but smooth, it will usually be cheaper to apply a coat of polymer-modified cement and sand, in proportions of about one part of cement to two parts of sand, before applying the first coat of rendering.
3. If it is not possible to increase the dimensions of the structure, a surface coating can be applied. Such a coating should be of a type having low permeability to carbon dioxide and moisture, but it should preferably allow water vapor to escape from within the concrete.

5.3 Repair of Deteriorate and Cracked Concrete due to Sulfate Attack and Salt Scaling

Following procedure is recommended for repairing concrete structures affected by sulfate attack or salt scaling:

1. The unsound concrete that has deteriorated due to sulfate attack or scaling should be marked as a rectilinear shaped area. The marked area should be saw cut (or chipped off) to a depth of 12 mm and the concrete removed from this area to reach the sound concrete by sandblasting or other suitable means. If mechanical tools are used, damage to the surrounding sound concrete should be avoided. In case of reinforcement corrosion, concrete should be removed to a depth of 25 mm behind the bars.
2. Prior to the application of the repair material, the surfaces of the existing substrate should be roughened to provide an adequate key for bonding agent or repair materials. The surfaces should be made free of loose, broken and unsound material.
3. The substrate should be washed with potable water to remove dust and loose material.

4. For repairs using a cementitious material, a bonding agent should be applied on the old concrete. Alternatively, the substrate may be saturated with potable water and kept wet for 24 hours prior to placement of the repair material.
5. The repair material should be placed when the bonding agent is still tacky.
6. Patching mortars can be used for the repair of concrete affected by sulfate attack and scaling to a depth less than 50 mm. These may be used for the hand-patching of horizontal, vertical, and overhead surfaces. The repair material should be placed in layers, and the maximum thickness of each layer should be as recommended by the manufacturer of the repair material.

5.4 REPAIR OF CRACKS CAUSED BY ALKALI – SILICA REACTION

Map cracks due to alkali-silica reaction (ASR) begin to form one to ten years or more after casting and they continue to get progressively deeper and wider. A gel usually exudes from cracks and hardens into a brittle white material.

Map cracks can be avoided by selecting a quality limestone aggregate before the construction so that ASR reaction does not take place in concrete.

The procedures for repair of cracks due to ASR are similar to that utilized for repair of concrete affected by sulfate attack or salt scaling.

5.5 REPAIR OF CRACKS AND DAMAGE CAUSED BY DYNAMIC LOADING AND VIBRATIONS

Structures fail and crack due to excessive loading beyond the load-bearing capacity of the reinforced concrete elements. Excessive loading can be static due to faulty design and construction or dynamic caused by vibrations in industrial plants or earthquakes, in general.

Following a damage assessment, the structure should be demolished and reconstructed if the damage is severe and extensive, as the repair will not be cost effective, and the safety of the structure will not be restored

The five types of structural strengthening methods are as follows:

- Internal restoration,
- Exterior reinforcement,
- Exterior post tensioning,
- Jackets and collars, and
- Supplemental members.

Each method is well suited to a particular field condition.

Internal restoration: Internal dowel-type of repairs cannot be made in deteriorated concrete. The concrete must be strong enough to develop a full bond with the reinforcing steel. This requires that the concrete strength is determined before each repair.

The most common way to provide strengthening across cracked sections of structural members is to install a new interior reinforcement. The dowels are usually deformed steel bars, stainless steel bars or bolts. Galvanized or epoxy coated steel rods or graphite fiber reinforced or glass fiber reinforced plastic bars are also acceptable so long as they are chemically compatible with the bonding agent.

Figure 5.5.1 illustrates the method of internal restoration of a cracked structure.

Exterior reinforcement: Steel plates or channels are effective in stopping the spread of cracks. They can also be used for providing exterior reinforcement for structural elements. Steel plates (or glass fiber graphite fiber reinforced plastic wrapping) will provide excellent shear and moment resistance when bolted or epoxy bonded to the cracked area of the structural members, such as overhead beams or columns. In the case of walls, failure can be caused due to the following reasons:

- a) Pivotal settlement,
- b) Differential settlement, or
- c) Lateral pressure.

The failed wall can be rehabilitated by steel rod staples. In both the repair methods, the arrangement of drilled holes should be staggered to avoid creating a line of weakness.

Figure 5.5.2 is an illustration of a reinforced concrete beam strengthened with a bonded steel plate.

External post tensioning: Pre-stressing strands or tie rods with threaded ends may be used very effectively as external post tensioning. The post tensioning tie rods or strands can be steel or graphite fiber reinforced plastic rods.

Figure 5.5.3 is an example of external post tensioning.

Jackets or Collars: Concrete members that are cracked or deteriorated throughout their entire cross-section, may be restored either by constructing a new reinforced concrete collar or by installing a series of tensioned steel straps around the existing member. Concrete jackets are commonly utilized to restore cracked or deteriorated concrete compression members such as columns and piles. Beams that have failed in shear can be restored by installing a series of tensioned steel straps around the cracked section. Glass fiber or graphite fiber reinforced plastic rapping also can be used for retrofitting, such as structural members. These exterior reinforcing methods are commonly used together with epoxy injection of the cracks.

Figure 5.5.4 is an example of rehabilitating a deteriorated concrete component by the use of external strap.

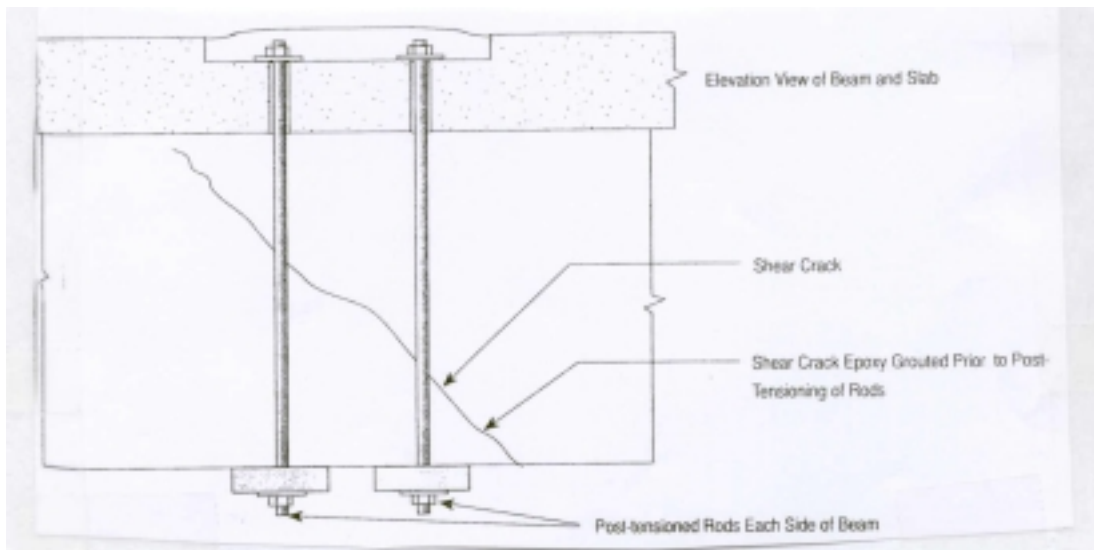


Figure 5.5.1. Internal restoration of a cracked structure.

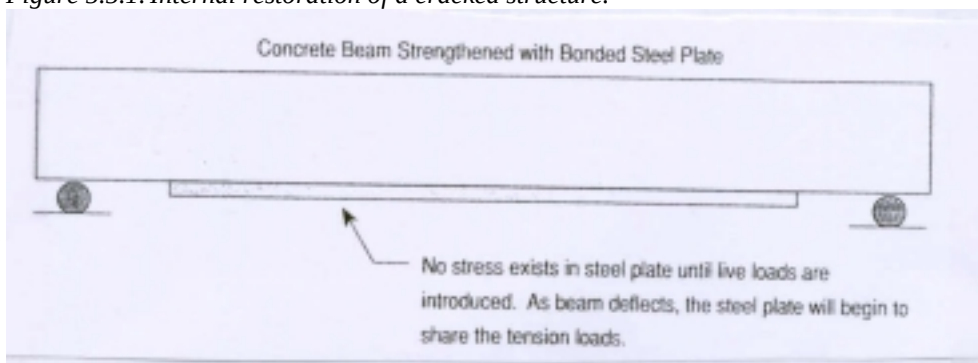


Figure 5.5.2. Reinforced concrete beam strengthened with a bonded steel plate.

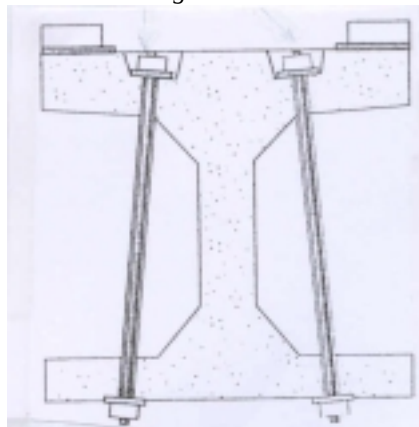


Figure 5.5.3. External post tensioning of a beam.

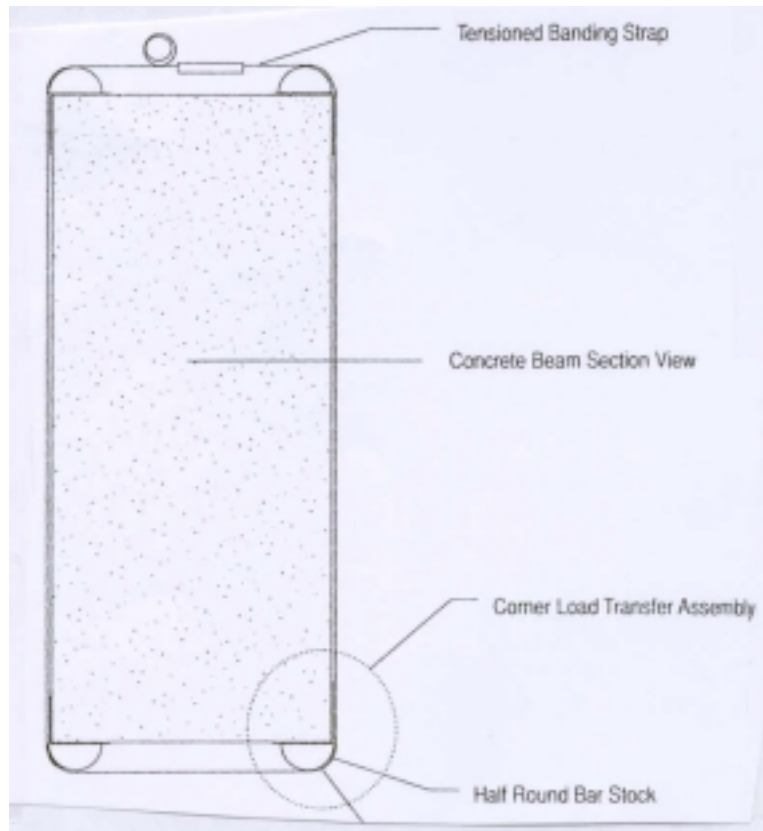


Figure 5.5.4. Rehabilitation of a deteriorated concrete component by the use of external strap.

Supplemental members: These are simply new columns or beams installed to support damaged structural members or systems. They are obvious and distracting, and therefore, are the least desirable methods of repair.

The five structural repair methods discussed above can be used singly or in various combinations. If none of the repair methods is feasible, then it may be necessary to either reduce the allowable working loads on the area or remove the damaged area and completely rebuild that part of the structure.

5.6 REPAIR OF DETERIORATION DUE TO EXPOSURE TO CHEMICALS

Following procedure is recommended for repairing concrete structures affected by chemical exposure:

1. The unsound concrete should be marked as a rectilinear shaped area. The marked area should be saw cut and the concrete removed from this area to reach the sound concrete by sandblasting or other suitable means. If mechanical tools are used, damage to the surrounding sound concrete should be avoided. In case of reinforcement corrosion, the concrete should be removed to a depth of 25 mm behind the steel bars.
2. Prior to the application of the repair material, the surfaces of the existing substrate should be roughened to provide an adequate key for the repair material. The surfaces should be made free of loose, broken and unsound material.

3. The substrate should be washed with potable water to remove dust and loose material. Alternatively, the dust and loose material may be removed by air blasting.
4. For repairs using a cementitious material, a bonding agent should be applied on the old concrete. Alternatively, the substrate may be saturated with potable water and kept wet for 24 hours prior to placement of the repair material.
5. The repair material should be placed when the bonding agent is still tacky.
6. Resin-based repair mortars can be used for the repair of concrete affected to a depth less than 50 mm. These may be used for the hand patching of horizontal, vertical, and overhead surfaces. The repair material should be placed in layers, and the maximum thickness of each layer should be as recommended by the manufacturer of the material.
7. If cement-based repair materials is utilized it should be cured by covering the repaired surface with a wet burlap for a minimum of seven days.
8. An acid-resistant coating should be applied on the repaired surface after it has been cured and dried.

5.7 REPAIR OF DETERIORATION AND CRACKING DUE TO EXPOSURE TO HIGH TEMPERATURE AND FIRE

High temperatures, above 100 °C, will dry the concrete and will disintegrate the aggregate and cement mortar due to high temperature oxidation of inorganic matter and burning of organic components. Excessive exposure of concrete structures to high temperatures will deteriorate the integrity of the concrete and reduce its compressive strength. Therefore, concrete structures exposed to a fire should be thoroughly inspected and the compressive strength of all structural members should be determined before any decision is taken for the usability of the structure after a fire.

The purpose of repair to a fire damaged structure is to restore in the repaired structure the performance it had before the fire, both in relation to strength and fire resistance. The possibility of repair and consideration of the repair method cannot be known until a comprehensive damage evaluation is conducted. The damage survey will establish the type, extent, and specific location of damage. Once the damage to the concrete is defined, and the specification for repair drawn up, removal and replacement of the damaged material can be initiated.

5.7.1 Materials

The cost viable and most commonly used materials for the repair of fire damaged structure are cementitious mortars and concrete. Since the resin- and polymer-based materials may soften at relatively low temperatures (80 °C) it is possible that they may neither provide adequate fire protection to the reinforcement nor will they be able to retain structural integrity at temperatures encountered in a fire. Consequently, the use of resinous repair materials is restricted to the following situations:

1. When the material is adequately fire protected by other materials and retains its structural properties at the expected fire temperatures at the relevant depth in the section.
2. Loss of strength or other properties of the material will not cause unacceptable loss of structural section or fire resistance.

Resin- and polymer-modified mortars are chiefly utilized in hand repairs of small areas between 12 and 30 mm depth. However, polymer-based fire protection coatings are used to protect the structural steelwork. These coatings are specially formulated materials, containing fillers, which expand on exposure to heat, and depending on the thickness of cover, are capable of keeping heat and flame at bay for up to 2 hours.

5.7.2 Method of Repair

The repair procedures chosen can be grouped into three different techniques: (1) structural repair, (2) surface repair, and (3) stiffening.

STRUCTURAL REPAIR

If the cracks in the concrete are considered to be of a nature that would lead to serious structural problems, then a structural repair, using epoxy injection techniques, should be specified. Once all the loose particles are removed to expose even the finest cracks, all the cracks are sealed with a fast setting, cementitious, patching compound that cures within minutes after application. A low viscosity, structural adhesive can then be injected into the cracks to their full depth. This effectively "welds" the concrete back together to a strength greater than the concrete itself.

SURFACE REPAIR

Cementitious repair materials, such as concrete and mortar, are generally utilized for surface repairs. They are placed using the three following methods:

- Recasting in formwork
- Spraying (shotcrete)
- Hand applied mortars

The choice of the method is usually determined on practical and cost considerations. Spraying, or recasting, will be more suited for large volumes, large area applications, and where speed is required. Hand applied mortars are more suitable for patch repairs and lesser volumes.

The method of recasting in formwork is particularly used when larger volumes of material are to be placed and a high standard of surface finish is required. The formwork should be constructed to provide a suitable minimum thickness so that the concrete can flow and attain the required reinforcement cover. It must be well sealed against the existing structure, rigidly and firmly fixed. In order to achieve placing and compaction in the restricted situation, a high workability concrete with appropriate maximum aggregate size should be used. The required high workability can be obtained by the use of a water reducing or superplasticizing admixture.

Repair by hand applied cementitious mortar is similar to rendering application with the exception that a slurry bond coat is used in the former. The slurry coat usually consists of a 1:1 diluted latex mixed with cement to produce a thick, creamy consistency. In some instances, sand is added to the slurry mix to provide a stippled surface texture that improves the bond of the mortar to the concrete. The slurry grout coat of cement/latex is applied by brush to the prepared concrete surface, which is dampened just prior to placing of the mortar to prevent excessive suction. The repair mortar is then placed while the bond coat is tacky. Where thickness greater than 25 mm are required, the repair should be built up in layers of about 25 mm maximum thickness to avoid slumping of the mortar. The surface of the previously placed layer should be furrowed, and subsequent layers applied, as soon as the previous layer has set sufficiently to accept a further layer without disturbance. The final surface should be finished to a smooth surface with a steel trowel.

STIFFENING

Often, repairs to structural elements will necessitate upgrading of the older structure to comply with present day specifications. For example, some older specifications may not adequately address the deflection of suspended floors. Consequently, when there is severe damage to the ceiling or the floors in some older structures, stiffening of the structure would be required. Stiffening of a suspended floor can be done by the incorporation of dowels bonded to the concrete by epoxy adhesive, welding of steel mats to the dowels, and placing of a 50 mm, high strength (50 MPa) topping.

5.8 REPAIR OF SPALLED CONCRETE

5.8.1 Hand-applied Repairs

Following is the procedure for hand-applied repairs to spalled concrete:

1. Remove the unsound concrete. The area to be cut out should be delineated with a saw cut to a depth of about 5 mm in order to provide a neat edge, the remainder of the cutting out can be done with percussive tools. If any corroded reinforcement is present, the concrete should be cut back far enough to ensure that all corroded areas are exposed so that they can be cleaned. When the bars are of small diameter, i.e. 12 mm and below, they should be exposed to the full perimeter.

If the cause of deterioration is reinforcement corrosion due to carbonation, carbonated concrete must be cut back. Whenever extensive cutting is required, temporary structural support must be provided.

If reinforcement corrosion is due to the presence of chloride at the steel-concrete interface, all the contaminated concrete should be removed. This may sometimes involve the complete removal of a structural member or even the demolition of the structure. The half-cell potential contours could be used to identify areas where future corrosion is most probable, in which case they also should be cut out and repaired.

2. Dust should be removed, as far as possible, from the surface of the structure. Oil-free compressed air jets are effective on small areas, but they tend merely to

redistribute the dust on large areas. For these, industrial vacuum cleaners are more effective.

3. Apply a protective coating to the reinforcing steel. The type of material to be used will be governed largely by the repair material selected. Slurry coating of polymer latex and cement can be used if a cement-based repair is to follow. Resin-based coatings that are suitable for use with cement-based and with resin-based repair materials are available, and these are sometimes blended with coarse sand in order to provide a key for the subsequent patch.
4. After the surface to be repaired has been prepared, a bonding coat should be applied to all the exposed surfaces. The surface of the steel bars should be cleaned before the application of the bond coat. The bond between the old concrete and the new concrete can be achieved by saturating the prepared concrete surface with water. In addition to this, the use of a bonding coat is advisable. It can consist of slurry of a cement and water only, but it is nearly always desirable to incorporate a polymer admixture. Typical proportions would be two parts of cement (by volume) to one part polymer latex, but the supplier's advice may vary. Alternatively, some polymers are used alone, without any cement addition.
5. The first layer of the patching material should be applied immediately after applying the bond coat. This is most important because a bonding coat that has been allowed to dry out will reduce the bond instead of increasing it. If some delay is inevitable, there are some resin-based bonding agents that have a longer 'open time' than the cement slurry. There are also some resin coatings that are blended with coarse grit while they are still sticky and then allowed to harden, in which case the grit provides a key for the subsequent patch. Resin coatings do not in themselves provide an alkaline environment in immediate contact with steel reinforcement. However, there are some proprietary systems, in which a resin containing Portland cement is used, which may provide some alkalinity.

Hand repairs usually consist of cement and sand mortar in proportions of 1:2.5 to 1:3. Lightweight fine aggregates are sometimes used, especially in the overhead works, and some proprietary pre-packed materials that contain cement and specially graded sands in correct proportions are available. It is usually beneficial to incorporate a polymer admixture, such as styrene-butadiene rubber (SBR) or an acrylic emulsion, to improve adhesion to the substrate and increase the strain capacity of the repair mortar, thus reducing the risk of debonding or cracking as a result of shrinkage or thermal stresses. A commonly used concentration is 10% polymer solids in the mortar but, as usual, the suppliers' instructions should be followed.

Polymer-modified cement mortars should usually be mixed using a forced action mechanical pan mixer, because of the tendency of the polymers to entrain air in the mix. Repairs should be built up in layers, and each layer should normally be applied as soon as the preceding one is strong enough to support it. The thickness of each layer should not normally exceed 20 mm. If there is likely to be a delay between layers, the first layer should be scratched to provide a key and a fresh bonding coat should be applied when the work is resumed.

6. A protective coating should be applied on the repaired surface after it has been adequately cured and dried. The surface preparation and the coverage rate should be that specified by the manufacturer.

Figure 5.8.1 is an illustration of hand applied repair.

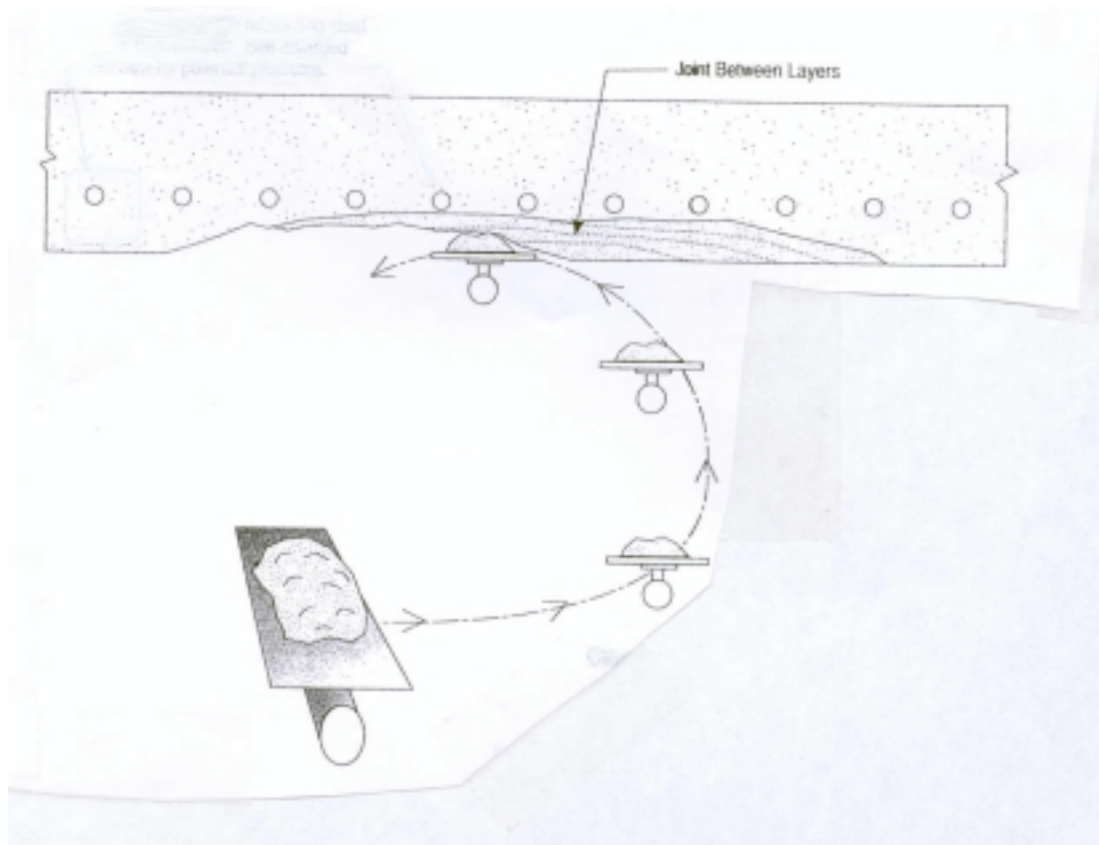


Figure 5.8.1. Illustration of a hand applied repair.

5.8.2 Large Volume Repairs

This is a conventional method of repair and is advisable when the hand application or spraying of the repair material is not feasible. In this type of repair, it is necessary to fix some kind of formwork and fill it with concrete or grout. The method of preparing the surface to be repaired is similar to that outlined in the patch repair. The defective concrete must be removed so that the sound surfaces are exposed, and reinforcing steel cleaned. Further, the surface to be repaired is cleaned with compressed air or water immediately before the repair material is placed.

Formwork can be of a conventional rigid type, that either encloses the member to be repaired, or is sealed to it at its edges, or it may consist of a flexible fabric, particularly when grout is to be used. In grouted aggregate work, transparent panels are sometimes

provided so that the progress of grouting can be watched. When mixed concrete is to be used for the repair, provision must be made in the formwork for placing and consolidating it. This means that the formwork has to be built up in stages as the work proceeds or temporary openings can be provided in the forms through which access can be obtained. All the joints between the sections of formwork, and between the formwork and the existing concrete, must be sealed so as to prevent leakage, and the seal must be maintained while the concrete is being consolidated. When the grout is used, some provision must be made for venting air at the top as the grout rises.

When conventional concrete is utilized, the mix design will depend partly on the dimensions and location of the repair. In components where there is an easy access for placing and consolidating concrete and for which the thickness of the repair is 100 mm or greater, a mix containing 20 mm maximum size aggregate is commonly necessary. Addition of a superplasticizer may be necessary when consolidation is difficult. The method of placing the concrete is similar to the new construction. Consolidation is best achieved by internal vibration if there is access for vibrators. A limited amount of external vibration may sometimes be required in conjunction with the internal vibration to achieve complete consolidation.

5.8.3 Grouted Aggregate Repair

In this technique, coarse aggregate is filled into the spaces between the formwork and the structure, and grout is then pumped in to fill the interstices between the aggregate particles. The grout should be introduced at the lowest points of the formwork in order to prevent the formation of air pockets. Injection tubes may be built into the formwork at several levels if complete filling from the bottom would require too great an injection pressure. Alternatively, injection pipes may be inserted from the top, reaching to the bottom of the formwork, before the aggregate is placed. They are gradually withdrawn as the level of grout rises. The aggregate grading must be such that the grout can flow freely between the particles. This usually means that single-sized aggregate, generally 20 mm or larger, should be used. This method is particularly suitable for underwater work and for conditions where access is difficult.

Figure 5.8.2 is an illustration of the process of grouted aggregate repair.

5.8.4 Repair by Sprayed Concrete

This type of repair is suitable when the concrete cannot be formed by conventional techniques. In this method of repair, a thin layer of high quality fine concrete is sprayed on the surface of a structure to which it will bond strongly, restoring the protective cover to the steel reinforcement, making good concrete that has spalled or become abraded.

Shotcrete typically contains well-graded aggregates of 10 mm or less in size. The water to cement ratio is usually 0.4 or less. Shotcrete can add strength to weakened structures damaged by chemicals, weather, fire, and overloading.

Modifications to the physical properties such as improved durability and adhesion to the substrate are obtained by the use of admixtures, fiber reinforcement, and pozzalanic materials. For example, superplasticizers and latex admixtures, when included in the mix, reduce permeability and rebound, and improve adhesion to the substrate. Silica fume, in

like manner, reduces porosity and rebound waste and improves strength and durability. Steel fibers improve the mode of failure permitting large deformations to be sustained prior to complete failure. Deformed steel fibers are usually used in amounts of approximately 1 to 2% by cement volume.

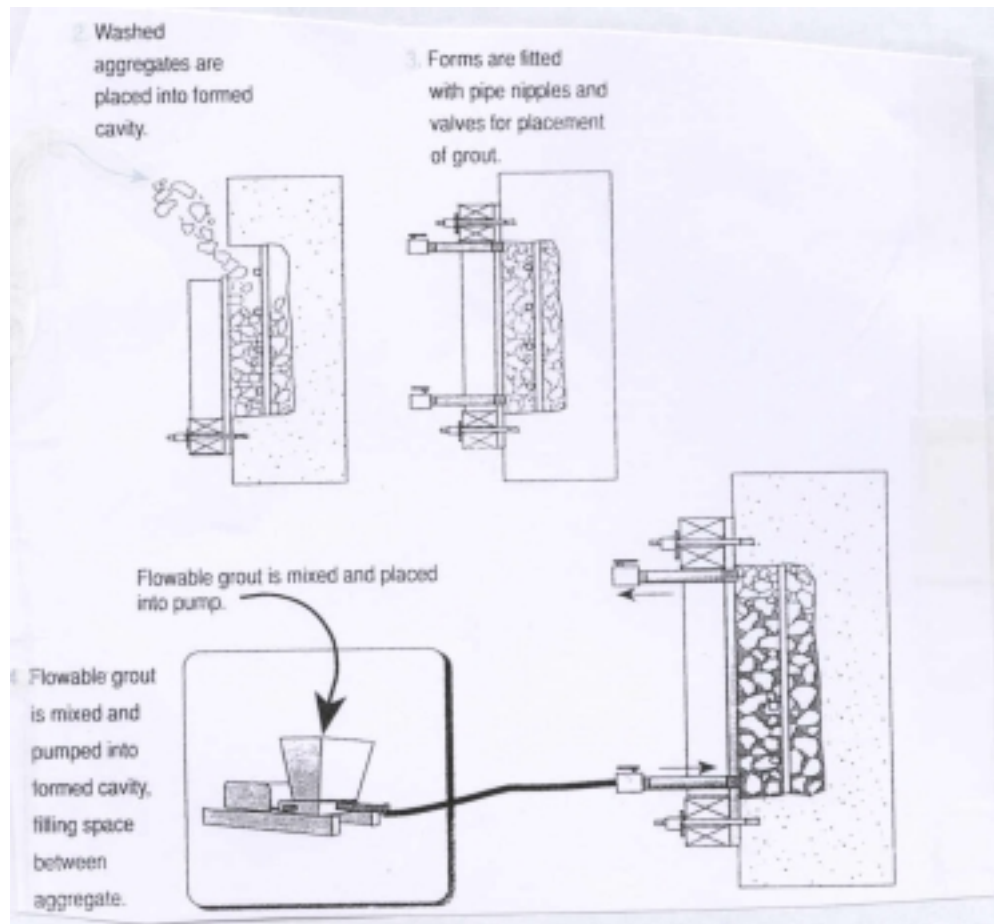


Figure 5.8.2. Illustration of the process of concrete repair by grouted preplaced aggregate.

Surface preparation is the most important factor affecting the quality of a shotcrete repair and it is essential to completely remove all spalled, deteriorated, loose, and unsound concrete prior to applying the shotcrete.

Since shotcrete is most often used in vertical and overhead applications, light chipping hammers are used (usually 7 kg or less) to reduce operator fatigue and damage to the sound concrete. Usually a minimum of 25 mm of concrete is removed and the perimeter of the area is cut 25 mm deep to provide good mechanical interlock to the existing concrete.

There are two processes for shotcreting-dry and wet mix methods.

Dry Mix: A summary of the dry mix process is as follows:

- Site batched cement and aggregates or packaged premixed materials are thoroughly mixed in a transmit mixer, volumetric proportioning mixer.
- Water is added, in some cases, to bring the shotcrete mixture to a dry consistency of 3 to 6% moisture.
- The mix is then transferred to the shotcrete delivery equipment and compressed air is used to convey the shotcrete down the hose of the gun.
- Water is introduced under pressure at the nozzle.

- Shotcrete is expelled from the nozzle at high speed onto the concrete surface. The force of the impacting jet of shotcrete compacts the underlying material in place.

Wet Mix: Wet mix shotcrete contains a predetermined ratio of cement, aggregates, water, and admixtures previously batched and mixed. This mix is discharged into a conventional concrete pump and pumped to a discharge nozzle. Compressed air is used at the nozzle to project the material at high velocity onto the concrete structure. A rapid setting accelerator is normally added at the nozzle to accelerate the set of the material to facilitate build-up of thicker layers without sagging and sloughing off.

Figures 5.9.1 and 5.9.2 illustrate the process of concrete repair by dry mix shotcrete and wet mix shotcrete, respectively.

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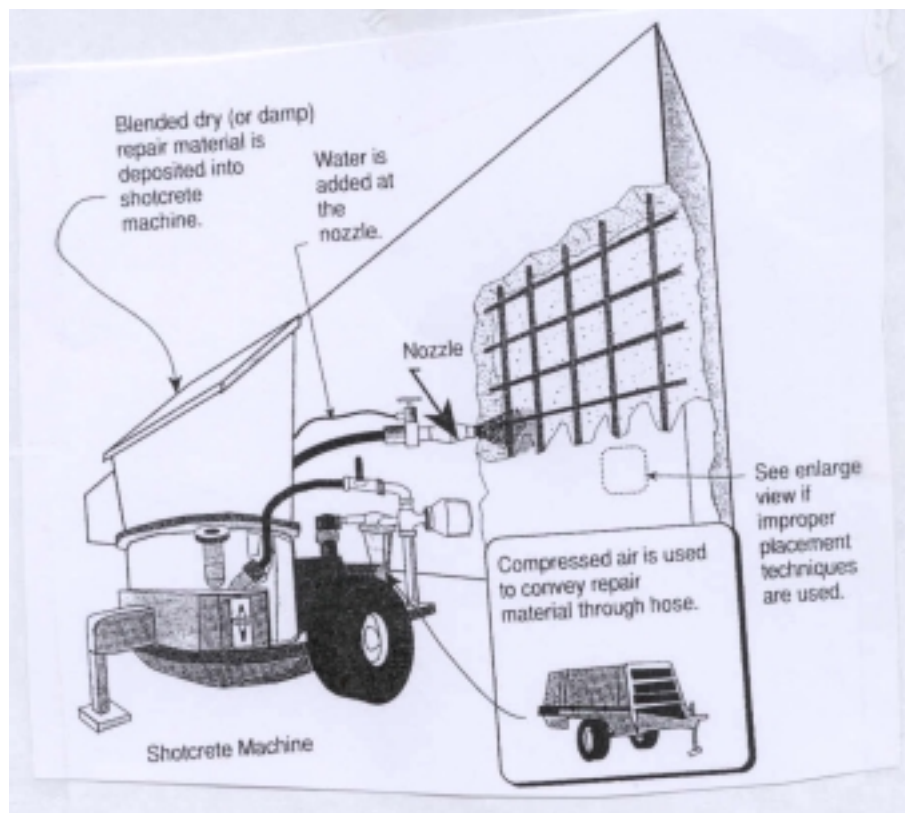


Figure 5.9.1. Illustration of concrete repair by dry mix shotcrete.

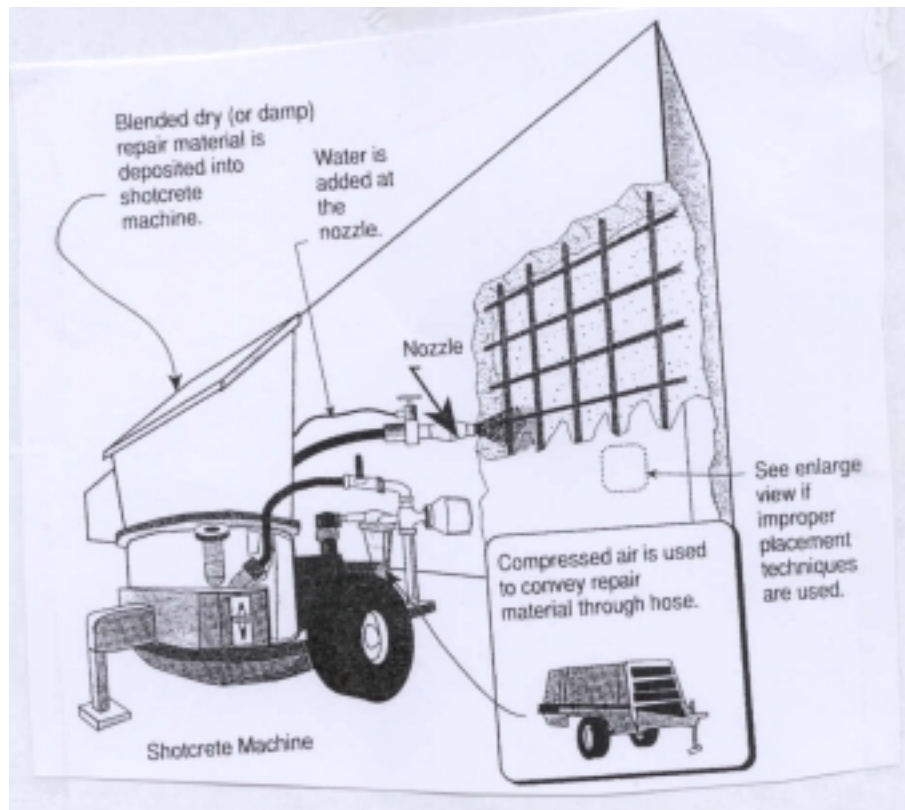


Figure 5.9.2. Illustration of concrete repair by dry mix shotcrete.

CHAPTER 6

REPAIR SYSTEMS FOR SERVICE ENVIRONMENTS IN SAUDI ARAMCO

6.1 INTRODUCTION

The repair systems commonly utilized to repair deteriorated concrete structures are shown in Table 6.1. The repair systems that are appropriate for repairing concrete structures in Saudi Aramco are discussed in this chapter. More than one repair system is suggested for each of the exposure conditions. This will provide flexibility with regard to the choice of an appropriate repair system depending on the size of the repair. For example, a certain repair system may be appropriate for small repairs while the other may be more appropriate for large repairs. Further, it is possible that the recommended systems would be updated with advancement in technology.

6.2 REPAIR SYSTEMS FOR REPAIR OF MARINE STRUCTURES

Concrete deterioration due to reinforcement corrosion is a major possibility in the structures exposed to marine conditions. The deterioration in these structures will be mainly concentrated at the splash zones due to the availability of both oxygen and moisture.

Two repair systems, namely, system 7 and system 1, are recommended for repairing concrete structures exposed to marine conditions. System 7 is more appropriate for large repairs while system 1 may be utilized for medium to small repairs.

The following procedure should be adopted when system 7 is utilized.

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the old concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregates. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal.
3. Water should be sprinkled on the exposed surface so that the concrete is in a saturated condition. The excess water should be removed by blowing air.
4. Site batched cement and aggregates or packaged premixed materials may be used. Sprayed concrete should be a dry mix with maximum water to cement ratio of 0.38. To minimize rebound, the ingredients should be thoroughly blended.
5. Shotcreting should be done by experienced workmen. The thickness of shotcrete should be as constant as possible. Abrupt changes will decrease the bonding capability and increase chances of forming voids or a poor quality shotcrete.

Table 6.1. Description of the repair systems.

System	Repair mortar	Bond coat	Rebar primer	Surface coating *
1	Free flowing micro-concrete	Wetting only (saturated surface dry condition)	Single-component zinc-rich epoxy	Chloride/Sulfate barrier
2	Pre-bagged acrylic modified mortar	3-component epoxy resin and modified cement based slurry	Single component zinc-rich epoxy	Chloride/Sulfate barrier
3	Portland cement mortar/concrete (max. w/c ratio = 0.38)	Wetting only	Composite cement epoxy	Chloride/Sulfate barrier
4	Portland cement/micro-silica mortar (max. w/c+s = 0.38) (micro-silica = min 5% of total cement)	Portland cement/micro-silica slurry (proportions as mortar)	Composite cement epoxy	Chloride/Sulfate barrier
5	Portland cement micro-silica mortar (max. w/c + s = 0.38) (micro-silica = min. 5% of total cement)	Portland cement/micro-silica slurry (proportions as mortar)	Composite cement epoxy	Chemically resistant epoxy
6	Resin mortar	None	Single-component zinc rich epoxy	Chemically resistant epoxy
7	Sprayed concrete (dry mix) Portland cement (max. w/c = 0.38)	None	None	Chemically resistant epoxy
8	Sprayed concrete (dry mix) Portland cement + micro-silica (max. w/c = 0.38) (micro-silica = min. 10%)	None	None	Chemically resistant epoxy
9	Epoxy injection	None	None	Chloride/moisture resisting, i.e. polymer modified cement
10	Polyurethane injection	None	None	Chloride/moisture resisting, i.e. polymer modified cement

* Penetrating sealers can be applied in lieu of the surface coating for above grade structures and structures that will not be subjected to hydrostatic pressure. Recommended penetrating sealers are: Amercoat 2298-S4, Thorosilane or Sikahaurd 70.

6. The finished surface should be trowelled lightly to produce a plain surface and protected by covering with wet hesian sheets. Curing of the surface should continue for at least seven days.
7. After appropriate curing, for at least seven days, and drying, apply a chemically resistant surface coating on the repaired surface. Surface preparation and the number of coats and the coverage rate of the coating should be as per the manufacturer's recommendations.

All the materials and procedures should conform to Saudi Aramco standards and should be approved by the relevant departments.

System 1 may be utilized for small to medium repairs, such as patch repairs or in situations where shotcreting is not feasible.

The following procedure should be adopted for repairing structures using repair system 1:

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal. Additional reinforcement should be provided if more 25% of the original steel is lost due to corrosion.
3. Apply a single-component zinc-rich epoxy-based steel primer to the exposed reinforcing steel. The rate of application should be that recommended by the manufacturer. The steel primer should be uniformly applied so that there are no pinholes and uncovered areas.
4. After the steel primer has dried, wet the exposed surface by sprinkling potable water so that the concrete is in a saturated surface dry condition. Excess water should be cleaned by an air blast.
5. Apply free flowing micro-concrete to the exposed surface. The thickness of each layer should not be more than that recommended by the supplier. Suitable formwork may be installed to contain the repair material. Cure the repair material by covering the repaired surface with hessian for at least seven days. Alternatively, a curing compound may be applied on the repaired surface.
6. Apply a chloride/sulfate barrier coating after the repair material has dried. The surface preparation, number of coats and the coverage rate should be that recommended by the supplier.

All the materials and procedures should conform to Saudi Aramco standards and should be approved by the relevant departments.

6.3 SYSTEMS FOR REPAIR OF BELOW GROUND STRUCTURES

Concrete deterioration in the structures exposed to below ground conditions may be due to reinforcement corrosion or sulfate attack.

Repair system 6 may be utilized where small areas are to be repaired. However, for the repair of large areas, system 1 may be utilized.

The procedure for repairing below ground structures using system 6 is as follows:

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal. Additional reinforcement should be provided if more 25% of the original steel is lost due to corrosion.
3. Apply a single-component zinc-rich epoxy-based steel primer to the exposed reinforcing steel. The rate of application should be that recommended by the manufacturer. The steel primer should be uniformly applied so that there are no pinholes and uncovered areas.
4. Apply a resin mortar to the exposed surface. The thickness of each layer should not be more than that recommended by the supplier. Suitable form work may be installed, if required, to place the repair material in the repair area.
5. Apply a chemically resistant surface coating after the repair material has dried. The surface preparation, number of coats, and the coverage rate of the coating should be that recommended by the supplier.

All the materials and procedures should conform to Saudi Aramco standards and should be approved by the relevant departments.

The procedure for repairing the deteriorated below ground structures using system 1 is similar to that elaborated in Section 6.2.

6.4 STRUCTURES EXPOSED TO SULFUR FUMES

In the structures exposed to sulfur concrete deterioration will be mainly due to acid attack.

Repair system 6 may be used for repairing concrete structures exposed to sulfur fume. However, in situations where the resin-based repair mortar (system 6) cannot be used, such as structures requiring large repairs, either repair systems 5 or 8 may be utilized.

The procedure for repairing concrete structures exposed to sulfur fumes, using system 6, is as elaborated in Section 6.3. The procedure for repairing structures exposed to sulfur fumes using system 8 is similar to that described for system 7 in Section 6.2 except that 10% silica fume should be used as partial replacement of cement.

The procedure for repairing structures exposed to sulfur fumes, using system 5, is as follows:

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.

2. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal. Additional reinforcement should be provided if more than 25% of the original steel is lost due to corrosion.
3. A composite cement epoxy-based steel primer should be applied to the exposed reinforcing steel. The rate of application should be that recommended by the manufacturer. The steel primer should be uniformly applied so that there are no pinholes or uncoated surface.
4. Apply cement/micro silica slurry to the exposed surface. The water to cement ratio of the slurry should not be more than 0.38 by weight. The micro silica should be 10% of the total cementitious material. The slurry should be uniformly spread so that the thickness is uniform and there are no pinholes or uncovered areas.
5. Place the portland micro-silica mortar in the exposed area before the cement slurry dries out. The micro silica content should not be more than 10% of the total cementitious material. The cement to sand proportion should be 1:2.5 by weight cement and the water to cement ratio should not be more than 0.38 by weight of the cementitious material. Sand, cement and micro silica should be thoroughly mixed in a mechanical mortar mixer of appropriate capacity to obtain a uniform color of the mortar.
6. The mortar should be placed in the exposed area and consolidated by tamping with a steel rod to remove entrapped air. If necessary the repair material may be placed in layers. The repair surface should be leveled with a trowel.
7. Cure the repaired surface either by the application of a curing compound or wet-hessian. In case of wet curing, it should be continued for at least seven days.
8. Apply a chemically resistant surface coating after the repair material has dried. The surface preparation, number of coats, and the coverage rate of the coating should be that recommended by the supplier.

All the materials and procedures should conform to Saudi Aramco standards and should be approved by the concerned departments.

6.5 STRUCTURES EXPOSED TO ACID

Acid spillage is the major cause of concrete structures deterioration in Saudi Aramco facilities. Repair system 6 should be utilized for repair concrete damaged due to acid exposure. However, in situations where the resin-based repair mortar (system 6) cannot be used, such as structures requiring large repairs, either repair system 5 or 8 may be utilized.

The procedures for repairing concrete structures exposed to acid, using either system 6, 8 or 5, have been discussed in Sections 6.2 through 6.4.

6.6 REPAIR SYSTEMS FOR SWEET AND SALINE WATER RETAINING STRUCTURES

Cracking, due to differential exposure temperature within and outside, may be the major cause of concrete in the water retaining structures. Either system 9 or system 10 may be utilized for the repair of sweet and saline water retaining structures.

The following procedure is recommended for repairing sweet water or saline water retaining concrete structures utilizing repair system 9 or 10.

1. Fix injection ports along the length of the crack. These injection ports can either be glued to the concrete surface or inserted into the concrete by drilling holes. When the injection ports are fixed by drilling holes, the holes must be drilled from either side of the crack. The holes must be away from the crack so that they do not split the concrete when the injection port is tightened.
2. Seal the crack with an epoxy putty. Allow the epoxy to set.
3. When using polyurethane injection material (system 10), inject the crack with water to saturate the concrete.
4. Pump the injection material through the injection ports. Manufacturer's instructions should be followed in the mixing of the resin and hardner and the pumping pressure. The injection material should be injected through each injection port till the material flows out of the next one. For cracks in vertical and inclined surfaces injection should start at the lowest point and proceed upwards.
5. Allow the injecting material to set and remove the injection ports. Close the holes of the injection ports with a suitable epoxy or mortar. Grind the putty and the sealant at the locations of inject ports.
6. Apply polymer-modified cement coating on the structure. Manufacturer's instructions as to the surface preparation and the number of coats and coverage rate should be followed strictly.

All the materials and procedures should conform to Saudi Aramco standards and should be approved by the concerned departments.

6.7 REPAIR SYSTEMS OF FIRE DAMAGED STRUCTURES

Cracking, delamination and spalling is the major of form of concrete deterioration that is noted in the structures exposed to fire. Either system 7 or system 8 may be utilized for the repair of structures damaged by fire.

The following procedure is recommended for repair of structures damaged by fire, utilizing system 7 or system 8.

1. Conduct a survey to identify concrete damaged by fire. This can done by using a schmidt hammer or pulse velocity measurements.

2. Remove the concrete weakened by fire with light pneumatic tools. Cutting and removal of concrete should provide a profile suitable for the repair. Provide additional reinforcement if necessary.
3. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal.
4. Water should be sprinkled on the exposed surface so that the concrete is in a saturated condition. The excess water should be removed by an air blast.
5. Site batched cement and aggregates or packaged premixed materials may be used. Sprayed concrete should be a dry mix with maximum water to cement ratio of 0.38. To minimize rebound, the ingredients should be thoroughly blended. When utilizing system 8, the micro silica content should be 10% of the total cementitious material.
6. Shotcreting should be done by experienced workmen. The thickness should be kept as constant as possible. Abrupt changes will decrease the bonding capability and increase chances of forming voids or a poor quality shotcrete.
7. The finished surface should be trowelled lightly to produce a plain surface. The finished surface should be protected by covering with wet hesian sheets. Curing of the surface should continue for at least seven days.
8. After appropriate curing, at least seven days and drying apply a chemically resistant coating on the repaired surface. Surface preparation and the number of coats and the coverage rate of the coating should be as per the manufacturer's recommendations.

All the materials and procedures should conform to Saudi Aramco standards and should be approved by the concerned departments.

6.8 COST ANALYSIS

The cost break down for the repair systems discussed in Sections 6.2 through 6.7 is provided in Table 6.2.

Table 6.2. Cost breakdown for repair systems for service environments in Saudi Aramco.

Work Details	Cost, SR		
	Material	Manpower	Total
SYSTEM 1*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of single-component/zinc rich epoxy on steel bars	20	40	60
Application of free flowing micro-concrete	650	300	950
Application of chloride/sulfate barrier coating on repaired surface	20	20	40
TOTAL COST	690	610	1300
SYSTEM 2*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of composite cement epoxy on steel bars	35	40	75
Application of 3-component cement-based epoxy resin	25	15	40
Application of pre-bagged acrylic modified mortar	750	300	1050
Application of chloride/sulfate barrier coating on repaired surface	20	20	40
TOTAL COST	830	625	1455
SYSTEM 3*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of composite cement epoxy on steel bars	35	40	75
Application of portland cement mortar (max. w/c ratio 0.38)	25	300	325
Application of chloride/sulfate barrier coating on repaired surface	20	20	40
TOTAL COST	80	610	690
SYSTEM 4*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of composite cement epoxy on steel bars	35	40	75
Application portland cement /micro-silica slurry as bond coat material	2	8	10
Application of portland cement/micro-silica mortar (max w/c+s ratio 0.38, min. micro-silica 5% of cement)	30	300	330
Application of chloride/sulfate barrier coating on repaired surface	20	20	40
TOTAL COST	87	618	705

Table 6.2 (contd.) Cost breakdown for repair systems for service environments in Saudi Aramco.

Work Details	Cost, SR		
	Material	Manpower	Total
SYSTEM 5*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of composite cement epoxy on steel bars	35	40	75
Application of portland cement /micro-silica slurry as bond coat	2	8	10
Application of repair mortar, portland cement/micro-silica mortar (max w/c+s ratio 0.38, min. micro-silica 5% of cement)	30	300	330
Application of chemically resistant epoxy coating on repaired surface	35	20	55
TOTAL COST	102	618	720
SYSTEM 6*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of single-component/zinc rich epoxy on steel bars	20	40	60
Application of resin based repair mortar	4500	400	4900
Application of chemically resistant epoxy coating on repaired surface	35	20	55
TOTAL COST	4555	710	5265
SYSTEM 7*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of sprayed concrete (dry mix) portland cement (max. w/c=0.38)	25	210	235
Application of chemically resistant epoxy coating on repaired surface	35	20	55
TOTAL COST	60	480	540
SYSTEM 8*			
Concrete breaking, surface preparation, and cleaning of rebars	0	250	250
Application of sprayed concrete (dry mix) portland cement + micro-silica (max w/c=0.38; micro-silica 10%)	30	240	270
Application of chemically resistant epoxy coating on repaired surface	35	20	55
TOTAL COST	65	510	575
SYSTEM 9**			
Surface preparation including placing of nipples and application of putting	25	15	40
Injection of repair material, resin injection grout	30	30	60
Application of chloride/moisture resisting (polymer modified cement) coating on repaired surface	25	20	45
TOTAL COST	80	65	145
SYSTEM 10**			
Surface preparation including placing of nipples and application of putting	25	15	40
Injection of repair material, polyurethane injection grout	45	30	75
Application of chloride/moisture resisting (polymer modified cement) coating on repaired surface	25	20	45
TOTAL COST	95	65	160

*Cost per square meter of repaired area and thickness of 100 mm.

**Injection cost is for linear meter of crack length and one square meter of coating application.

6.9 APPENDIX 6.A (Summary of Repair Procedures)

Repair Systems

Repair systems that could be utilized to repair deteriorated concrete structures, are summarized in the table below and Repair system procedures are detailed in the following pages for easy reference. As a result of the study that was conducted, systems 2, 3 & 4 were eliminated.

Structure	Repair System	Remarks
Marine	1 or 7	Mostly in splash zone. System 7 is for large repairs. System 1 is for medium to small repairs
Below Ground	1 or 6	Reinforcement corrosion or sulfate attack. System 1 is for large areas. System 6 is for small areas.
Exposed to Sulfur Fumes	5 or 6 or 8	Mainly acid attack. System 6 is ideal. System 5 or 8 is ideal for large repairs or when resin-based mortars cannot be used.
Exposed to Acid	5 or 6 or 8	System 6 is ideal. System 5 or 8 is ideal for large repairs or when resin-based mortars cannot be used
Sweet & Saline Water Retaining	9 or 10	Cracking, due to differential exposure temperatures within and outside, may be the major cause.
Fire Damage	7 or 8	Cracking, delamination and spalling is the major form of concrete deterioration in structures exposed to fire.

System 1 Procedure

The following procedure should be adopted for repairing structures using repair system 1:

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal. Additional reinforcement should be provided if more 25% of the original steel is lost due to corrosion.
3. Apply a single-component zinc-rich epoxy-based steel primer to the exposed reinforcing steel. The rate of application should be that recommended by the manufacturer. The steel primer should be uniformly applied so that there are no pinholes and uncovered areas.
4. After the steel primer has dried, wet the exposed surface by sprinkling potable water so that the concrete is in a saturated surface dry condition. Excess water should be cleaned by an air blast.
5. Apply free flowing micro-concrete to the exposed surface. The thickness of each layer should not be more than that recommended by the supplier. Suitable formwork may be installed to contain the repair material. Cure the repair material by covering the repaired surface with hessian for at least seven days. Alternatively, a curing compound may be applied on the repaired surface.
6. Apply a chloride/sulfate barrier coating after the repair material has dried. The surface preparation, number of coats and the coverage rate should be that recommended by the supplier.

System 5 Procedure

The procedure for repairing structures exposed to sulfur fumes, using system 5, is as follows:

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal. Additional reinforcement should be provided if more than 25% of the original steel is lost due to corrosion.
3. A composite cement epoxy-based steel primer should be applied to the exposed reinforcing steel. The rate of application should be that recommended by the manufacturer. The steel primer should be uniformly applied so that there are no pinholes or uncoated surface.
4. Apply cement/micro silica slurry to the exposed surface. The water to cement ratio of the slurry should not be more than 0.38 by weight. The micro silica should be 10% of the total cementitious material. The slurry should be uniformly spread so that the thickness is uniform and there are no pinholes or uncovered areas.
5. Place the portland micro-silica mortar in the exposed area before the cement slurry dries out. The micro silica content should not be more than 10% of the total cementitious material. The cement to sand proportion should be 1:2.5 by weight cement and the water to cement ration should not be more than 0.38 by weight of the cementitious material. Sand, cement and micro silica should be thoroughly mixed in a mechanical mortar mixer of appropriate capacity to obtain a uniform color of the mortar.
6. The mortar should be placed in the exposed area and consolidated by tamping with a steel rod to remove entrapped air. If necessary the repair material may be placed in layers. The repair surface should be leveled with a trowel.
7. Cure the repaired surface either by the application of a curing compound or wet-hessian. In case of wet curing, it should be continued for at least seven days.
8. Apply a chemically resistant surface coating after the repair material has dried. The surface preparation, number of coats, and the coverage rate of the coating should be that recommended by the supplier.

System 6 Procedure

The procedure for repairing below ground structures using system 6 is as follows:

1. Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregate. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal. Additional reinforcement should be provided if more 25% of the original steel is lost due to corrosion.
3. Apply a single-component zinc-rich epoxy-based steel primer to the exposed reinforcing steel. The rate of application should be that recommended by the manufacturer. The steel primer should be uniformly applied so that there are no pinholes and uncovered areas.
4. Apply a resin mortar to the exposed surface. The thickness of each layer should not be more than that recommended by the supplier. Suitable form work may be installed, if required, to place the repair material in the repair area.
5. Apply a chemically resistant surface coating after the repair material has dried. The surface preparation, number of coats, and the coverage rate of the coating should be that recommended by the supplier.

System 7 or 8 Procedures

The following procedure should be adopted when system 7 is utilized.

1. **Remove the delaminated, deteriorated or spalled concrete. A chipping hammer that does not cause damage to surrounding areas should be utilized for this purpose.
2. After removal of the deteriorated concrete, the surface of the old concrete should be thoroughly cleaned by abrasive blasting in order to remove all dirt, and expose the aggregates. Where steel is exposed, the full circumference of the bar should be cleaned to bare metal.
3. Water should be sprinkled on the exposed surface so that the concrete is in a saturated condition. The excess water should be removed by blowing air.
4. Site batched cement and aggregates or packaged premixed materials may be used. Sprayed concrete should be a dry mix with maximum water to cement ratio of 0.38. To minimize rebound, the ingredients should be thoroughly blended. **(For System 8, 10% silica fumes should be used as partial replacement of cement)**
5. Shotcreting should be done by experienced workmen. The thickness of shotcrete should be as constant as possible. Abrupt changes will decrease the bonding capability and increase chances of forming voids or a poor quality shotcrete.
6. The finished surface should be trowelled lightly to produce a plain surface and protected by covering with wet hesian sheets. Curing of the surface should continue for at least seven days.
7. After appropriate curing, for at least seven days, and drying, apply a chemically resistant surface coating on the repaired surface. Surface preparation and the number of coats and the coverage rate of the coating should be as per the manufacturer's recommendations.

** For structures damaged by Fire, the following two items should replace Item 1.

1. Conduct a survey to identify concrete damaged by fire. This can be done by using a schmidt hammer or pulse velocity measurements.
2. Remove the concrete weakened by fire with light pneumatic tools. Cutting and removal of concrete should provide a profile suitable for the repair. Provide additional reinforcement if necessary.

System 9 or 10 Procedures

The following procedure is recommended for repairing sweet water or saline water retaining concrete structures utilizing repair system 9 or 10.

1. Fix injection ports along the length of the crack. These injection ports can either be glued to the concrete surface or inserted into the concrete by drilling holes. When the injection ports are fixed by drilling holes, the holes must be drilled from either side of the crack. The holes must be away from the crack so that they do not split the concrete when the injection port is tightened.
2. Seal the crack with an epoxy putty. Allow the epoxy to set.
3. When using polyurethane injection material (system 10), inject the crack with water to saturate the concrete.
4. Pump the injection material through the injection ports. Manufacturer's instructions should be followed in the mixing of the resin and hardener and the pumping pressure. The injection material should be injected through each injection port till the material flows out of the next one. For cracks in vertical and inclined surfaces injection should start at the lowest point and proceed upwards.
5. Allow the injecting material to set and remove the injection ports. Close the holes of the injection ports with a suitable epoxy or mortar. Grind the putty and the sealant at the locations of inject ports.
6. Apply polymer-modified cement coating on the structure. Manufacturer's instructions as to the surface preparation and the number of coats and coverage rate should be followed strictly.

CHAPTER 7

LONG-TERM MONITORING STRATEGIES

7.1 INTRODUCTION

As discussed in the earlier sections of this manual, repair of concrete should address the causative factors in addition to being compatible with the parent concrete. There should be physical, chemical, and thermal compatibility between the parent concrete and the repair material. Physical compatibility ensures that the properties of the repair material are as close to those of the parent concrete. Similarly, the chemical compatibility of concrete and the repair material ensures that incipient anodes are not formed due to differences in the chemical composition of the repair material and the parent concrete. Chemical incompatibility between the repair material and the concrete leads to the deterioration of the latter, particularly due to reinforcement corrosion, as a result of crevice effects. Differential shrinkage characteristics of the repair material and the parent concrete also lead to cracking of the repair material, thus leading to the debonding of the repair material with the parent concrete. Variation in the coefficients of thermal expansion of the repair material and the parent concrete also initiates cracking of the repair material. Therefore, it is essential to carefully design a repair system that minimizes the incompatibility between the repair material and the parent concrete.

After a structure is repaired, it should be monitored continuously, more frequently at the earlier stages of repair and it can be spaced at later stages, to assess the performance of the repair systems. The performance of a new repair can be evaluated by: (i) visual inspection, (ii) debonding survey, (iii) monitoring the chloride and moisture variations, (iv) measuring the depth of carbonation, and (v) assessing reinforcement corrosion.

A monitoring program should be set up to assess the performance of the repair system, firstly, to ascertain that quality work has been carried out and, secondly, to ascertain that the repair and the parent concrete are acting as a composite section. Monitoring of the repair is also essential to plan for future maintenance programs.

Some of the techniques that can be utilized to monitor the performance of a repair are elucidated in the following sections. There is no established methodology to assess the performance of the repair system, however, experience with the methodology utilized to assess deterioration in parent concrete could be extrapolated for the repair part.

7.2 VISUAL INSPECTION

Visual inspection of a repaired structure provides useful information on the physical compatibility between the repair material and the parent concrete. Excessive shrinkage of the repair material may lead to its cracking. A visual survey should therefore be aimed at identifying cracks that have appeared after the structure has been repaired. Shrinkage cracks, due to incompatibility between the repair material and the parent concrete, appear after curing has been terminated. The crack width should be monitored over a period of time to assess whether they are live cracks or dormant cracks. Signs of rust stains, efflorescence, laitance, etc., provide an indication of other deterioration phenomena, such as reinforcement corrosion, alkali-aggregate reaction, sulfate attack, etc.

7.3 DEBONDING

Debonding between the parent concrete and the repair material may be caused due to improper application of the repair material or to other deterioration phenomenon, such as reinforcement corrosion. Debonding due to the former cause can be detected at the early stages of repair, while that due to the latter causes is noticeable only after a long period of time.

The simplest method of detecting the debonding of the repair material with the parent concrete is a hammer survey. A hammer survey indicates the location of the debonded areas. When a repair area is struck with a lightweight hammer, a hollow sound emanates from the debonded area. The technique of chain drag is also utilized to assess the debonding of the repair material with the parent concrete.

While the hammer survey provides information on the location of the debonded areas, the depth of the debonded area, i.e., whether the debonding is between the parent concrete and the repair material or within the repair material itself, cannot be assessed by the hammer survey. A suitable technique to assess the location of debonding between the concrete and the repair material is the measurement of the pulse velocity.

Pulse velocity through concrete can be measured by the direct or indirect methods. In order to measure the depth of debonding, the transmitting transducer is stationed at a certain location on the structure while the receiving transducer is moved in equal incremental distances. The transmission time is plotted against the total path length. In a sound medium, the transmitting time increases almost linearly with increasing path length. However, when there is a wide crack or debonding, the transmission time-path length flattens out. The distance at which the transmission time-length curve flattens out indicates the depth at which debonding should be expected. This technique is widely utilized to detect concrete delamination due to reinforcement corrosion or the depth of concrete damage due to fire.

7.4 MONITORING THE CHLORIDE AND MOISTURE CONTENT

In the structures exposed to marine or below ground conditions, the adequacy of repairs can also be evaluated by measuring the chloride and moisture concentration profiles at varying locations. Chloride content measurements basically involve collecting powder samples and analyzing them for either water-soluble or acid-soluble chloride concentration.

A comparison of the chloride concentration in the repair material and/or the parent concrete with that of the base value, i.e., determined prior to the repair, provides an indication of the efficacy of the repair in preventing the diffusion of chloride ions into the repaired areas. Similarly, the sulfate concentration in the repair material and in the concrete prior to repair also provides an indication of the efficacy of the repair.

Moisture is required for the corrosion process. A non-uniform distribution of moisture provides the potential for the initiation of the reinforcement corrosion process. Therefore, the measurement of moisture distribution is now considered to provide an indication of the possibility of reinforcement corrosion. The variation of moisture content in the repair material

also contributes to differential shrinkage of the repair material leading to differential cracking due to the restraint provided by the concrete substrate.

The moisture content in the concrete powder can be determined by drying the powder sample either in a microwave oven or in a laboratory oven at 110 °C for 24 hours. Portable equipment to measure the electrical resistivity in-situ has been developed based on the Wenner four-probe electrical measurement technique. However, a good calibration curve needs to be developed prior to interpolating the results. The electrical resistivity measurements may be affected by the presence of both cations and anions, such as chlorides, sulfates, bromides, etc. Therefore, electrical resistivity measurements should be interpreted with caution.

The electrical resistivity measurements are also utilized to assess the possibility of corrosion of reinforcing steel, as will be discussed in the later part of this chapter.

7.5 MEASUREMENT OF CARBONATION DEPTH

The depth of carbonation, in the cement-based repair materials, provides an indication of the performance of the repair system, particularly when it is designed as an anti-carbonation measure. The usual practice of measuring the carbonation depth is to take a small diameter core and spray phenolphthalein solution on its surface. However, in repaired structures, it is advisable to measure carbonation on powder samples. Concrete powder samples are taken at various depths and phenolphthalein solution is sprayed on the powder. Powder from the carbonated zone remains colorless while that from the uncarbonated area becomes purple. Another method that can be utilized is to drill a hole of about 12 mm diameter in the structure up to the reinforcing steel. The drilled hole should be cleaned of loose dust by blowing dry air with a bulb. Phenolphthalein should then be applied on the walls of the drilled hole with a cotton swab. Colorless areas in the hole could be examined with a pencil torch and the depth of carbonation measured with the metallic end of a vernier caliper.

7.6 ASSESSING REINFORCEMENT CORROSION

The development of incipient anodes is a probability when repairing reinforced concrete structures due to reinforcement corrosion. A periodic monitoring of the repaired reinforced concrete often provides an indication of the formation of incipient anodes. Reinforcement corrosion can be assessed by the following techniques:

- Resistivity measurements,
- Measurement of corrosion potentials,
- Measurement of corrosion rate utilizing polarization resistance measurements, and
- Corrosion probes.

A discussion of the above methods is provided in the following subsections.

7.6.1 Resistivity Measurements

The resistivity of near-surface concrete can be measured non-destructively by placing electrodes on the concrete surface, applying a voltage, and measuring the resulting current. Several arrangements can be used: one electrode (the reinforcing steel is the second electrode), two electrodes, and four electrodes.

Most measurements use alternating current (AC) with a frequency between 50 and 1000 Hz, usually sinusoidal. DC is not recommended because it may involve errors due to electrode polarization. Principally, a resistance value is measured, which depends on the geometry of the electrodes, which has to be converted to resistivity, the geometry-independent material property.

If the concrete composition is relatively homogeneous, mapping the resistivity may show wet and dry areas. If the resistivity values are between 100 and 500 $\Omega\cdot\text{m}$, the extreme values can be interpreted as indicating relatively wet and relatively dry areas. If, on the other hand, the exposure (so the moisture content) is relatively uniform, variations in resistivity (say from 50 to 200 $\Omega\cdot\text{m}$) can be interpreted as caused by local variations in the water-to-cement ratio. Areas with 50 $\Omega\cdot\text{m}$ will be more susceptible to penetration of chloride from the environment than areas with 200 $\Omega\cdot\text{m}$.

The interpretation of resistivity values with regard to risk of corrosion is shown in Table 7.1.

Table 7.1. Concrete resistivity and risk of reinforcement corrosion at 20 °C.

Concrete resistivity, $\Omega\cdot\text{m}$	Risk of corrosion
<100	High
100-500	Moderate
500-1000	Low
>1000	negligible

7.6.2 Measurement of Corrosion Potentials

Mapping of corrosion potentials has been shown to be a powerful, rapid and non-destructive technique both in condition assessment and during repair. The half-cell potential measurement is obtained by voltage measurements between a reference electrode and the working electrode, i.e. the reinforcing steel. The reference electrode is placed on the surface of the concrete and a voltmeter with high impedance is used to measure the potentials between reinforcement and the reference electrode. However, prior to the measurement, local removal of some concrete is necessary to enable a direct electrical connection to the reinforcing steel by means of a clamp. Then the rebars have to be connected to the positive terminal of the voltmeter. The negative (ground) terminal of the voltmeter is connected to the reference electrode. In most cases, copper-copper sulfate electrodes (CSE) or saturated calomel electrodes (SCE) are used.

To obtain reliable results, electrical continuity of reinforcement within the areas to be investigated must be ensured before the measurements. Continuity is checked by measuring the resistance between locally separated areas. Resistance values of 0.3 Ω or lower indicate

electrical continuity. Placing an external reference electrode on the concrete surface and taking the potential readings on a regular grid on the free concrete surface carry out potential measurements. Potential values can be interpreted in a way that more negative potentials suggest a higher probability for the occurrence of corrosion.

7.6.3 Measurement of Corrosion Rate

The corrosion rate measurements are based on the measurement of the linear polarization resistance method (LPRM). LPRM is a traditional dc technique of measuring corrosion rates of steel in aqueous systems. In this technique, the polarization resistance (R_p) is determined by conducting a linear polarization scan in the range of ± 10 mV of the corrosion potential. A potentiostat/galvanostat is used for this purpose. The corrosion current density is then calculated using the following relationship:

$$I_{\text{corr}} = B/R_p$$

where: I_{corr} = corrosion current density, $\mu\text{A}/\text{cm}^2$
 R_p = polarization resistance, $\Omega \cdot \text{cm}^2$
 $B = (\beta_a \cdot \beta_c) / 2.3(\beta_a + \beta_c)$

Typical values of corrosion current density (I_{corr}) and the resulting rate of corrosion are given in Table 7.2.

Table 7.2. Typical corrosion rates for steel in concrete.

Corrosion Current Density ($\mu\text{A}/\text{cm}^2$)	Rate of Corrosion)
10 - 100	High
1 - 10	Medium
0.1 - 1	Low
< 0.1	Passive

7.6.4 Monitoring Corrosion utilizing Corrosion Probes

Probes have been utilized to monitor reinforcement corrosion, particularly in the repaired portions. Two types of probes, namely resistance probes and corrosion probes have been developed.

The major advantage of the electrical resistance method is that time-dependent changes in the rate of corrosion can be determined. From this point of view, the method has significant advantages over the potential measurement technique, but it also has some fairly significant disadvantages. For example, the electrical resistance method can only give an assessment of the corrosivity of the environment and the rate of corrosion to be expected at a particular location where the probe is situated. Because of the wide expanse of most reinforced concrete

structures and the different conditions throughout the structure, extrapolation of this value to the rest of the steel in the structure can be difficult.

The electrical resistance probe, however, can be used successfully to monitor the effectiveness of corrosion prevention methods. The probe can be placed into the structure immediately before the application of a repair material. If the environment around the probe becomes adequately corrosive, then the effectiveness of these ameliorative measures can be judged by monitoring the change in resistance of the probe.

A recent development in the corrosion assessment is the corrosion probes. The standard sensor consisting of six single anodes. Each of the six black steel anodes is positioned 50 mm from the next one to prevent interactions between these measuring electrodes.

The cables are lead through stainless steel fixtures to the measuring device. The layout of the sensor system allows, besides the readings of the electrical currents, also other measurements improving the information on the overall corrosion risk within the monitored structure.

The measurement of the potential between the anodes and the noble cathode gives further information on the corrosion behavior of the reinforcement, especially the availability of oxygen. The simultaneous measurement of the temperature by means of the incorporated temperature sensor allows a more detailed interpretation of the readings.

The anodes are additionally used as measuring electrodes for AC resistance measurements at the different distances from the concrete cover. These readings are especially important, e.g. to monitor the efficiency of coatings in preventing water ingress into the concrete.