BUILDING REPAIRS AND MAINTENANCE

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BUILDING REPAIRS AND MAINTENANCE

SYLLABUS

Repairs and Maintenance of Buildings

Repairs and Maintenance of buildings, principles of construction and detection of defects, and advice on the course of action to be taken: planning.

General knowledge of the life serviceability and strength of building materials in common use for the purpose of maintenance.

The preparation of schedules of dilapidation and bill of quantity with specifications for repairs.

Methods of measurement, recording and pricing of work.

Building survey for assessment of damage due to fire, explosion, earthquake or any other peril for insurance purpose and preparation of estimate for insurance claim.

Maintenance of plants providing services and refurbishment cost in use and prediction of performance in building.

Dilapidations

Liability from dilapidations. Nature of waste.

Implied and statutory obligation to repair as between landlord and tenant under different tenancy / lease agreements. Fair wear and tear.

Recent amendments in the rent control act, Housing and Area Development Authority Act for different States, its effects on property market for real estate.

Site visits and preparing report on repairs and maintenance of buildings.

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INTRODUCTION

Objective

- ❖ To understand the terms used in repair and rehabilitation of structures.

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1.2. Quality of concrete

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1.6. Repairs and Rehabilitation

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1.8. Maintenance of structures

1.9. Summary

1.10. Keywords

1.11. Intext Questions

1.1. General

Structures are designed to withstand safely a particular predetermined load during their life period. Generally reinforced concrete (RC) structures can suffer varying degrees of damage due to several reasons including material deterioration, construction technique adopted, poor workmanship, overloading, aggressive environments, fatigue and corrosion of steel reinforcement embedded in concrete. The RCC buildings have been extensively constructed throughout the world since 1950. The deterioration of RCC is a natural phenomena and has started exhibiting in large number of concrete structures. Hence a systematic approach is needed in dealing with such problems.

1.2. Quality of Concrete

The performance of concrete structures depends not only on the improper use but also the quality of concrete. In most of the cases the root cause of concrete disintegration is the inherent porosity, which in turn leads to absorption, diffusion and permeability of water. The quality of concrete depends on the following factors.

1. water cement ratio
2. sand/stone ratio
3. cover depth
4. chloride content in constituents
5. moisture content
6. oxygen
7. pH value
8. temperature

9. permeability of concrete
10. method and time of curing
11. electrical resistivity of concrete
12. crack width
13. type and size of reinforcement bars.

Concrete provides excellent protection to reinforcing steel. But large numbers of structures have been reported in which corrosion of reinforcement has caused damage to concrete structures within a few years. Repairing of corrosion damaged reinforced concrete elements is very difficult in practice. Reinforcement corrosion caused by carbonation is arrested to a great extent through repairs executed in a sound manner. But, the treatment of chloride induced corrosion is very difficult and more often the problem continues even after extensive repairs have been carried out. It invariably re-occurs in a short period of time. When chlorides are present in concrete, it is very difficult to protect reinforcing steel from chloride attack.

1.3. Damages in RC Structures:

- Cracking
- Leakage
- Settlement
- Over Deflection
- Wearing
- Spalling
- Disintegration
- Delamination
- Over loading
- Aggressive Environments
- Materials used for construction
- Fatigue and Corrosion

1.4. Deterioration of Structures

The major factors responsible for concrete deterioration are permeability, carbonation, chemical attack, alkali aggregate reaction and physical aggression like thermal shock and abrasion. The general deterioration of concrete structures is usually accompanied by cracking and spalling. Reinforcement corrosion plays the most vital role in the deterioration of the concrete structure. Inspite of advanced developments in building construction, all the buildings deteriorate from the time they are completed. The rate of deterioration depends upon a number of factors. Not all the factors are under the control of the occupants. During the design and construction stages, the following factors are considered as essential parameters for durability of structures:

1. Right choice of material
2. Proper construction methods

3. Adequate specifications for construction and installation work.
4. Effective supervision throughout the construction period and rectification of defects prior to final handover of the buildings.
5. Provision of adequate space for landscaping with proper design.

1.5. Damage Assessment of Structures

To identify the suitable repair procedure, it is necessary to have a planned approach to investigate the condition of concrete and reinforcement. This will require a thorough technical inspection and an understanding of the behavior of the structural component, which is being repaired. By the visual inspection, a detailed mapping of affected areas, documentation of type and location of symptoms, their history and photographic evidences are prepared. A comprehensive inspection data helps in making an effective strategy for repair and rehabilitation.

Early detection of structural damage is an important issue to minimize the cost of repairs.

Non-Destructive Tests (NDT) can be effectively employed to evaluate the damages in structures and to choose a suitable method of repair technique to extend their service life. The strength and life depends on deterioration of structures and in turn change the structural parameters like stiffness of a member. The extent of damage can be effectively assessed using the stiffness degradation in the member. The level of damage in RC structures should be effectively assessed in order to ensure safety and serviceability conditions.

1.6. Repair and Rehabilitation

Repair and Rehabilitation mean restoring the damaged structures to make them fit for serviceability condition. Rehabilitation of structurally deteriorated RC structures is one of the major tasks for the construction industries worldwide. Use of properly selected repair materials can solve this tough task. Durable repair can be obtained only by matching the properties of the base concrete with those of the repair material intented for use (Neelamegam, 2001). The selection of repair materials is based on their properties and some of them are listed below:

i. Dimensional stability
ii. Modulus of elasticity
iii. Permeability
iv. Chemical resistance
v. Adhesion with parent concrete
vi. Coefficient of thermal expansion
vii. Easy to use

Selecting a most appropriate material and repair methodology is very important to achieve durable, effective and economic repairs. Matching the response of repaired sections with the main structure is an important task. Compatibility of materials and matching specifications are essential in any repair job. Just as building durable construction requires understanding of structural engineering, material science, and environment/exposure conditions, repair jobs also require the same level of attention in these areas. The engineer incharge should be familiar with repair methodology and repair materials. The engineer undertaking such specialized jobs should have thorough knowledge of new materials, repair methodologies, its limitation and the fundamentals of structural engineering to ensure safety and serviceability of the building during repair and thereafter.

1.7. Repair Management

A repair job involves three distinct stages at different levels. In the first stage, documentation of damage, type of damage and its extent, forecasting the repaired structure and recommendations on repair methodology. The second requires preparation of detailed drawings, sketches, execution guidelines and notes, material and works specifications and tender document. The third stage is actual execution of repairs. For specialized jobs, experts in this repair field and resources in terms of tools and plants should be engaged. The engineer incharge should have a good understanding of the procedures and give an attentive supervision. To find out the effectiveness of repairs, various tests before and after the repairs have been employed.

1.8. Maintenance of Structures

Building maintenance is work undertaken to keep, restore or improve every facility at every part of the building, its services including horticulture operations to a currently acceptable standard and to sustain the utility and value of the facility. The objectives of maintenance include the following:

1. To preserve machinery, building and services, in good operating condition.
2. To restore it back to its original standards,
3. To improve the facilities depending upon the development that is taking place in the building engineering.

Maintenance aims at effective and economic means of keeping the building and services fully utilizable. It involves a lot of skills as influenced by occupancy and the performance level expected of a building. Programming of works to be carried out to keep the building in a good condition calls for high skills. Feedback from the maintenance should be a continuous process to improve upon the design and construction stages. Preventive maintenance is also carried out to avoid breakdown of machinery and occurrence problems in buildings and services. It should be carried out on the basis of regular inspection of buildings.

1.9. Summary

The terms used in repair and rehabilitation of structures such as damage, quality of concrete, assessment, repair techniques and maintenance are discussed in this chapter.

1.10. Keywords

Repair – Rehabilitation – Damage assessment – maintenance.

1.11. Intext Questions

1. What are the factors affecting the quality of concrete?
2. List the damages occurred in RC structures.
3. What are the factors to be considered during design and construction stages for durable structures?
4. Why is damage assessment necessary?
5. How will you select a repair material?
6. What are the objectives of maintenance of structures?

❑

CRACKS

Objective

❖ To study about the causes, types and control of cracks in concrete strucutures.

Contents

2.1. General
2.2. Wall cracks
2.3. Causes of concrete cracking
2.4. Types of cracks
    2.4.1. Plastic shrinkage cracks
    2.4.2. Plastic settlement cracks
    2.4.3. Drying Settlement cracks
    2.4.4. Thermal cracks
    2.4.5. Map cracks due to alkali aggregate reaction
    2.4.6. Longitudinal cracks due to corrosion
    2.4.7. Transverse cracks due to loading
    2.4.8. Shear cracks due to loading
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2.6. Macro cracks
2.7. Aggressive Deteriorating Chemical Agents
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    2.7.2. Carbonation
    2.7.3. Chlorides
    2.7.4. Sulphate attack
    2.7.5. Alkali Silica Reaction
2.8. Causes and stages of disintegration
2.9. Summary
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2.1. General

A common adage is that there are two guarantees with concrete. One, it will get hard and two, it will crack. Cracking is a frequent cause of complaints in the concrete industry. Cracking can be the result of one or a combination of factors such as drying shrinkage, thermal contraction, restraint (external or internal) to shortening, subgrade settlement, and applied loads. Cracking can not be prevented but it can be significantly reduced or controlled when the causes are taken into account and preventative steps are taken. Another problem associated with cracking is public perception. Cracks can be unsightly but many consumers feel that if a crack develops in their wall or floor that the product has failed. In the case of a wall, if a crack is not structural, is not too wide (the acceptable crack ranges from 1/16” to 1/4”) and is not leaking water, it should be considered acceptable.

2.2. Wall Cracks

Diagonal cracks that extend nearly the full height of the wall are often an indication of settlement. Diagonal cracks emanating from the corner of windows and other openings are called reentrant cracks and are usually the result of stress build-up at the corner. Diagonal reinforcement at the corner of openings can reduce the instance of crack formation and will keep the cracks narrow.

Other procedures which can reduce cracking in concrete include the following practices.

1. Use proper subgrade preparation, including uniform support and proper subbase material at adequate moisture content.
2. Minimize the mix water content by maximizing the size and amount of coarse aggregate and use low-shrinkage aggregate.
3. Use the lowest amount of mix water required for workability; do not permit overly wet consistencies.
4. Avoid calcium chloride admixtures.
5. Prevent rapid loss of surface moisture while the concrete is still plastic through use of spray-applied finishing aids or plastic sheets to avoid plastic-shrinkage cracks.
6. Provide contraction joints at reasonable intervals, 30 times the slab thickness.
7. Provide isolation joints to prevent restraint from adjoining elements of a structure.
8. Prevent extreme changes in temperature.
9. To minimize cracking on top of vapor barriers, use a 100-mm thick (4-in.) layer of slightly damp, compactable, drainable fill choked off with fine-grade material. If concrete must be placed directly on polyethylene sheet or other vapor barriers, use a mix with a low water content.
10. Properly place, consolidate, finish, and cure the concrete.
11. Avoid using excessive amounts of cementitious materials.
12. Consider using a shrinkage-reducing admixture to reduce drying shrinkage, which may reduce shrinkage cracking.
13. Consider using synthetic fibers to help control plastic shrinkage cracks.

Cracks can also be caused by freezing and thawing of saturated concrete, alkali- aggregate reactivity, sulphate attack, or corrosion of reinforcing steel. However, cracks from these sources may not appear for years. Proper mix design and selection of suitable concrete materials can significantly reduce or eliminate the formation of cracks and deterioration related to freezing and thawing, alkali-aggregate reactivity, sulphate attack, or steel corrosion.

2.3. CAUSES OF CONCRETE CRACKING

I. Physical damage
II. Structural damage
III. Chemical and electrochemical damage
IV. Construction damage

I. Physical damage

a. Plastic shrinkage
b. Plastic Settlement
c. Drying Shrinkage
d. Thermal effects
e. Freeze and thaw
f. Abrasion
g. Erosion and cavitation
h. Fire

II. Structural damage

a. Design errors
b. Overloading
c. Settlement
d. Creep
e. Deflection
f. Fatigue

III. Chemical and electrochemical damage

a. Corrosion of reinforcement
b. Alkali-aggregate reaction
c. Sulphate attack
d. Acid attack
e. Carbonation

IV. Construction damage

a. Movements of the ground and formwork
b. Construction movement
c. Vibration

2.4. TYPES OF CRACKS

The following eight types of cracks are generally observed in buildings.

1. Plastic Shrinkage Cracks
2. Plastic Settlement Cracks
3. Drying Shrinkage Cracks
4. Thermal Cracks
5. Map Cracks due to alkali aggregate reaction
6. Longitudinal Cracks due to Corrosion

7. Transverse Cracks due to loading
8. Shear Cracks due to loading

2.4.1. Plastic Shrinkage Cracks

These cracks form during construction on a concrete surface, such as roof or road slab, if rapid evaporation of moisture from the concrete takes place (Figure 2.1). Plastic-shrinkage cracks are of varying lengths spaced from a few centimeters (inches) up to 3 m (10 ft) apart and often penetrate to mid-depth of a slab.

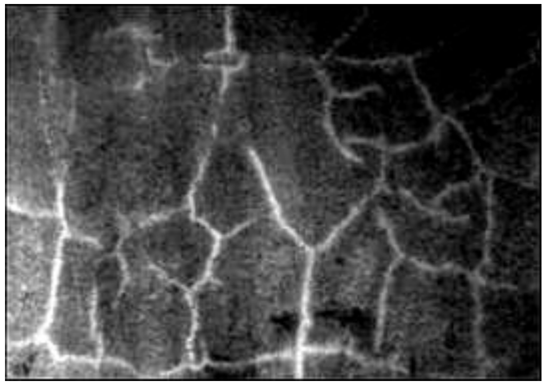

Figure 2.1. Plastic Shrinkage Cracks

These cracks are harmless unless the concrete surface will be exposed to salt or contaminated dust. Shrinkage of this type manifests itself soon after the concrete is placed in the forms while the concrete is still in the plastic state. Loss of water by evaporation from the surface of concrete or by the absorption by aggregate or subgrade is believed to be the **main reasons** of plastic shrinkage. The loss of water results in the reduction of volume. The **factors affecting** the plastic shrinkage are (i) Ambient temperature (ii) Relative humidity (iii) Wind velocity (iv)Temperature of concrete.

Volume change is one of the most detrimental properties of concrete, which affects the long-term strength and durability. To the practical engineer, the aspect of volume change in concrete is important from the point of view that it causes unsightly cracks in concrete. High water/cement ratio, badly proportioned concrete, rapid drying, greater bleeding, unintended vibration etc., are some of the reasons for plastic shrinkage. It can also be further added that richer concrete undergoes greater plastic shrinkage.

The preventive measures for plastic shrinkage are listed below:

1. Dampening of subgrade and forms
2. Controlling the wind velocity by erection of windbreaks
3. Minimizing placing and finishing time
4. Using membrane curing, begin curing as soon as possible after finishing
5. Using monomolecular films (evaporation retarders) or fog spray immediately after the screeding to maintain the water/cement ratio at the surface
6. Using surface dry aggregates

2.4.2. Plastic Settlement Cracks

These cracks form during construction in concrete due to settlement of concrete and bleeding of excess water from the concrete. Longitudinal cracks over the reinforcement will form and are a common cause of serious rusting of reinforcement. After concrete is placed, concrete bleeds, ie the solids settle down and the mix water rises up to the surface. If there is no restraint this merely produces a slight lowering of the concrete surface. But if the concrete is locally restrained from settling (eg: a reinforcing bar, duct or insert) while the adjacent concrete continues to settle, there is the potential for a crack to form over the restraining element (Figure 2.2). Water which collects under the reinforcement displaces the cement grout and leaves the reinforcement unprotected.

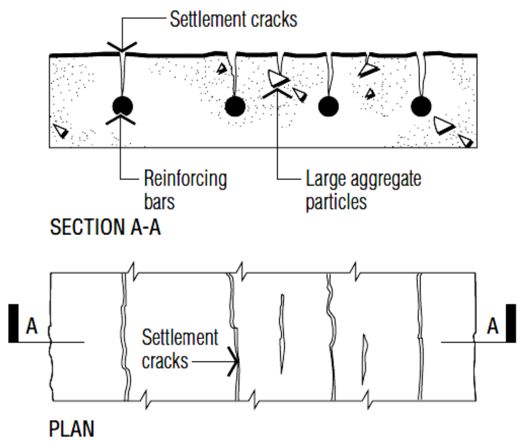

Figure 2.2. Plastic Settlement Cracks

Plastic settlement cracks are distinguished from plastic shrinkage cracks by their distinct pattern which typically mirrors the pattern of the restraining elements such as the reinforcement. The amount of settlement tends to be proportional to the depth of concrete, ie the deeper the section the greater the settlement. At changes of section such as the section at a beam/slab junction, the different amount of settlement can lead to cracks forming at the surface as shown in Figure 2.3.

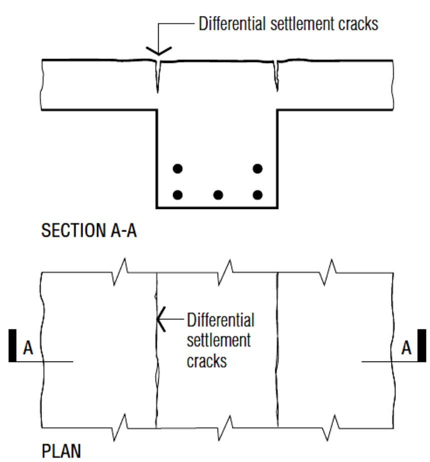

Figure 2.3. Differential Settlement Cracks

The risk of plastic settlement cracks can be minimized by the following practices:

1. Use mixes with lower bleeding characteristics, eg: Lower slump and more cohesive mixes.
2. Increase the ratio of cover to reinforcing bar diameter, ie, by increasing the cover or decreasing the size of the bars.
3. Wet the subgrade before placing concrete to avoid excessive water loss from the base of the concrete.
4. Set all the formwork accurately and rigidly so that it will not move during concrete placement.
5. Place concrete in deep sections first (including columns) and let it settle prior to placing and compacting the layers(ensuring that the two layers blend together)
6. Fully compact the concrete
7. Cure the concrete promptly and properly

The repair of plastic settlement cracks can be carried out in pre-hardened concrete as follows:

The most effective repair is to close the cracks shortly after formation by re-vibration and reworking the surface while the concrete is still plastic. Careful timing is essential to ensure the concrete re-liquefies under the action of the vibrator so that the cracks are fully closed. Re-vibrate too soon and cracks may reform; too late and the bond to the reinforcement may be damaged. Mechanical re-trowelling of the surface may be sufficient to close the cracks and compact the concrete around the reinforcement provided the cover is not too great, but the best result is where this is combined with some form of vibration. Caution needs to be exercised in the use of re-trowelling alone since it may just form a skin (which can fracture with subsequent shrinkage, thermal or traffic impacts) over the cracks but not close them. If used it must be done as soon as the cracks become evident.

In the hardened concrete, plastic settlement cracks may be chased out and filled using a suitable quality material. It is almost impossible to make a proper bond. However this repair simply ensures the durability and the wear characteristics of the surface.

2.4.3. Drying Shrinkage Cracks

The excess water evaporation after hardening of concrete results the drying shrinkage cracks (Figure 2.4). Just as the hydration of cement is an everlasting process, the drying shrinkage is also an everlasting process when concrete is subjected to drying conditions. The drying shrinkage of concrete is analogous to the mechanism of drying of timber specimen. The loss of free water contained in hardened concrete, does not result in any appreciable dimension change. It is the loss of water held in gel pores that causes the change in the volume. Under drying conditions, the gel water is lost progressively over a long time, as long as the concrete is kept in drying conditions. Cement paste shrinks more than mortar and mortar shrinks more than concrete. Concrete made with smaller size aggregate shrinks more than concrete made with bigger size aggregate. The magnitude of drying shrinkage is also a function of the fineness of gel. The finer the gel the more is the shrinkage.

Floor

Above the doors

Figure 2.4. Drying Shrinkage Cracks

While drying, hardened concrete will shrink about 1/16 in. in 10 ft of length. One method to accommodate this shrinkage and control the location of cracks is to place construction joints at regular intervals. For example, joints can be constructed to force cracks to occur in places where they are inconspicuous or predictable. Horizontal reinforcement steel can be installed to reduce the number of cracks or prevent those that do occur from opening too wide. The combination of drying shrinkage and restraint develops tensile stresses within the concrete. Due to inherent low tensile strength of concrete, cracking will often occur (Figure 2.5).

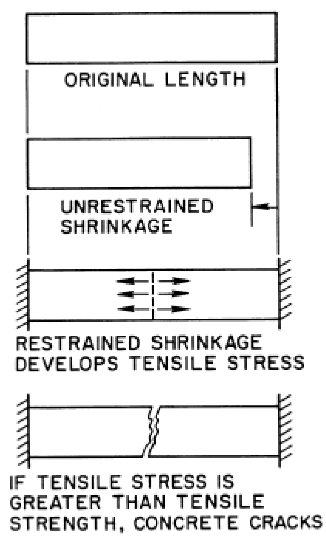

Figure 2.5. Cracking of concrete due to drying shrinkage

The major factor influencing the drying shrinkage properties of concrete is the total water content of the concrete. As the water content increases, the amount of shrinkage increases proportionally. Large increases in the sand content and significant reductions in the size of the coarse aggregate increase shrinkage because total water is increased and because smaller size coarse aggregates provide less internal restraint to shrinkage. Use of high-shrinkage aggregates and calcium chloride admixtures also increases shrinkage.

The factors affecting the drying shrinkage are shown in Figure 2.6 tabulated in Table.2.1. If a structural member is free to deform, there is no build-up of internal stress. But most of the structural members are restrained, stress build-up occurs and can be very significant. When stress build-up is relieved, it will occur in the weakest portion of the structural member or its connection to other parts of the structure. The developed stress causes tension cracks, shear cracks and buckling.

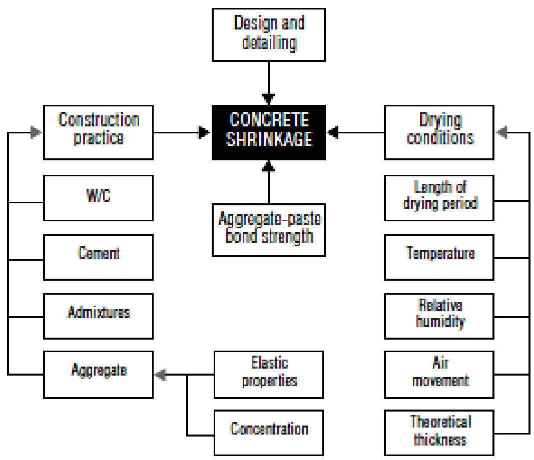

Figure 2.6. Factors affecting the drying shrinkage

Table 2.1. Factors affecting the drying shrinkage

Factor	Reduced Shrinkage	Increased Shrinkage
Cement type	Low grade	High grade
Cement content	325 kg/m^2	450 kg/m^2
Aggregate Size	40mm	20mm
Aggregate type	granite	Sand stone
Slump	50 - 75mm	125mm – 150mm
Curing	7 days	3 days
Placement temperature	15-20°C	30°C
Aggregate state	washed	dirty

2.4.4. Thermal Cracks

During hardening process of concrete, temperature rise due to cement hydration will take place (Figure 2.5). Concrete has a coefficient of thermal expansion and contraction of about 5.5×10^{-6} per °F. Concrete placed during hot midday temperatures will contract as it cools during the night. When cooling takes place, the concrete experienced tension cracks due to its very little tensile strength. A 40°F drop in temperature between day and night-not uncommon in some areas-would cause about 0.03 in. of contraction in a 10-ft length of concrete, sufficient to cause cracking if the concrete is restrained. Thermal expansion can also cause cracking.

Figure 2.5. Thermal Cracks

In thin members, such as pavements and bridge decks, thermal cracking is most likely to present problems when the concrete undergoes large temperature swings during the first several days after placement. Such effects are most pronounced in heavily reinforced structures such as continuously reinforced concrete pavementand decks; however, they can present serious problems for plain jointed pavements if midslab cracks result. The factors affecting the thermal cracks are as follows:

1. Initial temperature of materials
2. Ambient temperature
3. Large dimensions
4. Curing conditions
5. Early removal of formwork
6. More cement
7. Cement grade
8. Admixtures like flyash, etc.

Differential Thermal Exposure: The structural members having different thermal exposure conditions on the opposite faces, more particularly, those located on the exterior, are subjected to loading due to temperature gradient within the cross section. This is due to difference in temperature on the two faces of member during different times. As a result, tensile stresses in excess of tensile strength of concrete could develop across the cross section and result in formation of micro cracks. This process is cyclic due to daily and seasonal temperature variation conditions. Cyclic nature of loading is responsible for further increase as the crack depths propagate deeper with each cycle. The cyclic process of thermal loading of a structural member is shown in Figure 2.6.

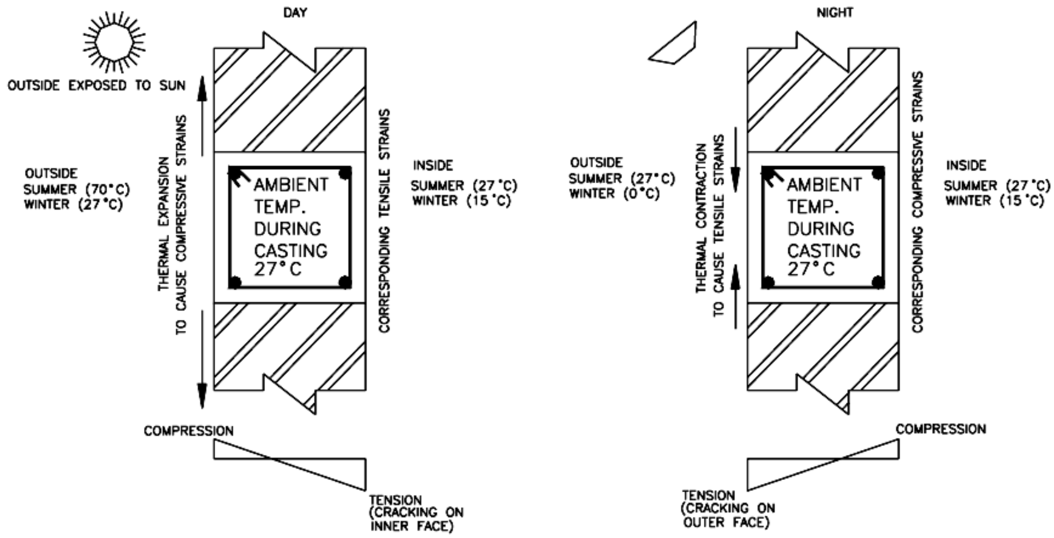

Figure 2.6. Cyclic process of thermal loading of a structural member

2.4.5. Map Cracks due to alkali aggregate reaction

The reaction of siliceous minerals (silica) in aggregate with alkalies (sodium oxide and potassium oxide) present in cement causes the swelling of concrete which results a pattern of cracking of concrete surface (Figure 2.7). The alkali aggregate reaction can be controlled by proper selection of non-reactive aggregates, use of low alkali cement, controlling moisture content and temperature.

Figure 2.7. Cracks due to alkali aggregate reaction

2.4.6. Longitudinal Cracks due to Corrosion

Corrosion of reinforcing steel and other embedded metals is one of the leading causes of deterioration of concrete (Figure 2.8). When steel corrodes, the resulting rust occupies a greater volume than steel. The expansion creates tensile stresses in the concrete, which can eventually cause cracking and spalling. Normally, the high pH of concrete protects the reinforcing steel from oxidation. The passivating effect of concrete on steel can be negated by the intrusion of chloride ions or by carbonation of the paste surrounding the reinforcement. The concrete between the reinforcement and the outer surface of the element thus serves as a barrier to the ingress of chloride ions or carbon dioxide.

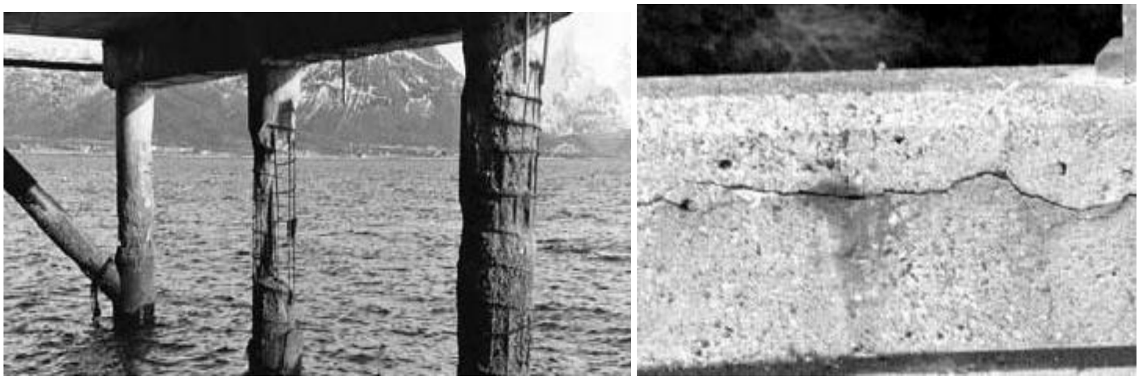

Figure 2.8. Cracks due to corrosion of reinforcement

These cracks form along the direction of the reinforcement in the members. In a beam, corrosion cracks form along the longitudinal direction and in a column, it forms in vertical direction. These cracks are a symptom of deterioration which will eventually lead to spalling and complete loss of cover of concrete.

2.4.7. Transverse Cracks due to loading

These cracks form in the concrete after it has hardened due to shrinkage, thermal contraction or structural loading. These cracks occurs perpendicular to the centerline of the pavement, or laydown direction, as shown in Figure 2.9. Transverse cracks are generally caused by thermally induced shrinkage at low temperatures. When the tensile stress due to shrinkage exceeds the tensile strength of the pavement surface, cracks occur. These cracks can be effectively treated with crack sealants. In RCC beams, cracks which are transverse to the main reinforcement occur along stirrups in vertical direction and cause corrosion of stirrups.

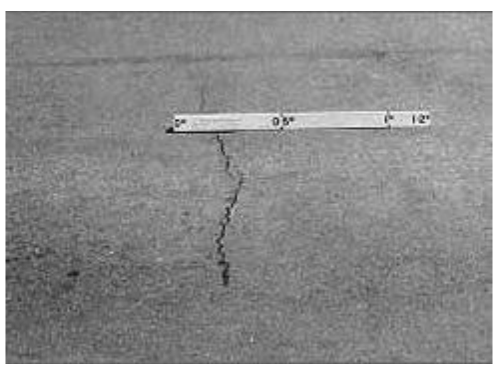

Figure 2.9. Transverse Cracks

2.4.8. Shear Cracks due to loading

Due to shear failure in beams and columns, shear cracks form at the support where the shear is maximum (Figure 2.10). It will form diagonally at the supports. These cracks can cause corrosion if they are left untreated.

Beam

Column base

Columns of bridge

Figure 2.10. Shear Cracks

2.5. Micro Cracks

During service life of a reinforced structure, it is subjected to various types of loading conditions (static and / or of cyclic nature) and also exposed to extreme exposure conditions of temperature variations (daily and seasonal). Micro-cracking combined with capillary porosity is generally responsible for ingress of aggressive chemicals in RCC. Figure 2.11 shows the crack propagation due to cyclic load in tensile zone of an RCC beam. The crack depths in structural members due to cyclic loading are higher than due to static loads of same intensity. Initial and after care of the structure in the form of periodical painting also plays an important role in controlling the adverse effect due to propagation of such micro-cracks.

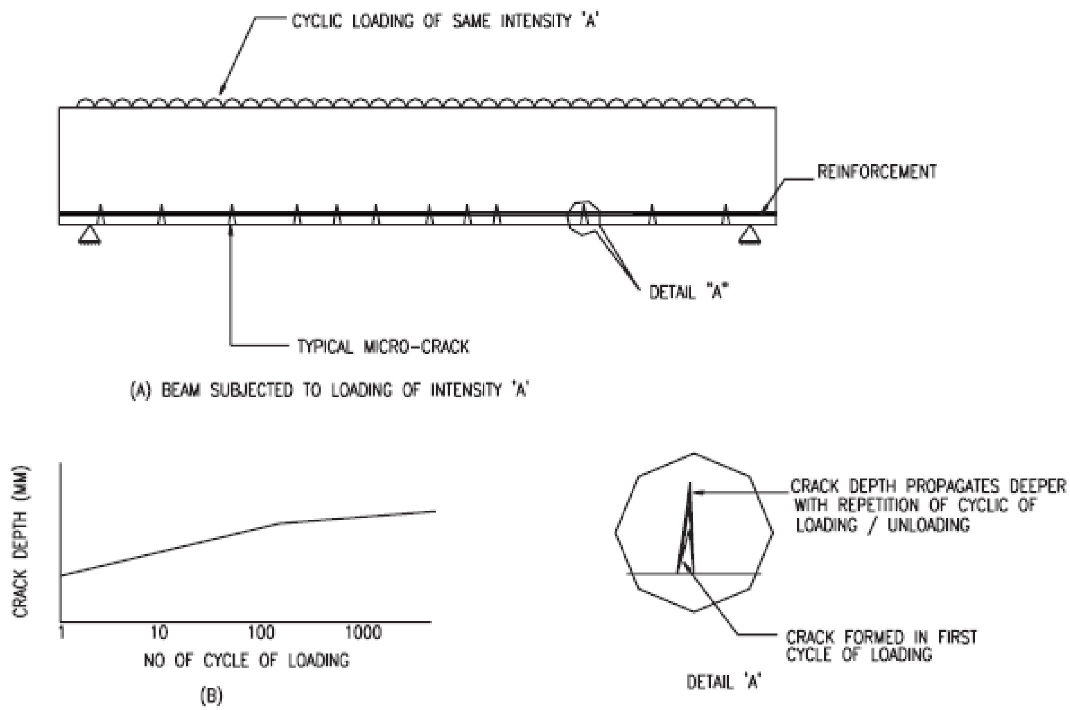

Figure 2.11. Cracks propagation in tensile zone of RCC beam

2.6. Macro Cracks

Some minor cracking in concrete structures would occur within the normal practice. Proper design and detailing coupled with proper construction practice can control the crack widths. Any crack width, which allows aggressive chemicals to travel freely into the concrete, is termed as Macro crack. The threshold limiting crack width has been defined by various codes of practices for RCC design. These vary from 0.1 mm to 3 mm. Any crack in concrete, which is wider than this, is likely to cause durability problems. The reasons for macro cracks are due to the following:

1. Improper placement of concrete
2. Settlement cracks of fresh concrete
3. Cracking due to
    - ❖ Intrinsic sulphate attack
    - ❖ Alkali aggregate reaction
    - ❖ Heat of hydration
    - ❖ Increased volume of corroded reinforcement exerting bursting pressure on concrete
4. Excessive loading

2.7. Aggressive Deteriorating Chemical Agents

The deterioration of RCC is related to loss of water tightness of cover concrete and migration of aggressive chemicals through interconnected porosity, which in turn chemically attacks on its constituents i.e. hydrated cement gel, aggregate and the reinforcement as following:

1. Corrosion of reinforcing bar – due to carbonation of concrete
   – due to ingress of chloride
2. Sulphate attack
3. Alkali Silica Reaction (ASR)

2.7.1. Corrosion of reinforcing bar

Steel reinforcement in concrete is protected from corrosion by a combination of the following:

a. The formation of a passivating protective layer on steel surface due to chemical reation under highly alkaline environment
b. The environment protection provided by the concrete cover

The hydration process of Portland cements release alkalies giving it a high pH in the range of 12 to 14. Even though oxygen and moisture may reach the steel surface, it will not corrode and will remain passive as long as high pH is maintained and the cover concrete is intact. The two common conditions that lead to the loss of passivity of steel in concrete are (a) reduction of alkalinity of concrete surrounding the steel with pH lower than 11 to 11.5 (b) presence of chemicals (chloride ions), which destroy the passivity even while the alkalinity of surrounding concrete remains high.

2.7.2. Carbonation

The Carbonation occurs in concrete because the calcium bearing phases present are attacked by carbon dioxide (CO_2) of the air and converted to calcium carbonate. Cement paste contains 25-50 wt% calcium hydroxide ($Ca(OH)_2$), which mean that the pH of the fresh cement paste is at least 12.5. The pH of a fully carbonated paste is about 7. As the concrete lost its alkalinity, the passive layer of the reinforcement bar breaks down and in the presence of water and air, corrosion starts. The carbonation process requires the presence of water because CO_2 dissolves in water forming H_2CO_3.

$$CO_2 + H_2O = H_2CO_3 \text{ (Acid)}$$

$$Ca(OH)_2 + H_2CO_3 = CaCO_3 + 2H_2O$$

(Alkali) (Acid) (Salt) (Water)

When $Ca(OH)_2$ is removed from the paste hydrated CSH will liberate CaO which will also carbonate. The rate of carbonation depends on porosity & moisture content of the concrete. If the concrete is too dry (RH <40%) CO_2 cannot dissolve and no carbonation occurs. If on the other hand it is too wet (RH >90%) CO_2 cannot enter the concrete and the concrete will not carbonate. Optimal conditions for carbonation occur at a RH of 50% (range 40-90%). Normal carbonation results in a decrease of the porosity making the carbonated paste stronger. Carbonation is therefore an advantage in non-reinforced concrete. However, it is a disadvantage in reinforced concrete, as pH of carbonated concrete drops to about 7; a value below the passivation threshold of steel. Carbonation may be recognized in the field by the presence of a discoloured zone in the surface of the concrete. The colour may vary from light gray and difficult to recognize to strong orange and easy to recognize.

Occasionally concrete may suffer from the so called bi-carbonation process. Bi-carbonation may occur in concrete with very high water to cement ratio due to formation of hydrogen carbonate ions at pH lower than 10. Contrary to normal carbonation, bi-carbonation results in an increase in porosity making the concrete soft and friable. Bi-carbonation may be recognized by the presence of large "pop-corn" like calcite crystals and the highly porous paste.

A common and simple method for establishing the extent of carbonation is to treat the freshly broken surface of concrete with a solution of phenophthalein in diluted alcohol. If the Ca(OH) is unaffected by CO_2 the colour turns out to be pink. If the concrete is carbonated it will remain uncloured. It should be noted that the pink colour indicates that enough $Ca(OH)_2$ is present but it may have been carbonated to a lesser extent. The colour pink will show even up to a pH value of about 9.5.

2.7.3. Chlorides

Chlorides may be present in the fresh mix or may penetrate from external source into the hardened concrete. During the use of the structures, chlorides may penetrate into the concrete from various sources. The most important one is sea water. Substantially greater amounts of chlorides may ingress into the hardened concrete via water transport mechanisms than via pure chloride ion diffusion. The following are the three forms of chloride can be existed in concrete mass:

a. Free Chlorides

They are the most dangerous forms of chlorides in the concrete as upon entering into concrete free chloride will diffuse through the pore water to attack the reinforcement bar by breaking down its passive oxide layer. The corrosion of reinforcement takes place due to chloride ions.

b. Physically Adsorbed Chlorides

Weakly bonded Chloride ions can be existed in the concrete due to the chemical composition of cement hydrate and type of surface area of hydrate. These forms of physically adsorbed Chlorides have the potential to move toward the reinforcement bar to start corrosion.

c. Chemically Adsorbed Chlorides

There are strongly chemically bonded chloride ions with the Calcium Aluminium Hydrate to form Friedell's salts. These forms of chlorides are safer as they can not proceed to the reinforcement to induce corrosion.

2.7.4. Sulphate attack

Sulphate attack is the chemical reaction between the sulphate ions from the ground-water and the different hydrate phase of cement hydrate, but mainly calcium aluminate hydrate to form calcium sulfo-aluminate hydrate, ettringite, or with calcium hydroxide to form gypsum. Initially these products may results in a void filling but eventually it expand and crack concrete. It leads to reduction of stiffness and strength of concrete.

Solid salts, such as sulphates, will not directly attack concrete however, when in solution, they can react with certain components of the cement paste leading to expansion, cracking and spalling of concrete. The most common forms of sulphate are:

1. Sodium sulphate Na_2SO_4
2. Potassium sulphate K_2SO_4
3. Magnesium sulphate $MgSO_4$
4. Calcium sulphate $CaSO_4$

The above sulphates are common in natural groundwater conditions and may exist singly or in combinations. Sulphates may also be present from unnatural sources such as fertilizers (ammonium sulphate) or contaminants in soils such as industrial effluent. The essential agents for sulphate attack are sulphate anions. These are transported to the concrete through diffusion in various concentrations in water, together with cations, the more common of which are calcium, magnesium and sodium.

Sulphate attack can be categorized as two separate forms:

1. The well-known 'conventional form of sulphate attack' leading to the formation of ettringite and gypsum

2. The more recently identified type producing thaumasite

Both conventional form and thaumasite sulphate attack can occur together in buried concrete. Thaumasite sulphate attack can only be occurred at a temperature below 15°C. Generally sulphate attack can only be identified when the physical signs of degradation such as expansion, surface erosion or softening of the cement paste matrix are observed. Figure 2.12 shows the severe suphate attack in a 30 year old highway bridge structure exposed to wet, pyritic clay fill.

Figure 2.12. Severe suphate attack in a bridge structure exposed to wet, pyritic clay fill

Conventional form of sulphate attack

The following factors are essential for the conventional form of sulphate attack to occur where expansive ettringite together with gypsum formed:

a) Source of sulphates, generally from sulphates or sulphides in the ground

b) The presence of mobile groundwater

c) Calcium hydroxide, calcium aluminate hydrate and calcium silicate hydrate (for magnesium sulphate) in the cement matrix.

Mechanism of sulphate attack

Sulphate attack is characterised by the chemical reaction between sulphate ions with the aluminate component, calcium and hydroxyl of hardened Portland cement. The reaction leads to the formation of expansive ettringite and to a lesser extent, gypsum. The reaction, providing there is enough water present, will cause expansion leading to cracking. This in turn will allow further ingress of sulphates and accelerate the degradation process. Sulphates will attack some or all of the three main hydrate components of hardened concrete: Calcium hydroxide, calcium aluminate hydrate, calcium silicate hydrate, depending on the type of sulphate in solution involved.

Attack of calcium aluminate hydrate ($CaO.Al_2O_3.H_2O$) components:

Sulphates will attack the calcium aluminate hydrate component, producing calcium sulfoaluminate (ettringite), an expansive product and soluble hydroxides: e.g.

$$2(3CaO.Al_2O_3.12H_2O) + 3(Na_2SO_4.10H_2O) \longrightarrow \underbrace{3CaO.Al_2O_3.3CaSO_4.31H_2O}_{\text{Ettringite}} \underbrace{+ 2Al(OH)_3 + 6NaOH}_{\text{soluble hydroxides}} + 17H_2O$$

In the highly alkaline pore solution (pH>10) provided by the sodium, potassium and calcium hydroxides liberated during the cement hydration reactions, sulphate ions that have penetrated the hardened concrete react with calcium aluminate hydrate to form calcium sulfoaluminate hydrate (ettringite, $CaO.Al_2O_3.3CaSO_4.31H_2O$). The formation of this mineral can be destructively expansive since it has a solid volume greater than the original constituents and it grows as myriad acicular (needle-shaped) crystals that can collectively generate high internal stresses in the concrete. Sulphates will attack the calcium hydroxide component in an ‘acid’ type attack, producing crystalline calcium sulphates (gypsum) and soluble hydroxide: e.g.

$$Ca(OH)_2 + Na_2SO_4.10H_2O \longrightarrow \underbrace{CaSO_4.2H_2O}_{\text{Gypsum}} \underbrace{+ 2NaOH}_{\text{soluble hydroxide}} + 8H_2O$$

This reaction product also has a greater solid volume than the original constituents and in some cases can contribute to degradation of the concrete.

Attack of calcium silicate hydrate ($CaO.SiO_2.H_2O$) components:

Certain sulphates such as magnesium sulphate will also attack the calcium silicate hydrate as well as the calcium aluminate hydrate and calcium hydroxide, producing very severe sulphate attack and expansive product and soluble hydroxides: e.g.

$$3CaO.2SiO_2.aq + MgSO_4.7H_2O \longrightarrow CaSO_4.2H_2O + \underbrace{Mg(OH)_2}_{\text{Low solubility hydroxide}} + SiO_2$$

The low solubility of the hydroxide means that the reaction proceeds until completion resulting in complete destruction of C-S H. If magnesium ions accompany the sulphates, they may also react with calcium hydroxide, producing brucite (magnesium hydroxide, $Mg(OH)_2$) which, because of its low solubility precipitates out of solution, also leading to increase in solid volume. Magnesium ions may also attack calcium silicate hydrates, the principal bonding material in set concrete. The first effect of the conventional form of sulphate attack is to increase the strength and density of the concrete as the reaction products fill the pore space. When it is filled, further ettringite formation induces expansive internal stresses in the concrete which, if greater than the tensile strength of the concrete, will expansively disrupt the affected region. This cracking together with white crystalline accumulations is the characteristic signs of the conventional form of sulphate attack.

2.7.5. Alkali Silica Reaction

The Alkali-Silica Reaction (ASR) is a reaction which occurs over time in concrete between the highly alkaline cement paste and reactive non-crystalline silica, which is found in many common aggregates. This reaction causes the expansion of the altered aggregate by the formation of a swelling gel of Calcium Silicate Hydrate (CSH). This gel increases in volume with water and exerts an expansive pressure inside the material, causing spalling and loss of strength of the concrete, finally leading to its failure. So, ASR can cause serious expansion and cracking in concrete, resulting in critical structural problems that can even force the demolition of a particular structure. The mechanism of ASR causing the deterioration of concrete can be described in four steps as follows:

1. The alkaline solution attacks the siliceous aggregate to convert it to viscous alkali silicate gel.
2. Consumption of alkali by the reaction induces the dissolution of Ca^{2+} ions into the cement pore water. Calcium ions then react with the gel to convert it to hard Calcium Silicate Hydrate.
3. The penetrated alkaline solution converts the remaining siliceous minerals into bulky alkali silicate gel. The resultant expansive pressure is stored in the aggregate.
4. The accumulated pressure cracks the aggregate and the surrounding cement paste when the pressure exceeds the tolerance of the aggregate.

The Alkali-Aggregate Reaction (AAR) is a general, but relatively vague, expression which can lead to confusion. The Alkali-Silica Reaction is the most common form of alkali-aggregate reaction. Two other types alkali-aggregate reaction are:

- the alkali-silicate reaction, in which layer silicate minerals (Clay), sometimes present as impurities, are attacked, and;
- the alkali-carbonate reaction which is an uncommon attack on certain argillaceous limestones, likely involving the expansion of the mineral Brucite ($Mg(OH)_2$.

Following are the various factors that promote Alkali – aggregate reaction.

1. Reactive type of aggregate
2. High alkali content in Cement
3. Availability of Moisture
4. Optimum temperature conditions (Ideal temperature for promotion of AAR is 10 – 38 degree centigrade)

2.8. Causes and stages of disintegration

The other causes like construction and / or design and detailing deficiencies, material and workmanship deficiencies and the effect of environment may be the reason for deterioration of structures. The deficiencies could be due to external or internal contributory factors as given in Table 2.2.

Table 2.2. Internal and External causes and stages of disintegration

Cause of distress	Initial damage stage of RCC	Accelerated damage stage of RCC
Construction deficiency: (INTERNAL) PHYSICAL:		
1. High W/C ratio	High capillary porosity in cement paste which allows the aggressive chemicals from its environment to penetrate easily and allows the concrete / reinforcement to get affected at an accelerated rate and initiate the onset of corrosion.	1. Carbonation reaches reinforcement level to depassivate steel a d initiate corrosion. 2. Increased volume of corrosion product exerts pressure on surrounding concrete 3. Increased cracking of concrete allows easy permeation of atmospheric. 4. Accelerated corrosion to cause increased cracking, then spalling of concrete. 5. Corrosion reduces steel bar cross section. 6. Corrosion product behind reinforcement pushes bar outward to make it buckle and fail in compression to cause collapse.
2. Inadequate Curing	- do -	Same as above.
3. Poorly graded aggregates	Porous concrete due to air voids and allows the aggressive chemicals from	Same as above

Cause of distress	Initial damage stage of RCC	Accelerated damage stage of RCC
	its environment to penetrate easily and allows the concrete / reinforcement to get affected at an accelerated rate and initiate the onset of corrosion.	
4. Inadequate compaction	- do -	Same as above
5. Shuttering joints not slurry tight	Honey combed concrete due to bleeding when cement paste is replaced by air voids near surface and allows the aggressive chemicals from its environment to penetrate easily and allows the concrete / reinforcement to get affected at an accelerated rate and initiate then onset of corrosion.	Same as above
6. Cover thickness being lesser	Protective cover thickness against external/environment chemical attack reduced and allows the concrete reinforcement to get affected early.	Same as above
7. Wrong placement of reinforcement	Cracking / crushing of concrete to cause macro-cracking form its environment to penetrate easily and allows the concrete / reinforcement to get affected at accelerated rate and initiate the onset of corrosion.	Same as above
CHEMICAL: 1. **Chloride** penetration either through construction water and / or aggregate.	Depassivation of steel reinforcement locally and formation of galvanic cells to initiate corrosion of bar.	1. Chloride ion acts as current carrier in presence of water and causes localized corrosion of reinforcement.
2. **Sulphate** penetration either through construction water or aggregate or diffusion from adjacent environment.	Formation of C_4A_3S an expansive product to cause disintegration due to bursting force within hardened concrete. (slow process)	2. Same as 2 – 6 of above.
3**. Reactive aggregates** containing amorphous silica or strained quartz to cause Alkali Silica Reaction(ASR).	Formation of expansive gel around aggregate particles of water and disintegration of concrete due to bursting force in hardened concrete. (slow process)	1. Bursting force n hardened concrete causes cracking and disintegration of concrete to make it weak in strength and more permeable. 2. Same as 2 – 6 of above
DESIGN DEFICIENCY 1. Wrong assessment of design loads	Deflection, crushing / cracking of structural member allows the aggressive chemicals from its environment to penetrate easily and allows the concrete / reinforcement to get affected at an accelerated rate	Same as 1 – 6 of above

Cause of distress	Initial damage stage of RCC	Accelerated damage stage of RCC
	and initiate the onset of corrosion.	
2. Factors like shrinkage, thermal movement, structural behavior, etc not considered	Disintegration of concrete, shrinkage cracks allows the aggressive chemicals from its environment to penetrate easily and allow the concrete/reinforcement to get affected at an accelerated rate and initiate the onset of corrosion.	Same as 1 – 6 of above
ENVIROMNEMNTAL: (EXTERNAL) PHYSICAL:		
1. Heating / cooling	Surface disintegration and micro cracking allows the aggressive chemicals from its environment to penetrate easily and allows the concrete/reinforcement to get affected.	Same as 1 – 6 of above
2.Wetting / drying	Increase capillary porosity due to leaching away of water soluble salts results in deletion of water soluble calcium hydroxide reducing the alkalinity which further allows the aggressive chemicals from its environment to penetrate easily and allows the concrete/reinforcement to get affected.	2. Same as 2 – 6 of above
3.Abrasion of surface	Surface disintegration and reduced cover thickness allows the aggressive chemicals from its environment to penetrate easily and allows the concrete/reinforcement to get affected.	Same as 1 – 6 of above
CHEMICAL EFFECT		
1. Chloride attack from sullage of toilets, sea water, atmospheric gases, acids, etc.	When chloride ions penetrate and reach reinforcement level, cause local depassivation of steel bars and formation of galvanic cells locally and initiate corrosion.	Same as 1 – 6 of above
2. Sulhate attack from soil, sub-soil water, industrial waste / gases, acids, etc.	React with calcium aluminate hydrate (C-A-H) in cement paste, it forms expansive compound, which exerts bursting pressure to cause disintegration and cracking of concrete up to depth of permeation to allow the aggressive chemicals from its environment to penetrate easily and allows the concrete/ reinforcement to get affected.	Same as 1 – 6 of above 2. Same as 2 – 6 of above 1. Bursting force in hardened concrete causes cracking and disintegration of concrete to make it weak in strength and more permeable. 2.Same as 2 – 6 of above

2.9. Summary

Causes, types and control of cracks are discussed in detail.

2.10. Keywords

Cracks – shrinkage – Thermal – Corrosion – Micro, Macro cracks – carbonation – chlorides – sulphates – ASR-AAR.

2.11. Intext Questions

1. How will you reduce cracks in concrete structure?
2. Mention the causes of concrete cracks.
3. What are the types of concerete cracks? Explain in detail.
4. What are called micro and macro cracks?
5. Explain about the following:
    a. Carbonation
    b. Sulphates attack
    c. Alkali Silice Reaction
    d. Alkali Aggregate Reaction

2.12. References:

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❑

CORROSION DETERIORATION OF CONCRETE STRUCTURES

Objective

- ❖ To know about causes, effect and control of corrosion of reinforcement in concrete.

Contents

3.1. Introduction
3.2. Effects of corrosion on reinforced concrete
3.3. Modeling of corrosion deterioration
3.4. Chemical Process of Corrosion
3.5. Causes of corrosion
3.6. Corbonation induced corrosion
3.7. Chloride induced corrosion
3.8. Experimental investigation
3.9. Corrosion control
3.10. Summary
3.11. Keywords
3.12. Intext questions

3.1. Introduction

The existing RC structures such as bridges, offshore platforms and multistoreyed buildings constructed in coastal regions are being affected by corrosion of reinforcement. This reinforcement corrosion in concrete is regarded as the predominant causal factor for the premature degradation of RC structures, leading to ultimate structural failure. RC structures accumulate serious damage within a relatively short period of time due to saline action and poor quality of concrete. The corrosion deterioration accounts for a significant percentage of annual expenditures on maintenance, repair or replacement.

3.2. Effects of Corrosion on Reinforced Concrete

Reinforced concrete is one of the most durable construction materials. Corrosion of reinforcing steel affects structural response and is the major cause of the deterioration of RC members. Damage resulting from corrosion of steel includes delamination, reduction of steel areas, cracking, debonding between rebar and concrete, and spalling. Thus, the loading capacity and stiffness of an RC component may deteriorate or failure of an RC structure may occur. Figure 3.1 show the diagrammatic representation of cracking-corrosion-cracking cycles.

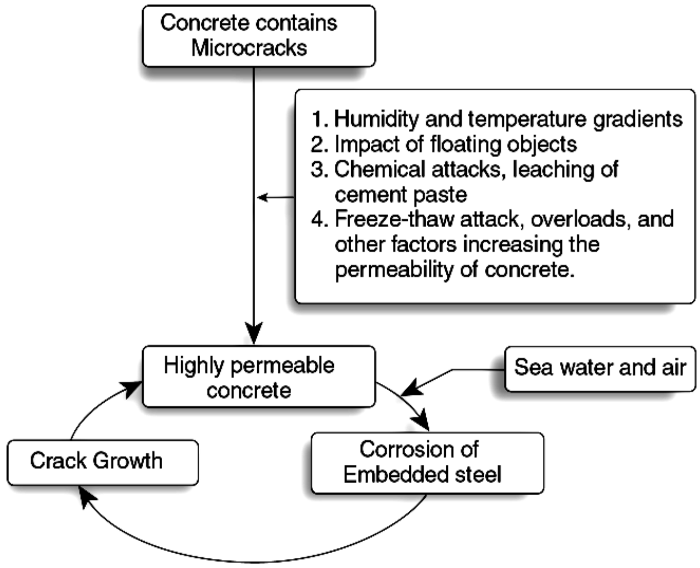


Figure 3.1. Diagrammatic representation of cracking – corrosion – cracking cycles

3.3. Modeling of Corrosion deterioration

Corrosion induced deterioration of reinforced concrete can b modeled in terms of three component steps. (1) Time for corrosion initiation (Ti); (2) Time, subsequent to corrosion initiation, for appearance of a crack on the external concrete surface (Crack Propagation), (Tp); and (3) Time for surface cracks to progress into further damage and develop into spalls,(Td), to the point where the functional service life, **Tf** is reached. Figure 3.2 shows schematic plot of cumulative corrosion damage versus time.

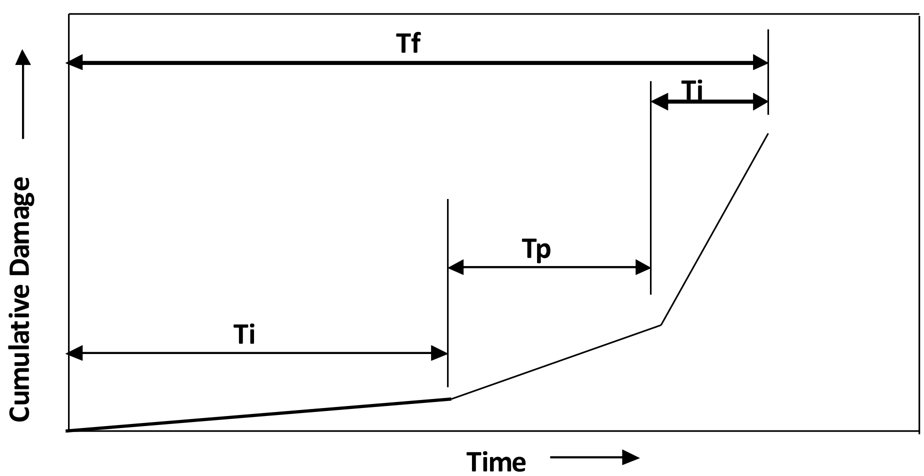


Figure 3.2. Modeling of cumulative corrosion damage versus time

3.4. Chemical Process of Corrosion

Corrosion of steel in concrete involves a complex series of reactions, the proportions of which may vary with environmental exposure and material characteristics. Generally, iron (Fe) atoms pass into solution as positively charged

(Fe^{2+}) hydrated ions at the anodic site and the liberated electrons flow through the metal to cathodic sites, where dissolved oxygen is available to consume them.

In a high pH environment, such as concrete, hydroxide (OH^-) is abundant in solution. Hydroxide ions may react with ion at the steel surface to create ferrous hydroxide, $Fe(OH)_2$. In electrochemical terms, this is the anodic reaction. Simultaneously, at an alternate location on the steel surface, dissolved oxygen (O2) reacts with water (H2O) and electrons released by the anodic reaction to form hydroxide ions (OH-). This is called the cathodic reaction. Together, the anodic reaction and the cathodic reaction form a corrosion cell, and the reactions proceed simultaneously. Further reaction of $Fe(OH)_2$ with water and oxygen results in transformation to the insoluble corrosion products, specifically hydrated iron oxide compounds in solution, which accumulate in the small pores and the interfacial zone around the steel. The iron oxides include ferric oxide (Fe_2O_3, red-brown rust) and magnetite (Fe_3O_4, black rust) (Liu and Weyers, 1998). The associated corrosion reactions are as follows:

Anode: $Fe \longrightarrow Fe^{++} + 2e^-$ (3.1)

Cathode: $O_2 + 2H_2O + 4e^- \longrightarrow 4OH^-$ (3.2)

$Fe^{++} + 2OH^- \longrightarrow Fe(OH)_2$ (3.3)

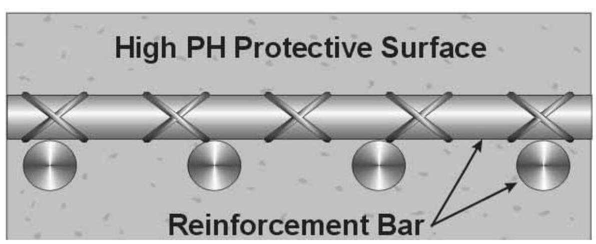
$Fe(OH)_2 + O_2 \longrightarrow Fe_2O_3.H_2O + 2H_2O$ (Red-brown rust) (3.4)

$6Fe(OH)_2 + O_2 \longrightarrow 2Fe_3O_4.H_2O + 4\ H_2O$ (3.5)

$Fe_3O_4.H_2O \longrightarrow Fe_3O_4 + H_2O$ (Black magnetite) (3.6)

3.5. Causes of Corrosion

The causes and effects of corrosion of reinforcement embedded in concrete are shown in Figure 3.3.



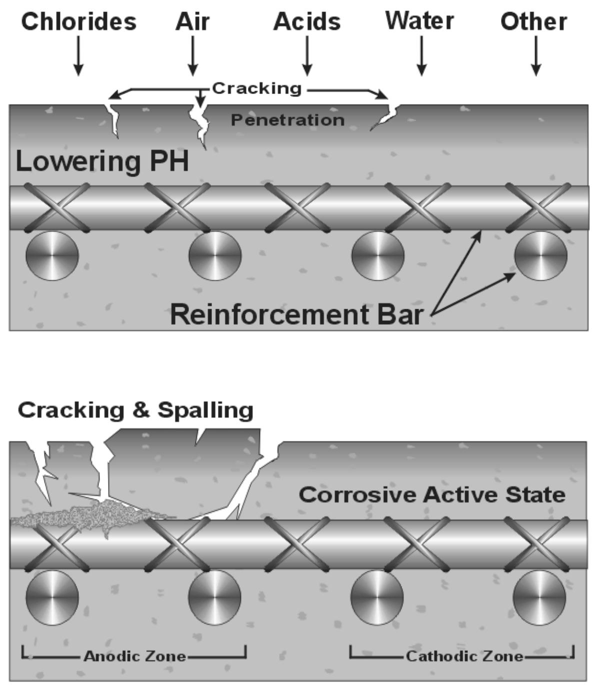


Figure 3.3. Causes and effects of corrosion of reinforcement

The corrosion of steel in concrete mainly occurs due to the diffusion of carbon dioxide (carbonation - induced corrosion) and chloride ions (chloride induced corrosion) into the concrete. Generally, chloride-induced corrosion is more serious than carbonation-induced corrosion. Figures 3.4 and 3.5 show typical symptoms of distress due to corrosion of reinforcement in a RC slab and beam respectively. Cracking along the bar can be an important indication that the reinforcements are subjected to corrosion.

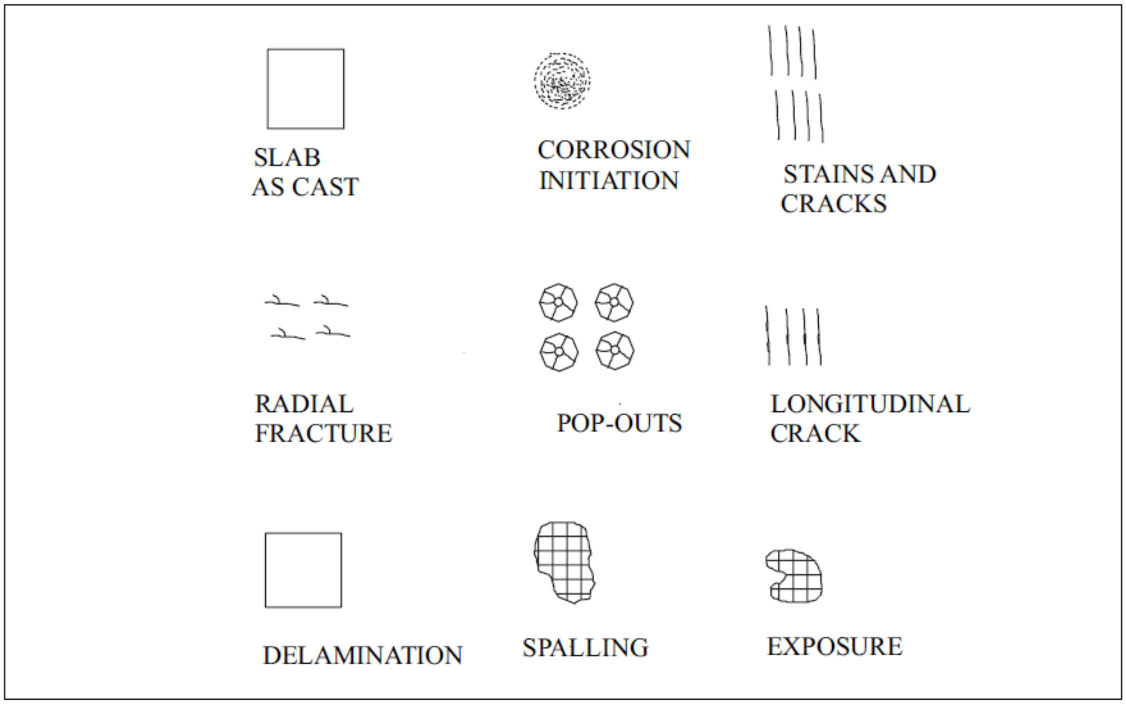


Figure 3.4. Symptoms of corrosion in a reinforced concrete slab

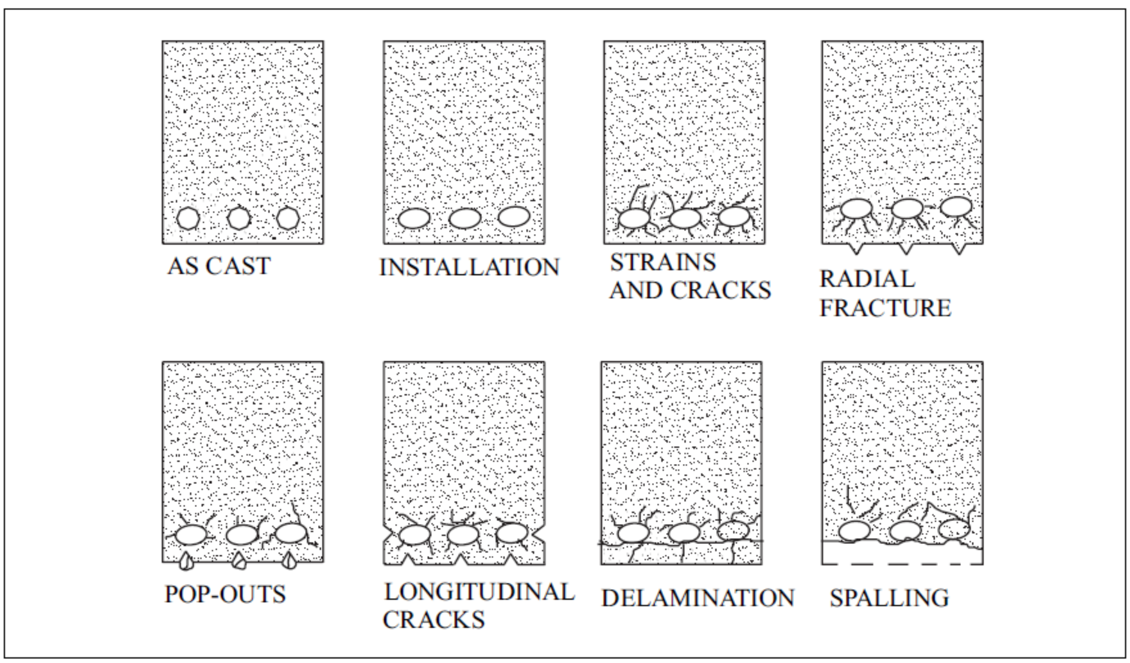


Figure 3.5. Symptoms of corrosion in a reinforced concrete beam

3.6. Carbonation-induced corrosion

Carbonation of moderate to good quality concrete is a slow process, and becomes significant in structures that are many decades of age, or exposed to environments containing high concentrations of carbon dioxide, such as mass rapid transit tunnels. Poor quality concretes with porous internal structure allow faster diffusion of carbon dioxide from the air, and therefore are more susceptible to carbonation. Carbonation occurs slowly in saturated concrete, but is accelerated in concrete where internal relative humidity ranges from 50% to 70%. The reaction product, calcium carbonate, has a pH of about 9.5, which is below the level necessary to sustain the passive layer. Thus, once the carbonation front reaches the depth of reinforcement, so long as sufficient moisture and oxygen are present, spontaneous corrosion may occur, especially at locations where passive layer may be damaged. One method of testing a structure for carbonation is to drill a fresh hole in the surface and then treat the cut surface with phenolphthalein indicator solution. This solution will turn [pink] when in contact with alkaline concrete, making it possible to see the depth of carbonation.

3.7. Chloride induced corrosion

In chloride induced corrosion, chloride ions that diffuse through liquid phases in the concrete to the steel surface can disrupt the passive layer and induce corrosion, even in a high pH environment. Chloride can get into the concrete at the time of mixing or can penetrate into the hardened concrete later on. Chloride ions react with iron compounds in the passive layer to create an iron - chloride complex. The local active areas behave as anodes, while the remaining passive areas become cathodes where reduction of dissolved oxygen takes place. The iron-chloride subsequently reacts with hydroxide ions within the surrounding concrete to form ferrous hydroxide, $Fe(OH)_2$ and in turn corrosion cell forms.

The use of de-icing salts on roadways, used to reduce the freezing point of water, is probably one of the primary causes of premature failure of reinforced or prestressed concrete bridge decks, roadways, and parking garages. The use of epoxy-coated reinforcing bars and the application of cathodic protection has mitigated this problem to some extent. Also FRP rebars are known to be less susceptible to chlorides. Properly designed concrete mixtures that have been allowed to cure properly are effectively impervious to the effects of deicers.

Another important source of chloride ions is from sea water. Sea water contains by weight approximately 3.5 wt.% salts. These salts include sodium chloride, magnesium sulfate, calcium sulfate, and bicarbonates. In water these salts dissociate in free ions (Na^+, $Mg_2^{+,}$ Cl^-, SO_4^{2-}, HCO_3^-) and migrate with the water into the capillaries of the concrete. Chloride ions are particularly aggressive for the corrosion of the carbon steel reinforcement bars and make up about 50% of these ions.

The chloride concentration is measured in terms of the mass of chloride per unit volume of concrete. The chloride concentration necessary to initiate corrosion in reinforced concrete is influenced by many factors including:

- concrete mix proportions
- cement type
- tri-calcium aluminate (C_3A) content of the cement
- w/c ratio
- temperature
- relative humidity
- source of chloride penetration
- Cover thickness

The unit volume of the final corrosion product is as large as six times of the original iron volume. This expansion creates cracking and spalling of corrosion products inside concrete, and finally destroys the integrity of the structural concrete and causes failure of buildings and infrastructures.

3.8. EXPERIMENTAL INVESTIGATION

To study the corrosion effects in beams, small size specimens were cast. Using M 30 grade of concrete, six numbers of beams of size 10 cm × 10 cm × 50 cm were cast. Two numbers of 10 mm diameter and 8 mm diameter reinforcing bars each were provided at bottom and top respectively. Three nos. of 6 mm diameter stirrups were used. To make different degrees of corrosion such as 5.0 %, 7.5 % and 15 % in the beams, a constant current of 0.5 A was applied. The time required for the corrosion process was evaluated using the Faraday's law. The corroded specimens are shown in Figure 3.6. The specimens were tested in the loading frame of 100 kN capacity and the test setup is shown in Figure 3.7. The specimen was placed in the loading frame in such a way that the loads and the reactions were made to act along a straight line through steel rods placed at the respective points of application. A calibrated proving ring was used to indicate the loads applied. The load was gradually applied and increased until the specimen failed. The maximum load applied to the specimen was recorded. The load deflection (midspan) curves for reference beams and corroded beams are shown in Figure 3.8.

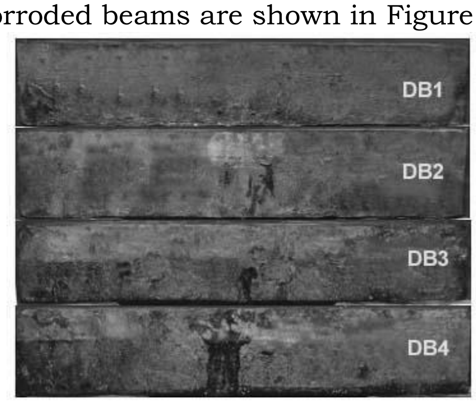


Figure3. 6. Corrosion damaged beams

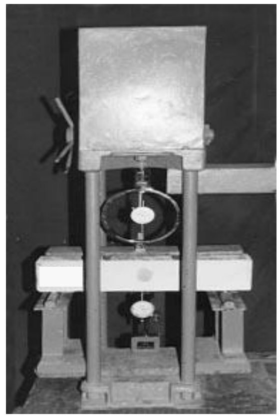


Figure 3.7. Corrosion damaged beams

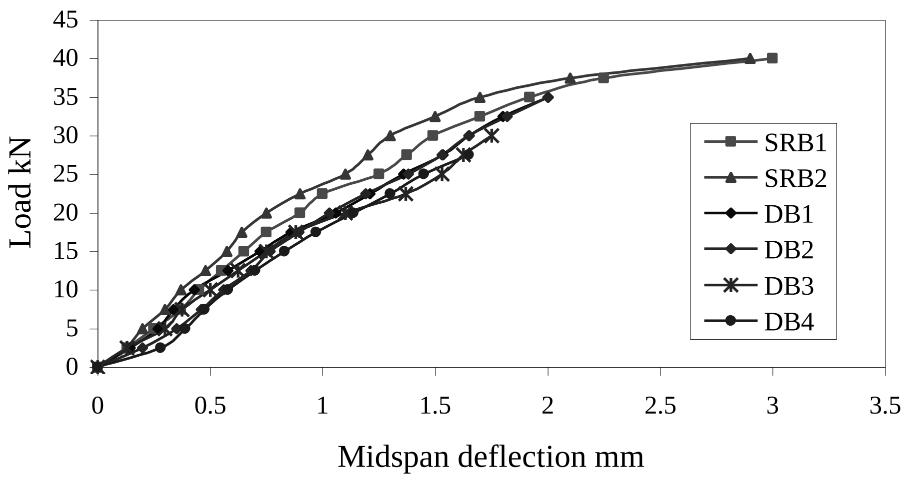


Figure 3.8. Load deflection curves

The average value of flexural strength in reference beams (SRB1 & SRB2) was recorded as 43.26 kN and the maximum deflection at the mid span was observed as 3.32 mm. The percentages of loss of weight in steel, reduction in flexural strength and reduction in deflection with respect to the reference beams are shown in Table 3.1.

Table 3.1. Comparison of damaged beams with reference beam

Serial number	Beam designation	With reference to reference beams		
		% Weight loss	% Reduction in flexural strength	% Reduction in deflection
1	DB1	5.0	10.36	32.60
2	DB2	5.0	11.84	33.70
3	DB3	7.5	19.20	35.24
4	DB4	15.0	36.29	48.80

Thus, the reduction in flexural strength and the deflection at the failure stage of corroded beams with respect to degree of corrosion in reinforcements are clearly seen from the Table 3.1. Further, at a particular load level, the deflection in beams increased with respect to the increase in corrosion damage.

3.9. CORROSION CONTROL

For most environments sufficient durability with respect to protection of reinforcement can be gained by an adequate concrete cover. Most corrosion control measures applied normally can be described as passive measures as they are obtained by adopting proper design and quality control measures and incorporating various types of chemical and mineral admixtures, inhibitors etc. In severe corrosive conditions, active measures such as cathodic protection also need to be provided in addition to the passive measures. For the majority of normal environmental conditions sufficient corrosion resistance of reinforced concrete structures can be achieved by adequate planning, design, composition of concrete, execution and curing of concrete. Various corrosion control measures commonly used are briefed below:

1. Planning and Designing

The structural form and structural detailing should be so selected as to keep concrete away from likely zones of wetting and drying. The design measures such as adequate cover, minimum crack width etc. also needs to be adopted at the planning state itself for ensuring corrosion resistance.

2. Concrete Composition

The type and the quantity of cement are the most influencing factors in the corrosion resistance properties of concrete. In severe environmental conditions, adequate amount of cement and w/c ratio of 0.4 to 0.5 and minimum cement content of 300 kg/m^3 is commonly recommended for outdoor conditions. Blended cements have shown better corrosion resistance as compared to OPC.

3. Construction Practice

The concrete should be adequately workable for achieving sufficient compaction especially in closely spaced reinforcement locations. Adequate moisture curing is a key to achieving good impermeable concrete cover. The required period of curing increases with increasing percentage of pozzolanic materials, decreasing cement content, decreasing environmental humidity and increasing aggresivity of environment. Depending on these parameters, the period of curing may vary from 2 to 28 days.

4. Coating of Rebars

After many years of use of different organic coating systems on reinforcement, electrostatically applied powdered epoxy coatings have been proved to perform better than other coating types. Adequate care needs to be taken while handling and using epoxy coated rebar's in construction as a slight damage or local defect in coating may prove to be detrimental in the long run.

5. Coating of Concrete

Coatings or impregnation on concrete reduce or impede chloride ion penetration, gas penetration (to decrease carbonation) or water penetration. Coatings consist of continuous film applied on the concrete surface with a thickness in the range of 100 to 300 micrometre. The coatings in the liquid form are sprayed or brushed on the concrete surface. Different coatings/seater materials are Acrylic, Butadiene, copolymer, chlorinated rubber, epoxy resins, polluter resins, polyurethane, vinyl etc. The other types of materials used for treating the concrete surface are poreliners which impregnate in the concrete pores and line them with water repellent materials. Materials used in this category as hydrophobic liner are silicons, siloranes and silanes. The third category of seaters is pore blockers which penetrate into the pores and then react with calcium hydroxide and block the pores. The materials available in this category are liquid silicates and silicofluoride.

6. Cathodic Protection

Cathodic protection is a method based on the knowledge that the corrosion of any metal is a result of an electrical current flowing from one part of the metal to the other. Corrosion (i.e., metal loss through oxidation) occurs where current leaves the steel. In the steel reinforced concrete corrosion process, the corrosion products, or rust, can build up excessive stresses in the concrete, which eventually lead to concrete cracking and delamination. In the corrosion circuit, those locations where current leaves the rebar are called the anodes or anodic areas, as opposed to other nearby areas on the rebar which receive current and are called the cathodes or cathodic areas. The cathodic protection process applied to reinforced concrete structures is almost always of an impressed current type and uses the application of direct current to the rebar using an introduced anode material in sufficient quantity to reverse or counteract the natural corrosion current. If this corrosion current is eliminated, corrosion stops and the rebar is said to be under cathodic protection. Where reinforced concrete structures are submerged in seawater than galvanic cathodic protection systems are often employed.

7. Mineral Admixture as Corrosion Inhibitors

The use of fly ash to replace cement is known to have beneficial effect on inhibiting sulphate attack, alkali aggregate reaction, reducing heat of hydration and increasing denseness. The addition calcium nitrite extends the time to corrosion initiation. The corrosion rate is also less with calcium nitrite.

8. Use of Stainless steel bars

Stainless steel containing a minimum of 12% chromium can be used instead of conventional black rods. On contact with air, the chromium forms a thin oxide film on the surface of the steel called passive layer and it resists corrosion. The addition of other elements such as nickel and molybdenum enhances the passivity and thus improves corrosion resistance.

9. Fibre Reinforced Plastic bars

Non-ferrous reinforcement using manmade fibres such as glass, carbon and aramid can be used as reinforcement in concrete. The fibres are used either in the form of ropes or combined with suitable resins to form rods. Because of their relatively low elastic modulus, they have been used for prestressing.

10. Improving the concrete

Codes and standards aim to achieve good durability of reinforced and prestressed structures in aggressive environments by specifying the following:

1. High cement content
2. Low water/cement ratio
3. Suitable minimum thickness of cover to the reinforcement
4. Use of mineral and chemical admixtures

Generally, codes and standards specify a number of different exposure conditions with limiting quantities for the above, which vary depending on the severity of the environment.

3.10. Summary

The causes, effects and control of corrosion in reinforcement are discussed in this lesson.

3.11. Keywords

Corrosion – causes – control – effects of corrosion.

3.12. Intext Questions

1. Explain about chemical process of corrosion.
2. What are the causes of corrosion?
3. What are the control measures of corrosion of reinforcement?

❑

LESSON – 4

DAMAGE ASSESSMENT OF STRUCTURES

Objectives

- To understand the composition, properties and tests on concrete.
- To study the various damage assessment procedure to assess the quality of concrete structure.

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4.8.18. Damage Assessment by vibration technique
4.9. Summary
4.10. Keywords
4.11. Intext Questions

4.1. Non-destructive testing

Non-destructive testing can be used in those situations as a preliminary to subsequent coring. Typical situations where non-destructive testing may be useful are, as follows:

- quality control of pre-cast units or construction *in situ*
- removing uncertainties about the acceptability of the material supplied owing to apparent non-compliance with specification
- confirming or negating doubt concerning the workmanship involved in batching, mixing, placing, compacting or curing of concrete
- monitoring of strength development in relation to formwork removal, cessation of curing, prestressing, load application or similar purpose
- location and determination of the extent of cracks, voids, honeycombing and similar defects within a concrete structure
- determining the concrete uniformity, possibly preliminary to core cutting, load testing or other more expensive or disruptive tests
- determining the position, quantity or condition of reinforcement
- increasing the confidence level of a smaller number of destructive tests
- determining the extent of concrete variability in order to help in the selection of sample locations representative of the quality to be assessed
- confirming or locating suspected deterioration of concrete like corrosion resulting from such factors as overloading, fatigue, external or internal chemical attack or change, fire, explosion,
- environmental effects
- assessing the potential durability of the concrete
- monitoring long term changes in concrete properties
- providing information for any proposed change of use of a structure for insurance or for change of ownership.
- investigating the homogeneity of concrete mixing
- finding the lack of grout in post tensioning ducts

- determining the density and strength of concrete in a structure
- determining the location of in-built wiring, piping, ducting, etc.
- determining if there is a bond between epoxy bonded steel plates and concrete members.

4.2. Types of concrete structures

Concrete is a mixture of stone and sand held together by a hardened paste of cement and water. When the ingredients are thoroughly mixed they make a plastic mass which can be cast or moulded into a predetermined size and shape. When the cement paste hardens the concrete becomes very hard like a rock. It has great durability and has the ability to carry high loads especially in compression. Since it is initially plastic it can be used in various types of construction; however the forms used to produce the final shape can not be removed until the concrete has developed enough strength by hardening. Where tensile stresses are imposed on the concrete, it must be reinforced with steel.

4.2.1. Reinforced concrete

Reinforced concrete is a combination of concrete and steel. Alone concrete is very strong in compression but very weak in tension. Since concrete bonds firmly to steel reinforcement the combination acts as one material which offers high compressive strength, high tensile strength and high shear strength. Reinforcement in concrete also helps to control cracking such as shrinkage and surface cracking. There are two main types of reinforcement: deformed bars (i.e. with grooves) and mesh sheets, such as rectangular mesh, square mesh and trench mesh. The position of reinforcement is always shown on drawings. Steel reinforcement must be securely fixed in the right position. To ensure that the correct concrete cover over the reinforcement is being achieved, plastic bar chairs or concrete blocks should be used at the specified distance from the forms. Timber, bricks or stones should not be used. Reinforcement may be bent, hooked or lapped to suit design requirements and improve the bond between the steel and the concrete. The reinforcement must be clean and free from grease, dirt or flaky rust. It is necessary to have enough room to place and compact the concrete around the steel. Congested reinforcement will make compaction using internal vibrators difficult and may result in voids. Reinforced concrete is used for concrete slabs, decks, concrete pavements, columns, walls, concrete bridges, retaining walls etc.

4.2.2. Prestressed concrete

The basic principle of prestressed concrete is that superimposing compressive stresses eliminate tensile stresses in the concrete. This involves the installation of high tensile strength steel as reinforcement, stretching the steel by applying a pre-stressing force, and holding the tension. The pre-stressing force in the steel wire or strand is transferred to the concrete, placing the concrete under compression. Pre-stressing of beams is done either by pretensioning or post tensioning. In pre-tensioning, the high strength steel wires or strands are tensioned against a fixed external anchorage before the concrete is poured. The concrete is then poured into the forms around the steel to develop bond: After the concrete has hardened, the pre-stress is transferred to the concrete by releasing the steel wires from the anchorage. In post tensioning, after the concrete has hardened, the high strength steel tendons are passed through ducts cast into the concrete and then tensioned. The tension is then transferred to the concrete putting it under compression. To protect the steel tendons from corrosion, the ducts are filled with grout. The location of pre-stressing tendons is sometimes required, as assurance that the ducts have been properly grouted. One of the common defects in post tensioned concrete bridges is the lack of grout in the post tensioning ducts. A lack of grout may allow the ingress of water and possibly the initiation of corrosion. As the failure mode of these bridges is “brittle” it is crucial to identify the ungrouted sections. Possible NDT investigation procedures include:

- ❖ Radiography - this method requires a people exclusion zone to be maintained during testing for safety reasons.
- ❖ Ground penetrating radar to locate the tendon ducts followed by careful drilling to check
- ❖ whether the duct is fully grouted or a void exists. An endoscope can be used to view any voids.

4.3. Composition of concrete

The main ingredients of concrete are:

- ❖ cement
- ❖ coarse aggregate (i.e. screenings, gravel, etc.)
- ❖ fine aggregate (i.e. sand)
- ❖ chemical admixtures (if necessary)
- ❖ water
- ❖ Acceptable concretes usually have proportions within the following ranges (by volume)
- ❖ water - 15% to 20%
- ❖ aggregate (coarse and fine) - 78% to 63%
- ❖ paste (water + cement) - 22% to 37%

It should be noted that the aggregates in the concrete mix constitute by far the bulk of the mass. The properties of the concrete produced depend upon the amount and type of materials used, and the way they are mixed, handled, compacted, finished and cured.

4.3.1. Cement

There are many types of cement available for making concrete. Some countries have a national standard, which prescribes the requirement for any cement manufactured or used in that country. In India, various types of Portland cements available are:

- ❖ 33 Grade Ordinary Portland Cement (IS:269)
- ❖ 43 Grade Ordinary Portland Cement (IS:8112)
- ❖ 53 Grades Ordinary Portland Cement (IS:12269)

- ❖ Rapid Hardening Portland Cement (IS:8041)
- ❖ Portland Slag Cement (IS:455)
- ❖ Portland Pozzolana Cement (Fly Ash based) (IS:1489 Pt-I)
- ❖ Portland Pozzolana Cement (Calcinated Clay based) (IS:1489 Pt-II)
- ❖ Hydrophobic Cement (IS:8043)
- ❖ Low Heat Portland Cement (IS:12600)
- ❖ Sulphate Resisting Cement (IS:12330)

Each type of cement produced has a different chemical composition and fineness and gives different properties to a concrete.

4.3.2. Portland cement

There are four main types:

(1) General purpose cement - the most common and the one used for the majority of structures.

(2) High Early strengths cement - gains strength quickly.

(3) Low Heat cement - produces less heat than general purpose and high early strength cement. It gains strength more slowly.

(4) Sulphate Resisting cement - cement that resists sulphate attack.

4.3.3. Blended cements

There are also two types of Blended Cements. These are mixtures of Portland cement, and either fly ash (FA) or blast furnace slag (SA). Blended cement generally has slower rate of strength gain, and less heat is generated during curing. However, with adequate curing, impermeable and durable concrete with higher strength than that of normal cement can be achieved.

4.3.4. Other cements

Other cements include white and coloured cements, which are used for decorative finishes.

4.3.5. Mixing water

Water, which is necessary for hydration, must be clean and fresh and not contain any impurities since these may affect the concrete properties. It is generally accepted that water, which is fit for drinking, is suitable for making concrete. Seawater should not be used in making concrete, particularly reinforced concrete, as it will corrode the steel reinforcement. Bore water must be analysed first. Industrial, waste or brackish water should not be used.Sugars and detergents have a retarding effect on the setting properties of concrete and, therefore, should not be added to a concrete mix.

4.3.6. Aggregates and their properties

Coarse aggregates are stones that are more than 5 mm in diameter and are either crushed rock generally from quarries, or gravels excavated from pits or dredged from river beds. Fine aggregates consist of fine and coarse sands or crushed rock finer than 5 mm. The aggregates used should cover a range of sizes. They should be clean and free of any contaminating substances, which may adversely affect the setting time, strength or durability of the concrete, or corrode the steel reinforcement. Aggregates should not contain:

- weak substances such as pieces of wood, humus or coal
- dirt, clay dust or silt coatings. These reduce the bond between aggregates and the cement paste
- water soluble salts such as sulphates or chlorides.

Aggregates should be strong, hard and durable in order to develop the full strength of the cement paste and maximum wear resistance of the concrete. Crumbly or flaky rocks like sandstone, slates or shales should not be used since they lack strength. Chert aggregates should not be used to avoid alkali-silica reaction. The shape of the aggregate particles is also important since the shape affects both the workability and strength of the concrete. Smooth rounded aggregates produce workable and easy to handle concrete. Angular materials tend to give a stronger concrete but reduce workability. Flaky and elongated materials promote segregation and reduce workability. They also require more sand and cement. The use of these aggregates should be limited. There are certain types of aggregates that can react with cement in the presence of moisture. Such reactions result in expansive compounds, which crack and deteriorate the concrete. Suspect aggregates should not be used.

4.3.7. Chemical admixtures

Chemical (usually in liquid form) can be added to concrete to change its properties. They usually affect the time it takes for concrete to harden and the workability of freshly mixed concrete. The most common types of chemical admixtures are:

(1) Set-accelerating admixtures, which speed up concrete setting

(2) Set-retarding admixtures, which slow down concrete setting

(3) Water reducing admixtures (or plasticizers), which help to improve the workability of concrete

(4) High range water reducing admixtures (super plasticizers) – these help improve workability of concrete and its ability to flow into congested areas of steel reinforcement

(5) Air-entraining admixtures – these put air bubbles into the concrete and make the concrete more workable and cohesive. They also reduce segregation. They are very useful in cold weather where they improve durability.

(6) Super plasticizing admixtures

Admixtures should be used in a controlled manner as part of the overall concrete mix design. Misuse can be detrimental to concrete’s properties.

4.4. Process of concrete manufacture

The process of concrete manufacture is simply:

Aggregates + Cement + Water + Chemical Admixtures = Concrete

However, the place of manufacture can either be at a construction site as a small batch produced in a portable concrete mixer or at a large batching plant at the construction site or transported by concrete mixing truck from a concrete plant some distance from the construction site. In the latter case the concrete is called ready mix concrete. If ready mix concrete is being ordered from a concrete plant the manufacturer needs to know the

- intended use of it (i.e. kerb, slab, etc.)
- amount required in cubic meters
- strength required (i.e. Megapascals, MPa)
- slump in mm
- maximum size aggregate (i.e. 14 mm, 20 mm, etc.)
- method of placement (i.e. pump, off the chute, etc.), and any admixtures required.

The concrete mix design used must take into account the required properties of the concrete in the plastic state, the method of placement and the in-service conditions of the concrete (i.e. traffic load, exposure conditions, chemicals, etc. However, the first factors to be considered are the desired compressive strength and slump since these are usually used to specify the concrete required. The proportions of each material in the mixture affect the properties of the final concrete, as follows:

- As the cement content increases both strength and durability increase.
- As the water content increases the concrete becomes weaker; hence, there should just be enough water to make the mix workable.
- As the water/cement ratio increases, strength and durability decrease.
- As the fine aggregate increases the mix becomes sticky and, after compaction, the top few millimeters of concrete become very sandy.
- As the coarse aggregate increases the mix becomes bony and some of the stones can protrude from the surface after compaction.

When concrete is placed in the formwork after thorough mixing, care must be taken not to damage or move the formwork or the reinforcing steel. Also care must be taken to ensure that the concrete does not segregate. For instance the concrete should not be dropped from heights greater than 2.0 meters. The formwork is filled by starting to place the concrete from the corners of the formwork, and from the lowest level if the surface is sloping. Place each load of concrete into the face of the previous plastic concrete, not away from it. Deposit the concrete in horizontal layers and compact before the next layer is placed. Do not place the concrete if the air temperature is below 5°C or above 35°C and never spread concrete with a poker vibrator as segregation will occur. At all times avoid delays. The concrete is then compacted by vibrating the concrete to force the air out and fill all the voids. Concrete is compacted to make it dense, strong and durable. Both external and internal vibration can be used. During external vibration a mechanical screed is used to compact flat slabs. Two workers pull the screed along the top of the forms and external vibrators are attached to the formwork. For internal vibration a poker vibrator is placed in the concrete while the concrete is still in the plastic state. It is kept vertical and taken out very slowly. This is to avoid making holes in the plastic concrete. There are different sizes of poker vibrators. To prevent cold joints the poker vibrator should be long enough to reach the previous layer of concrete. The internal vibrator should not be vibrated at any point for more than 15 seconds. Excessive vibration should be avoided. The formwork should not be touched with the poker. Do not rest the poker vibrator on the reinforcement. Do not move the concrete with the poker vibrator. Use a shovel if the concrete has to be moved. The required appearance of the concrete is obtained by levelling and smoothing. Levelling and smoothing are done by screeding, floating or trowelling. Initial finishing takes place right after placing and vibrating. The concrete is screeded (with a screed board) to the level of the formwork and bullfloated if necessary and left to set.

As the concrete sets, bleed water comes to the surface. Final finishing can only begin when this bleed water dries up, and the concrete can support finishers with only a slight indentation (about 5 mm). Any area with free surface water should not be finished since if the finishing is too early a weak surface and laitance will be produced. Cement should not be used to dry up surface water since this will produce a weak surface and cause cracking. Brooming, colouring or patterned finishes can be applied. Final finishing involves floating, trowelling, edging and jointing.

Floating is done using a wooden hand float or power float. Floating helps to smooth irregularities, embed large aggregate and close minor cracks, which can occur as the surface dries out. Hand floats produce a rougher texture. Steel trowelling is done after floating is finished. It provides a smooth, dense and hard surface, which is also durable and easy to clean. This kind of surface is slippery when wet. The surface should be trowelled at least twice. Trowelling can be done by hand or power trowel. Slab edges are finished with a special edge tool. This gives a neater and stronger edge. Joints are preplanned and cut into concrete during finishing. Redo edges and joints after trowelling to maintain uniformity and fine lines. During or after placing and finishing it may be necessary to protect concrete from the weather. After the concrete has been finished it must be cured. Curing is the process whereby the concrete is kept moist to maximize the concrete's strength and durability by maintaining the hydration reaction as long as possible to produce more cement products.There are two types of curing methods.

(1) Methods that supply more moisture to the concrete, e.g. ponding, sprinkling and wet coverings (i.e. hessian or sand). This prevents the concrete from crazing or cracking due to drying.

(2) Methods that stop the loss of moisture by sealing the surface, e.g. leaving the forms in place, covering with plastic sheets or using spray-on compounds.

Concrete should be cured for as long as practicable since concrete becomes stronger and more durable with longer curing. It is preferable to cure concrete for at least seven days. Both hot and cold temperatures can cause problems to concrete, particularly in its plastic state and early curing period. Adverse weather conditions also include dry, windy, low humidity or frost conditions. The main problems with hot weather are associated with cracking since the concrete stiffens quickly (loses its workability) and it is more difficult to place and finish. The shrinkage of the concrete also increases, increasing the tendency for cracking of the concrete surface (i.e. plastic shrinkage cracking). It also increases the danger of cold joints forming. In cold weather concrete takes much longer to set, gain strength and finish. Below freezing point the water in the concrete turns to ice which expands and can cause cracking of concrete.

4.5. PROPERTIES OF CONCRETE AND THEIR CONTROL

4.5.1. Plastic-state concrete

The two most important properties of plastic state concrete are workability and cohesiveness.

Workability describes the ease with which concrete is mixed, handled, placed, compacted and finished. Concrete, which is stiff or dry, may be difficult to mix, handle, place, compact and finish. Concrete, which is runny or wet, may be easy to place but more difficult to handle and properly compact to a dense material. There are many factors, which affect workability:

- The mix becomes harsher and less workable if the amount of cement is reduced provided the amount of aggregate remains the same.
- The mix becomes more workable if the amount of cement is increased provided the amount of aggregate remains the same.
- However, an excessive amount of cement produces a very sticky and unworkable mix. If the aggregate grading, size and shape are considered:
- Well-graded aggregates with different particle sizes (i.e. from large - about 20 mm, to small - about 14 to 10 mm) produce a more workable concrete. Also, well graded aggregates that are smooth, round and as large as possible improve workability.
- Rough, angular aggregates produce less workable concrete.
- Chemical admixtures increase the workability of concrete by lubricating and dispersing the cement particles.
- Never make concrete more workable by just adding water. Increasing the water content without an increase in cement content lowers the strength and durability of concrete.
- To make a more workable mix, add more cement (paste), use well graded aggregates and chemical admixtures.
- Cohesiveness measures how well the concrete holds together. Factors affecting

Cohesiveness, are:

- ❖ A mix that has too much water will not be cohesive and may separate and bleed. A dry mix can crumble, with the coarse aggregate segregating from the cement paste and sand.
- ❖ A well graded aggregate gives a more cohesive mix. Less fine aggregate (sand) gives a bony mix, which tends to segregate. Excess fine aggregate makes the concrete cohesive, but sticky and difficult to work and place.

4.5.2. Hardened concrete

The two most important properties of hardened concrete are durability and strength. Durability is described as the ability of concrete to resist wear and tear and other inservice conditions without breaking up. Concrete durability increases with strength. Durable concrete is dense and watertight. Durability is very important to protect steel in reinforced concrete.

Compressive Strength is a measure of concrete strength in the hardened state. Concrete is very strong in compression. It is not strong in tension because it has a low tensile strength.

Durability and strength increase with lower water content, higher cement content, higher densities, extended moist curing and correct type of cement. Therefore, if water-to-cement ratio is altered by raising water content, the concrete will be less durable and weaker. Proper compaction will also give higher densities and improve strength and durability. Curing time is also important. The longer the concrete is cured and kept damp, the stronger and more impermeable and durable it will be a lower cement content means weaker and less durable concrete. Different types of cement may gain strength quickly or slowly. They also have different resistance to aggressive conditions.

4.5.3. Durability of concrete

Durability can be defined as the ability of concrete to withstand the damaging effects of the environment and of its service conditions until it reaches a minimum level of performance. Durability is the performance of material against a complex nature of environmental effects on structures. Although, concrete or materials can be designed to be durable but in order to get true improved performances, a holistic approach encompassing the elements of architectural and structural design, processes of execution, inspection and maintenance procedures should be taken into consideration. The principal elements of durability consist of the combined transportation of moisture, chemical ions and heat, both within the concrete matrix and the microclimate - the surroundings and the parameters which influences the mechanism of transportation e.g. permeability, absorption and diffusion. Presence of moisture in the concrete is the most important factor in terms of deterioration of durability. As the moisture becomes the medium to carry chemicals, it needs an interconnected pore distribution inside the concrete matrix in order to move around. So the pore type, size, distribution and cracks including micro and macro cracks are the contributing factors of moisture transportation and degradation of concrete. Figure 4.1 shows illustration of permeability and porosity.

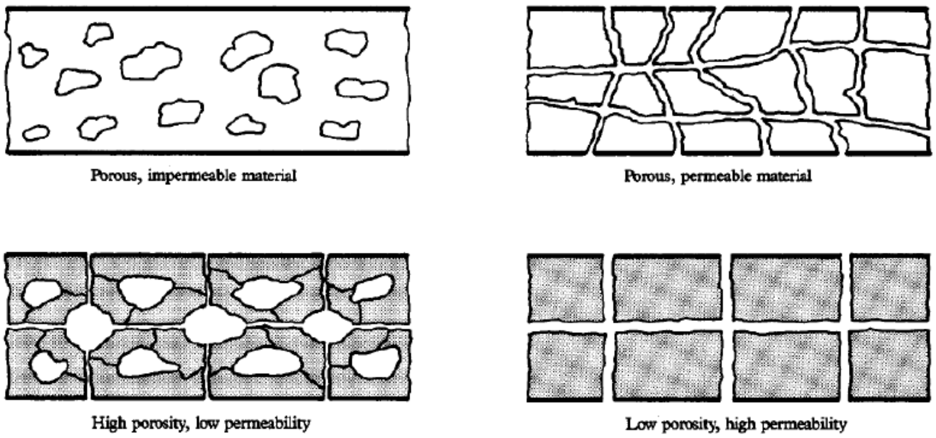


Figure 4.1. Illustration of permeability and porosity

Permeation and transportation mechanism:

Permeation of water and gasses can be divided into three distinct phenomena:

- Permeability
- Absorption
- Diffusion

Figure 4.2 has schematically illustrated the transportation mechanism or permeation properties of gas and liquids through concrete.

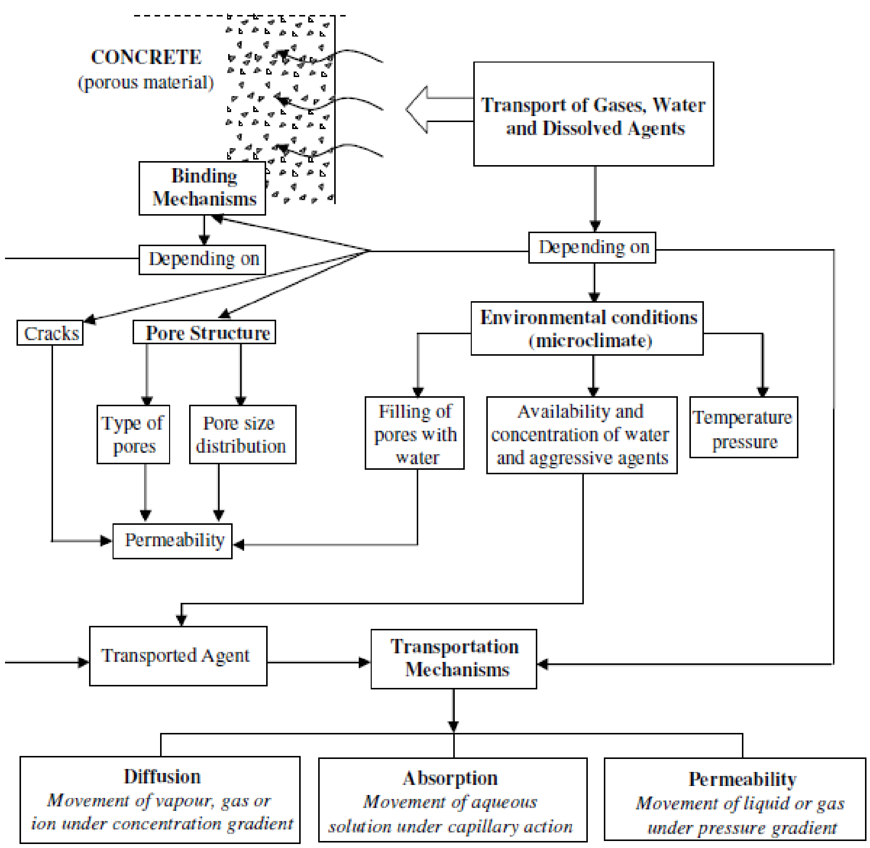


Figure 4.2. Illustration of permeation properties of concrete

4.5.4. Permeability of concrete

Permeability is the property of concrete which quantitatively measure the ease of flow under a pressure differential through the concrete mass. Permeability is a function of the pressure gradient, capillary pore size and pore interconnections in the concrete. Structures subject to water pressure such as dams, tunnel linings, water retaining structures, port, harbour, off-shore oil platforms are subjected to permeability. The factor influencing the permeability of concrete is shown in Figure 4.3.

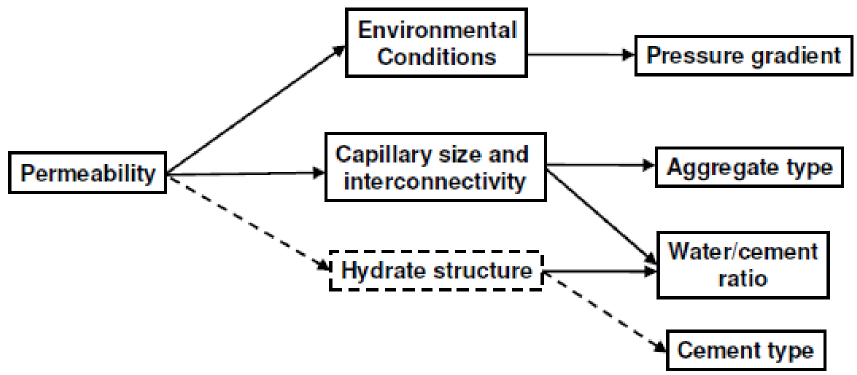


Figure 4.3. Factors affecting permeability of concrete

As deterioration process in concrete begins with penetration of various aggressive agents, low permeability is the key to its durability. Concrete permeability is controlled by factors like water-cement ratio, degree of hydration / curing, air voids due to deficient compaction, micro cracks due to loading and cyclic exposure to thermal variations. Apart from good quality materials, satisfactory proportioning and good construction practice, the permeability of the concrete is a direct function of the porosity and interconnection of pores of the cement paste. Interconnected porosity is related to:

Capillary porosity	-	High w/c ratio
	-	Inadequate curing
Air voids	-	Improper compaction
Micro cracks	-	Loading effects
	-	Weathering
	-	Initial care
	-	After care
	-	Secondary effects
Macro cracks	-	Placement
	-	Hardening process

Intrinsic chemical attack

Corrosion of reinforcement

4.5.5. Absorption of Concrete

Absorption is a process in which water will enter into the concrete through its capillary pores due to capillary action. Absorption is directly dependent on the moisture gradient, capillary pore size and pore interconnectivity of concrete. Structures subject to cyclic wet and dry are affected by absorption. Figure 4.4 shows the factors affecting the absorption of concrete.

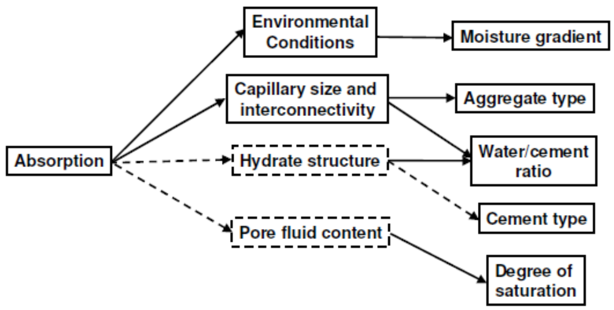


Figure 4.4. Factors affecting absorption of concrete

4.5.6. Diffusion of concrete

Action of concentration gradient of vapour, gas or ions would lead them to pass through concrete mass, which is diffusion. Beside concentration gradients, other factors such as capillary pore size, pore interconnection, degree of reactivity of concrete substrate are important factors to influence diffusion. Figure 4.5 shows the factors affecting the diffusion of concrete. Figure 4.6 shows the primary transport mechanism of various exposure zone of a concrete off-shore structure.

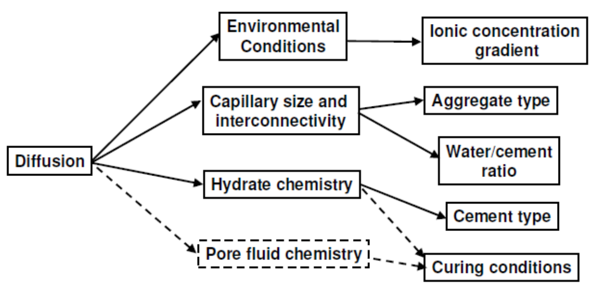


Figure 4.5. Factors affecting diffusion of concrete

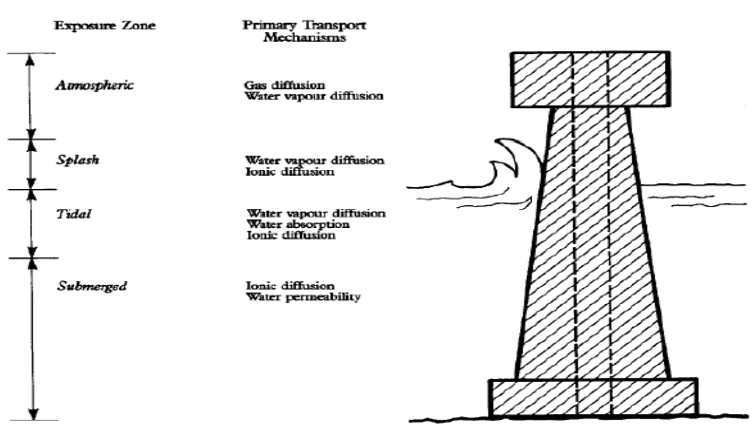


Figure 4.6. Primary transport mechanism of various exposure zone of a concrete off-shore structure

4.6. Factors adversely affecting the durability of concrete

Concrete is vulnerable to the environmental conditions it is exposed to in terms of chemical, physical and biological degradation. It is necessary to understand these factors by the concrete designers and specifiers in order to combat the adverse effect throughout the life cycle of structure.

A list of different category of the adverse factors affecting durability of concrete has been given below.

1. Physical process in concrete

a. Cracking

b. Freeze thaw attack

c. Abrasion, erosion and cavitations

d. Heat or fire damage

2. Chemical process in concrete

a. Chemical attack

b. Acid attack

c. Sulphate attack

d. Alkali attack

3. Biological process in concrete

4. Corrosion of reinforcement

a. Corrosion due to chloride ions

b. Corrosion due to carbonation

4.7. TESTING OF CONCRETE

4.7.1. Quality control tests

Two simple tests are used to control the quality of concrete: SLUMP TEST is used when the concrete is in the plastic state. COMPRESSION TEST is used when concrete is in the hardened state. Both tests are used for the quality control of concrete during manufacture. The compression test can also be used to test a structure, which has been in service for some time by drilling a core from the structure and testing it in compression.

4.7.2. Slump tests

Slump test assesses the consistency or workability of concrete. The acceptance of a load of concrete may depend on the results of a slump test. The first step in testing is to take a test sample from the batch of concrete within 20 minutes of the concrete arriving on site. Normally a visual inspection is also undertaken prior to placing, to estimate the slump and ensure consistency of the concrete. The sample is taken in one of two ways:

- Sampling after 0.2 m^3 of the load has been poured (the most common method), or
- Sampling from three places in the load, at equal intervals and equal portions, during the discharge.

The tools required to carry out the test are: standard slump cone (100 mm × 200 mm × 300 mm), small scoop, bullet-nosed tamping rod (600 mm × 16 mm), ruler, float and slump plate (500 mm × 500 mm). The test is performed by (a) cleaning and moistening the cone, and (b) placing it on the flat slump plate. Fill the cone one-third full with concrete and rod the layer exactly 25 times making sure that the whole area is rodded uniformly. Rodding means pushing a steel rod in and out of the concrete to compact it into a slump cone or a cylinder mould. Always rod in a definite pattern, working from outside into the middle. The cone must be held firmly by standing on the foot lugs while the concrete is being added during rodding. After rodding the first layer fill the cone with a second layer until two-thirds full and rod this layer uniformly 25 times just into the top of the first layer. Then fill the cone until it slightly overflows and rod this top layer 25 times uniformly just into the top of the second layer. The excess concrete is removed from the top with a straight edge so that the cone is exactly filled and the spilled concrete removed from around the bottom of the cone. The cone is then lifted straight up very slowly. Without disturbing the concrete further turn the cone upside down and place the rod across the up-turned cone. Measure the distance from the rod to the top of the slumped concrete. If the top of the slump is irregular, do not measure the high point or the low point. Try to get the average. If the slump is too high or too low compared to the specification, another must be taken. If this fails the remainder of the batch should be rejected.

4.7.3. Compression test

Compression test determines the strength of concrete under standard conditions. Concrete cylinders or concrete cubes are used for the compression test. The concrete test samples, whether cylinders or cubes, are made on site and tested in a laboratory with a compression test machine. Moulding the test sample should be completed within 20 minutes of obtaining the sample. The compressive strength of the test samples determines the acceptability of the concrete represented.

First clean the cylinder mould and slump plate and coat the inside of the mould and plate with a thin film of mineral oil to prevent adhesion of the concrete. For the 100 mm × 200 mm cylinders the mould is then filled to one-half and the concrete compacted by rodding with the tamping rod 25 times. The strokes should be uniformly distributed over the crosssectional area. The mould is then overfilled and compacted by rodding 25 times into the top of the first layer. If after compaction the top is not completely filled, add more concrete and work into the concrete surface. Each mould is tapped all around with a rubber mallet to remove air bubbles and assist compaction of the concrete. If the 150 mm × 300 mm cylinder mould are used the concrete is compacted in three equal layers instead of two. Then (a) the top of the concrete is leveled off with the tamping rod and any concrete around the mould is cleaned, (b) the surface of the concrete is smoothed with a wooden float, (c) the cylinders are capped, (d) the moulds identified with a code number and left in a cool dry place to set undisturbed for at least 24 h. The mould is then removed and the concrete cylinder marked and sent to the laboratory where it is cured for a specified period prior to testing in compression. All moulds are cleaned and oiled after use to prevent rusting. The curing period depends on the specification, although seven days and 28 days are commonly used.

4.8. DAMAGE ASSESSMENT

The assessment of damage in an RC structure and the selection of repair methods require a detailed investigation to determine the extent and causes of damage. To prevent serious failure caused by corrosion and others, it is essential to detect the damage level in existing buildings and infrastructures. Initially, visual observation is made to examine the structural defects. Visual observation uses direct inspection to detect obvious signs of deterioration, such as physical damage in the form of spalling or cracking.

4.8.1. Visual Observation

Visual testing is probably the most important of all non-destructive tests. It can often provide valuable information to the well trained eye. Visual features may be related to workmanship, structural serviceability, and material deterioration and it is particularly important that the engineer is able to differentiate between the various signs of distress which may be encountered. These include for instance, cracks, pop-outs, spalling, disintegration, colour change, weathering, staining, surface blemishes and lack of uniformity. Extensive information can be gathered from visual inspection to give a preliminary indication of the condition of the structure and allow formulation of a subsequent testing programme. The visual inspection however should not be confined only to the structure being investigated. It should also include neighbouring structures, the surrounding environment and the climatic condition. This is probably the most difficult aspect of the whole structural investigation or any diagnostic works since what appears obvious to one may not be so to another.

Tools and equipment for visual inspection

An engineer carrying out a visual survey should be well equipped with tools to facilitate the inspection. These involve a host of common accessories such as measuring tapes or rulers, markers, thermometers, anemometers and others. Binoculars, telescopes, borescopes and endoscopes or the more expensive fibre scopes may be useful where access is difficult. A crack width microscope or a crack width gauge is useful, while a magnifying glass or portable microscope is handy for close up examination. A good camera with the necessary zoom and micro lenses and other accessories, such as polarized filters, facilitates pictorial documentation of defects, and a portable colour chart is helpful in identifying variation in the colour of the concrete. A complete set of relevant drawings showing plan views, elevations and typical structural details allows recording of observations to be made.

General procedure of visual inspection

Before any visual test can be made, the engineer must peruse all relevant structural drawings, plans and elevations to become familiar with the structure. Available documents must also be examined and these include technical specification, past reports of tests or inspection made, construction records, details of materials used, methods and dates of construction, etc. The survey should be carried out systematically and cover the defects present, the current and past use of the structure, the condition of adjacent structures and environmental condition. All defects must be identified, the degree classified, similar to those used for fire damaged concrete and, where possible, the causes identified. The distribution and extent of defects need to be clearly recognized. For example whether the defects are random or appear in a specific pattern and whether the defect is confined to certain locations of members or is present all over the structure. Visual comparison of similar members is particularly valuable as a preliminary to testing to determine the extent of the problems in such cases. A study of similar structures or other structures in the local area constructed with similar materials can also be helpful in providing 'case study' evidence, particularly if those other structures vary in age from the one under investigation. There is a need to identify associated or accompanying defects, especially which particular defect predominates.

Segregation or excessive bleeding at shutter joints may reflect problems with the concrete mix, as might plastic shrinkage cracking, whereas honeycombing may be an indication of a low standard of construction workmanship. Lack of structural adequacy may show itself by excessive deflection or flexural cracking and this may frequently be the reason for an *in situ* assessment of a structure. Long term creep defections, thermal movements or structural movements may cause distortion of doorframes, cracking of windows, or cracking of a structure or its finishes. Material deterioration is often indicated by surface cracking and spalling of the concrete and examination of crack patterns may provide a preliminary indication of the cause. Systematic crack mapping is a valuable diagnostic exercise when determining the causes and progression of deterioration. Observation of concrete surface texture and colour variations may be a useful guide to uniformity. Colour change is a widely recognized indicator of the extent of fire damage.

Visual inspection is not confined to the surface but may also include examination of bearings, expansion joints, drainage channels and similar features of a structure. Any misuse of the structure can be identified when compared to the original designed purpose of the structure. An assessment may also need to be made of the particular environmental conditions to which each part of the structure has been exposed. In particular the wetting and drying frequency and temperature variation that an element is subjected to should be recorded because these factors influence various mechanisms of deterioration in concrete. For example, in marine structures it is important to identify the splash zone. Settlement of surrounding soil or geotechnical failures need to be recorded. Account must also be taken of climatic and other external environmental factors at the location, since factors such as freeze thaw conditions may be of considerable importance when assessing the causes of deterioration.

A careful and detailed record of all observations should be made as the inspection proceeds. Drawings can be marked, coloured or shaded to indicate the local severity of each feature. Defects that commonly need recording include:

- cracking which can vary widely in nature and style depending on the causative mechanism
- surface pitting and spalling
- surface staining
- differential movements or displacements
- variation in algal or vegetative growths
- surface voids
- honeycombing
- bleed marks
- constructional and lift joints
- exudation of efflorescence.

4.8.2. Sketches of typical defects found by visual inspection

Although experience is the best trainer, the following Figures.4.7–4.27 are sketches of typical defects found in concrete structures.

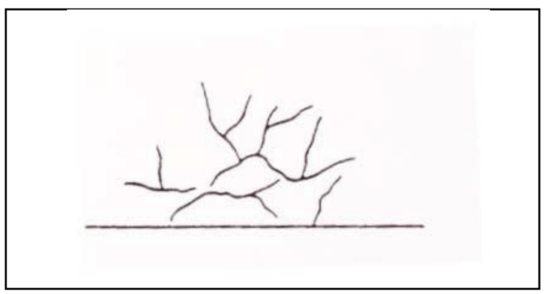


Figure 4.7. Sketch of surface appearance when concrete has been mixed for too long or the time of transport has been too long

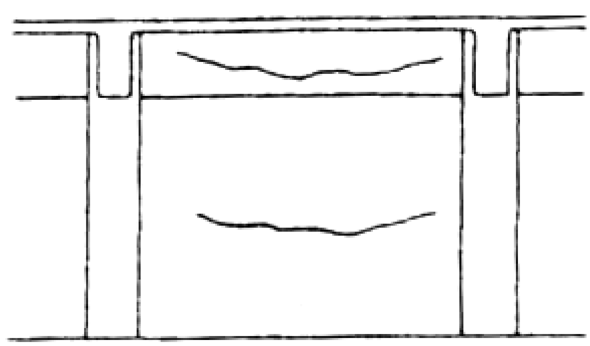


Figure 4.8. Sketch of crack due to concrete settling

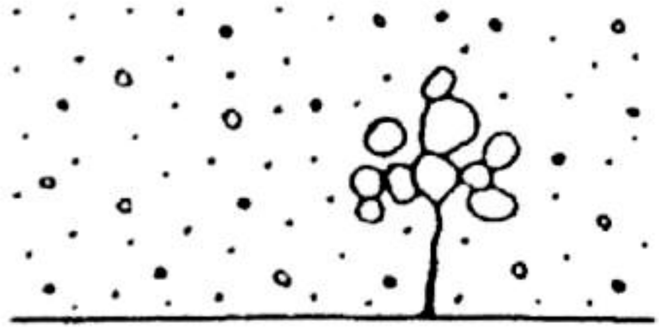


Figure 4.9. Sketch of exposed aggregate

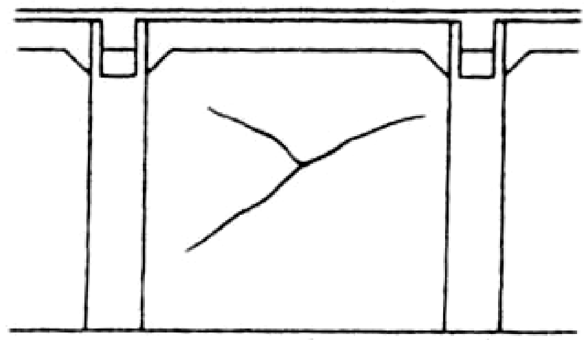


Figure 4.10. Unsuitable process at construction joint

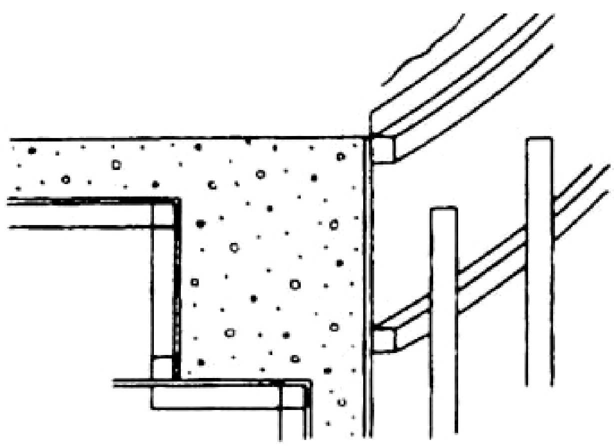


Figure 4.11. Sketch of cracking due to bowing of formwork

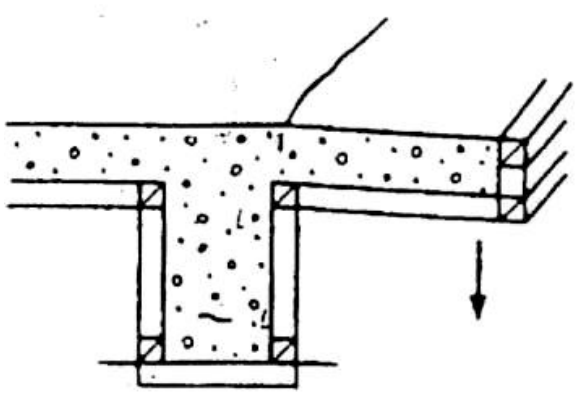


Figure 4.12. Sketch of cracking due to sinking of timbering

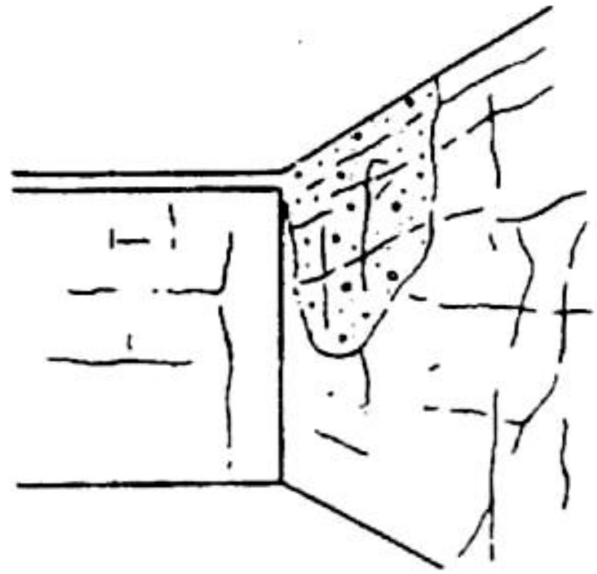


Figure 4.13. Sketch of severe rusting of reinforcing bars due to chemical action

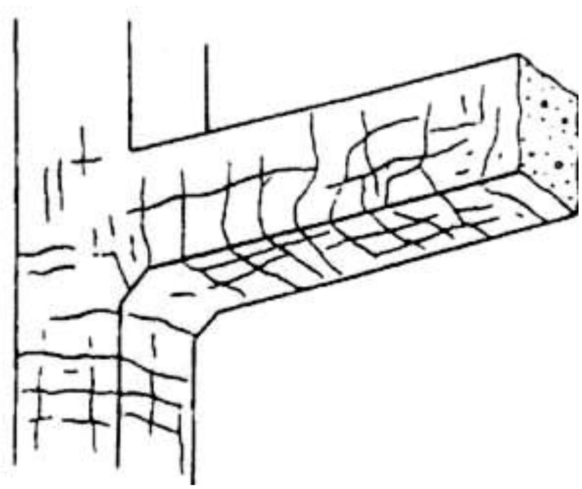


Figure 4.14. Sketch of effect of fire on concrete

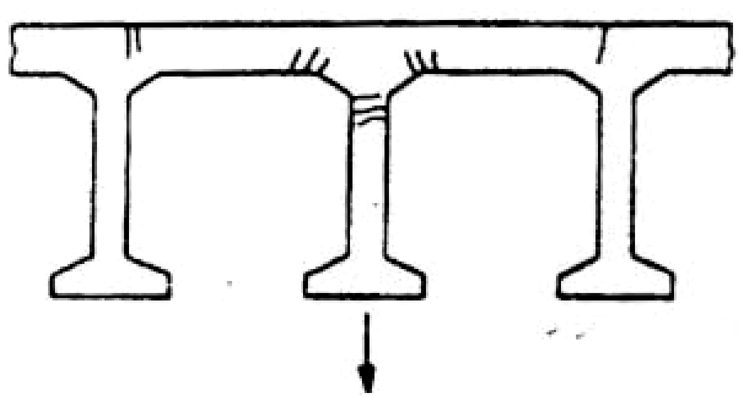


Figure 4.15. Cracks due to differential settlement of central column

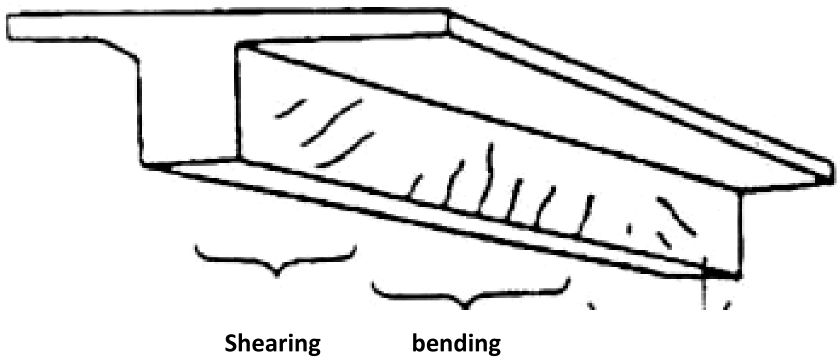


Figure 4.16. Cracks due to bending and shear stresses

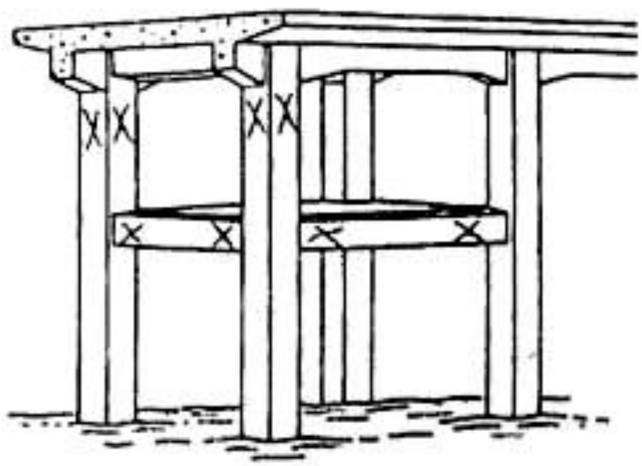


Figure 4.17. Cracking in columns and beams due to an earthquake

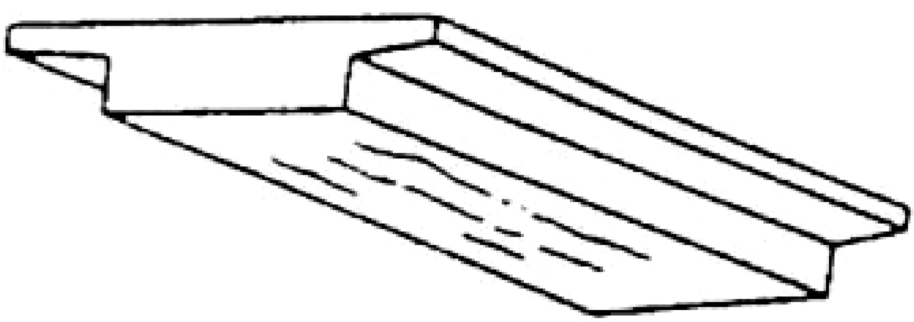


Figure 4.18. Cracks due to insufficient reinforcing bars

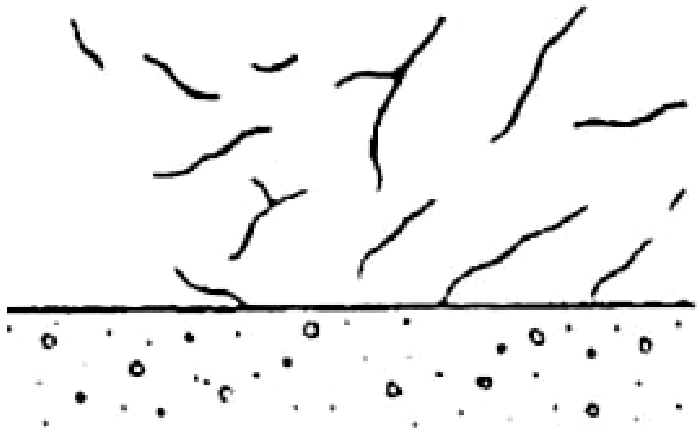


Figure 4.19. Cracks due to abnormal set of cement

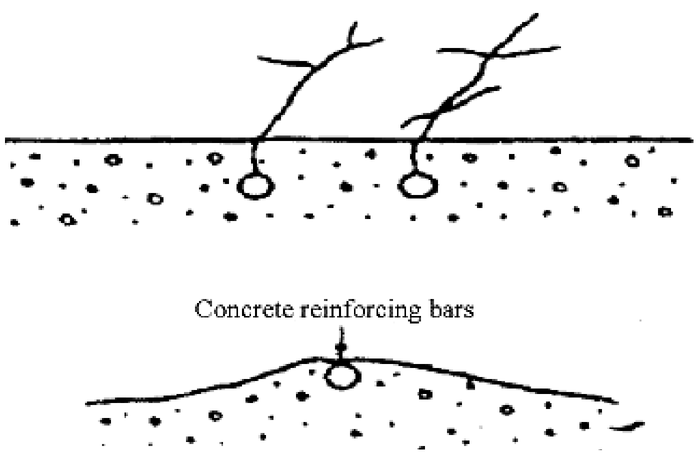


Figure 4.20. Sinking of concrete

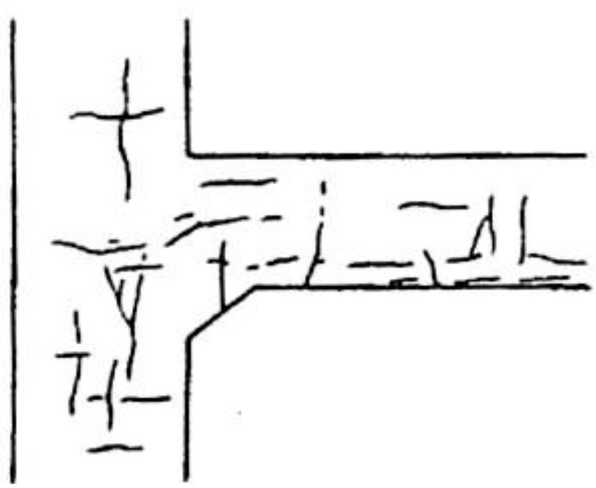


Figure 4.21. Rusting of reinforcing bars

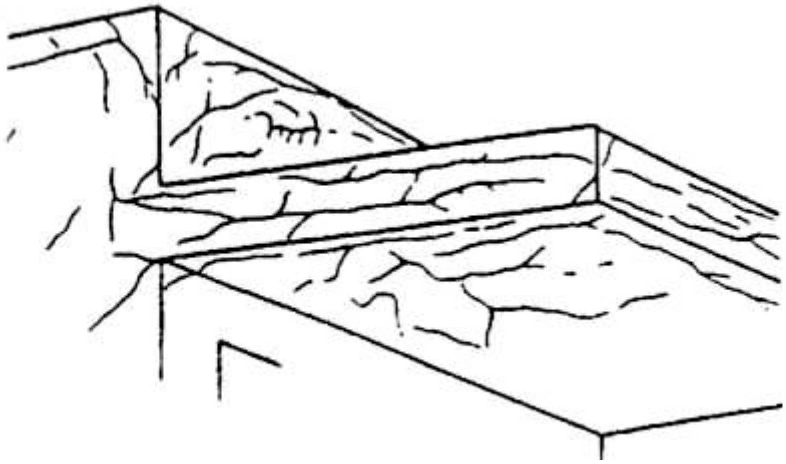


Figure 4.22. Effect of heating and freezing cycles

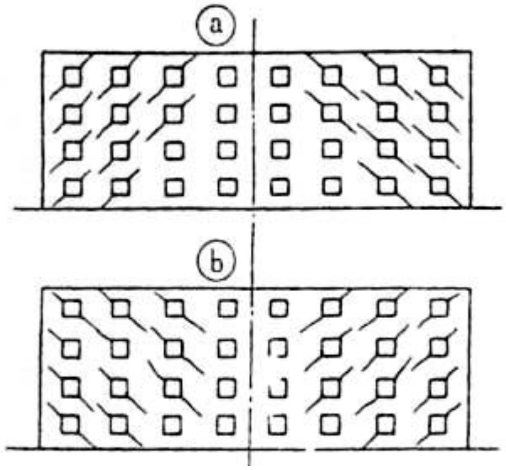


Figure 4.23. Effect of changing ground conditions:

low temperature or b) dryness

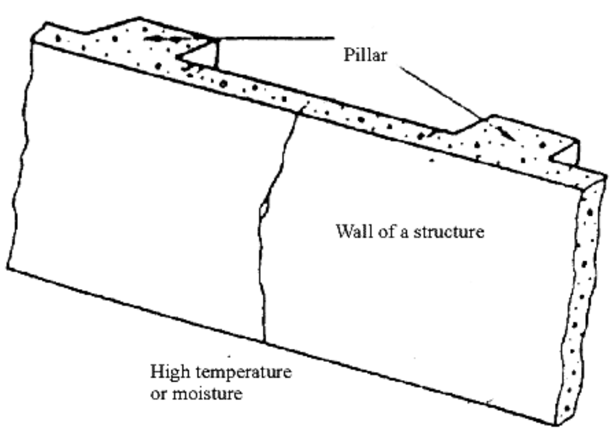


Figure 4.24. Effect of atmospheric conditions

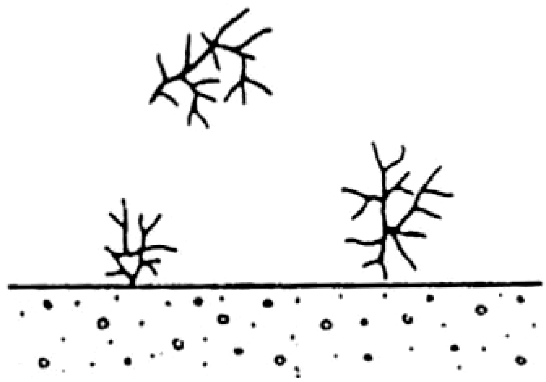


Figure 4.25. Non-uniformity of admixture

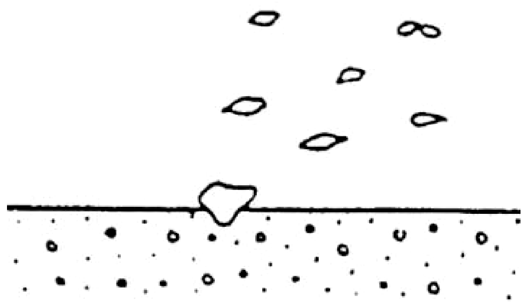


Figure 4.26. Pop-out due to reactive aggregate and high humidity

4.8.4. Half-cell potential survey

Half-cell potential measurements can be used to determine the probability of corrosion activity taking place at the time of the reading. Usually, the corrosion potential of the rebar is measured using a grid pattern over the concrete surface. According to the test method of ASTM C876, if potentials measured using $Cu/CuSO_4$ electrode over an area are more positive than –0.2V, there is a 90% probability that no corrosion is taking place in that area. If potential readings are more negative than –0.35V, there is a 90% probability that active corrosion is taking place. Potentials measured between –0.2V and –0.35V indicate a possible breakdown of passivity at the steel surface and the possibility of future corrosion activity. But this method is often inconclusive because the measurements depend on moisture level, the amount of carbonation, and salt concentration.

General procedure for Half cell potential method

Measurements are made in either a grid or random pattern. The spacing between measurements is generally chosen such that adjacent readings are less than 150 mV with the minimum spacing so that there is at least 100 mV between readings. An area with greater than 150 mV indicates an area of high corrosion activity. A direct electrical connection is made to the reinforcing steel with a compression clamp or by brazing or welding a protruding rod. To get a low electrical resistance connection, the rod should be scraped or brushed before connecting it to the reinforcing bar. It may be necessary to drill into the concrete to expose a reinforcing bar. The bar is connected to the positive terminal of the voltmeter. One end of the lead wire is connected to the half-cell and the other end to the negative terminal of the voltmeter. Under some circumstances the concrete surface has to be pre-wetted with a wetting agent. This is necessary if the half-cell reading fluctuates with time when it is placed in contact with the concrete. If fluctuation occurs either the whole concrete surface is made wet with the wetting agent or only the spots where the half-cell is to be placed. The electrical half-cell potentials are recorded to the nearest 0.01 V correcting for temperature if the temperature is outside the range 22.2 ± 5.5oC. The test setup of Half-cell potential measurement is shown in Figure 4.27.

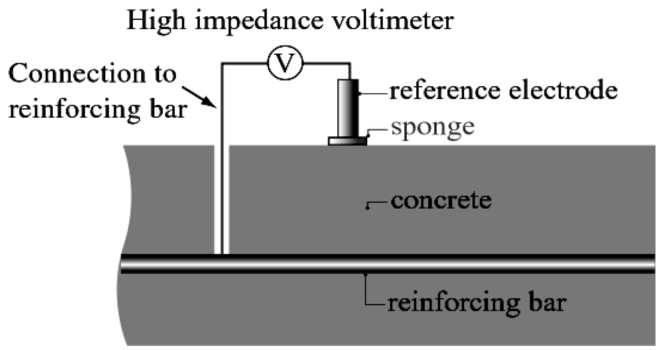


Figure 4.27. Half-cell potential setup

Measurements can be presented either with a equipotential contour map which provides a graphical delineation of areas in the member where corrosion activity may be occurring or with a cumulative frequency diagram which provides an indication of the magnitude of affected area of the concrete member.

Equipotential contour map

On a suitably scaled plan view of the member the locations of the half-cell potential values are plotted and contours of equal potential drawn through the points of equal or interpolated equal values. The maximum contour interval should be 0.10 V. An example is shown in Figure 4.28.

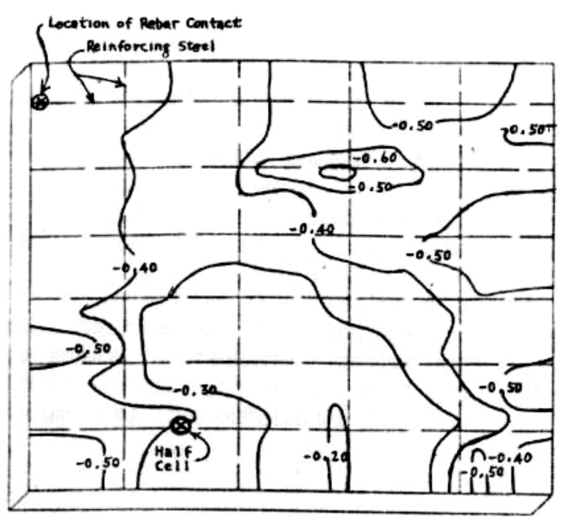


Figure 4.28. Equipotential contour map

Applications

This technique is most likely to be used for assessment of the durability of reinforced concrete members where reinforcement corrosion is suspected. Reported uses include the location of areas of high reinforcement corrosion risk in marine structures, bridge decks and abutments. Used in conjunction with other tests, it has been found helpful when investigating concrete contaminated by salts.

Range and limitations

The method has the advantage of being simple with equipment also simple. This allows an almost non-destructive survey to be made to produce isopotential contour maps of thesurface of the concrete member. Zones of varying degrees of corrosion risk may be identified from these maps. The limitation of the method is that the method cannot indicate the actual corrosion rate. It may require to drill a small hole to enable electrical contact with the reinforcement in the member under examination, and surface preparation may also be required. It is important to recognize that the use and interpretation of the results obtained from the test require an experienced operator who will be aware of other limitations such as the effect of protective or decorative coatings applied to the concrete.

4.8.4. Four-probe resistivity test

Four-probe resistivity test is used to identify the quality of concrete up to a shorter depth. In this test, a known current is applied between two outer probes and the voltage drop between the inner two elements is read off allowing for direct evaluation of resistance. The guidelines of resistivity values based on areas having probable corrosion risk can be identified in concrete structures as given in Table 4.1.

Table 4.1. Corrosion risk from resistivity

Resistivity (ohm cm)	Corrosion probability
Greater than 20,000	Negligible
10,000 – 20,000	Low
5,000 – 10,000	High
Less than 5,000	Very high

Fundamental Principles

There are many techniques used to assess the corrosion risk or activity of steel in concrete. The most commonly used is the half cell potential measurement that determines the risk of corrosion activity. Whilst the half cell potential measurement is effective in locating regions of corrosion activity, it provides no indication of the rate of corrosion. However, a low resistance path between anodic and cathodic sites would normally be associated with a high rate of corrosion than a high resistance path. Such resistivity measurements determine the current levels flowing between anodic and cathodic portions, or the concrete conductivity over the test area, and are usually used in conjunction with the half-cell potential technique. This is an electrolytic process as a consequence of ionic movement in the aqueous pore solution of the concrete matrix. An alternative technique to estimate the rate of corrosion, which is becoming increasingly popular, is the linear polarization resistance.

Equipment

Although other commercial devices like the less accurate two probe system are also available, the Wenner four probe technique is generally adopted for resistivity measurement of *in situ* concrete. The technique was first used by geologists to investigate soil strata. The technique can be used to determine resistivities quickly and with little or no damage to the concrete structures under study, Figure 4.29.

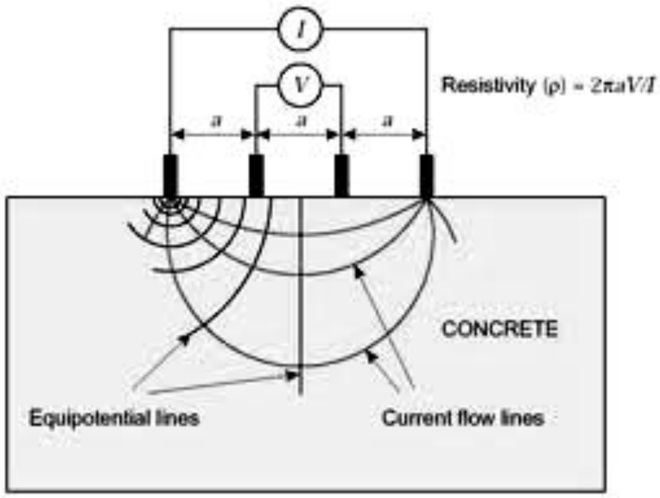


Figure 4.29. Four-probe resistivity

The equipment consists of four electrodes (two outer current probes and two inner voltage probes) which are placed in a straight line on or just below the concrete surface at equal spacings. A low frequency alternating electrical current is passed between the two outer electrodes whilst the voltage drop between the inner electrodes is measured. The apparent resistivity (ρ) in “ohm-cm” may be expressed as:

$$\rho = \frac{2\pi a V}{I}$$

V is voltage drop,

I is applied current,

a is electrode spacing.

The calculation assumes the concrete to be homogeneous and the inhomogeneity caused by the reinforcement network must be allowed for by properly placing the probes to minimize its effect.

General Procedure

Resistivity measurement is a fast, simple and cheap *in situ* non-destructive method to obtain information related to the corrosion hazard of embedded reinforcement. The spacing of the four probes determines the regions of concrete being measured. It is generally accepted that for practical purposes, the depth of the concrete zone affecting the measurement will be equal to the electrode spacing.

If the spacing is too small, the presence or absence of individual aggregate particles, usually having a very high resistivity, will lead to a high degree of scatter in the measurement. Using a larger spacing may lead to inaccuracies due to the current field being constricted by the edges of the structure being studied. In addition, increased error can also be caused by the influence of the embedded steel when larger spacings are employed. A spacing of 50 mm is commonly adopted, gives a very small degree of scatter and allows concrete sections in excess of 200 mm thick to be measured with acceptable accuracy.

The efficiency of surface coupling is also important. In order to establish satisfactory electrical contact between the probes and the concrete, limited damage to the concrete surface sometimes can not be avoided. In some commercial devices, wetting or conductive gel is applied when the probes are pushed against the concrete surface to get better contact. Prewetting of the surface before measurement is also advised. Small shallow holes may also be drilled into the concrete which are filled with a conductive gel. The probes are then dipped into each hole. However, this procedure is not practical for site use.

Applications

The ability of corrosion currents to flow through the concrete can be assessed in terms of the electrolytic resistivity of the material. This resistivity can determine the rate of corrosion once reinforcement is no longer passive. The presence of ions such as chloride will also have an effect. At high resistivity, the rate of corrosion can be very low even if the steel is not passive. For example, reinforcement in carbonated concrete in an internal environment may not cause cracking or spalling due to the very low corrosion currents flowing. The electrical resistivity of concrete is known to be influenced by many factors including moisture, salt content, temperature, water/cement ratio and mix proportions. In particular, the variations of moisture condition have a major influence on in situ test readings. Fortunately, in practice, the moisture content of external concrete does not vary sufficiently to significantly affect the results. Nevertheless, precautions need to be taken when comparing results of saturated concrete, e.g. those exposed to sea water or measurements taken after rain showers, with those obtained on protected concrete surfaces. Another important influence is the ambient temperature. Concrete has electrolytic properties; hence, resistivity will increase as temperature decreases. This is particularly critical when measurements are taken during the different seasons, with markedly higher readings during the winter period than the summer period.

4.8.5. Impact-echo/resonance frequency/stress wave test

A number of non-destructive test methods rely on the effect a structure has on the propagation of stress waves. The most common techniques are pulse-echo, impact-echo, impulse–response and spectral analysis of surface waves. The methods differ in the way that the stress waves are generated and on the signal processing techniques that are used.

Fundamental Principles

This is an effective method of locating large voids or delaminations in plate like structures, e.g. pavements or bridge decks, where the defect is parallel to the test surface. A mechanical impact produces stress waves of 1 to 60 kHz. The wavelengths of from 50 mm to 2000 mm propagate as if in a homogeneous elastic medium.

The mechanical impact on the surface generates compression, shear and surface waves. Internal interfaces or external boundaries reflect the compression and shear waves. When the waves return to the surface where the impact was generated, they can be used to generate displacements in a transducer and subsequently a display on a digital oscilloscope. The resulting voltage-time signal is digitized and transformed, in a computer, to amplitude vs. frequency plot. The dominant frequencies appear as peaks on the frequency spectrum. The dominant frequency is not necessarily the thickness signal. Using each of the frequencies identified as peaks on the frequency spectrum, the distances to the reflecting surfaces can be calculated from

$$d = \frac{V}{2f}$$

where

d is distance,

f is dominant frequency,

V is velocity of compression waves in the test material.

If the receiver is placed close to the impact point the reflected signals may not be seen because the transducer is still ringing due to the impact. The type of impact used has a significant influence on the success of the test. The shorter the contact time, the higher the range of frequencies contained in the pulse. An estimate of the maximum frequency excited is the inverse of the contact time:

$$f_{\max} = \frac{1}{tD},$$

where

tD is the contact time,

fmax is the maximum frequency.

Sansalone and Street gave an estimate of the maximum frequency for a steel ball bearing of diameter D:

$$f_{\max}(KHz) = \frac{291}{D} mm$$

Thus the contact time determines the depth of the defect that can be detected by impact echo testing. As the contact time decreases, the frequency increases and the depth of defect, which can be detected, decreases. Also short duration impacts are needed to detect defects close to the surface.

Equipment for Impact-echo testing

Examples of the equipment used for impact-echo testing are the systems developed by Impact-Echo Instruments as illustrated in Figure 4.30. There are two systems offered. Type A Test System comprising a Data Acquisition System, one cylindrical hand held transducer unit, 200 replacement lead disks for the transducer, Ten spherical impactors 3 mm to 19 mm in diameter (used to vary the contact time), one 3.7 m cable and one 7.6 m cable. Type B Test System comprising a Data Acquisition System, two cylindrical hand-held transducer units, 200 replacement lead disks for the transducer, ten spherical impactors 3 mm to 19 mm in diameter, one 3.7 m cable, one 7.6 m cable and a spacer bar to use with the two transducers.

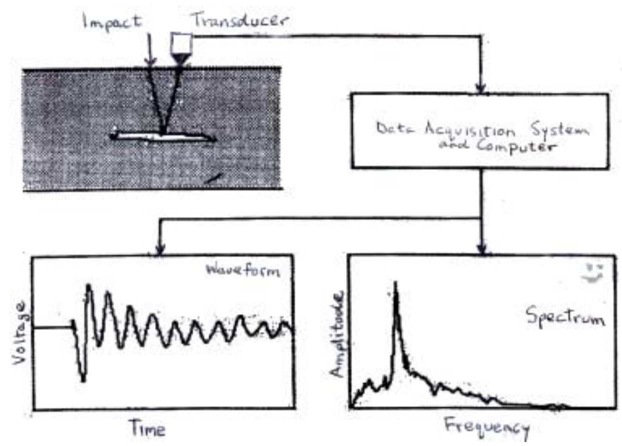


Figure 4.30. Schematic diagram showing how impact-echo works

General procedure for impact-echo testing

Using the Impact–Echo Instruments System A, the technique used is to vary the diameter of the impactor until a clear dominant frequency is obtained. Typically the diameter of the impactor has to increase as the thickness of the material being tested increases to obtain reflections from the rear surface of the material being tested.

Applications of and examples of the use of the impact-echo testing method

The investigation of cracking in the deck of a reinforced concrete railway bridge due to alkali-aggregate reaction resulted in horizontal cracking being detected at mid depth over the entire span. The cracking was verified by taking cores. The bridge was subsequently demolished. Another use has been in measuring the thickness of concrete pavements. The accuracy of the thickness measurement was found to vary depending on the sub base on which concrete is laid. For example the uncertainty of the thickness measurement was within 1% for a concrete pavement on lean concrete sub-base, 2% for pavement on an asphalt sub-base and 3% for pavement on an aggregate sub-base. Voids have also been located in grouted tendon ducts of a post-tensioned highway bridge. Areas of full or partial voids were found in 3 of 14 girders tested.

Delaminations have also been found in 200 mm thick concrete bridge deck with a 100 mm asphalt overlay. Extensive areas of delamination were detected at the top layer of the reinforcing steel. The delamination was confirmed by taking cores. The deck was subsequently repaired and a new asphalt overlay laid. Cracking has also been detected in the beams and columns of parking garage. Cracks were identified at flange to web intersections at certain T beam configurations and the extent of cracking was determined in columns.

Range and limitations of impact-echo testing method

In generic terms the impact-echo method is a commercial development of the wellknown frequency response function method (Frf) and the theory of vibration testing of piles. Further reading may be obtained in Ewens(1984) and Davis and Dunn (1974). The user should beware of the claimed accuracy of detecting defects or thickness in terms of an absolute measurement. It is better to think in terms of a multiple of the wavelength:

Velocity = frequency × wavelength

$$V = f\lambda$$

where, λ is wavelength.

For impact test work, recent research has shown that the “near field” detection capability of impact-echo (Martin, Hardy, Usmani and Forde, 1998) is:

minimum depth of detectable target = $\lambda/2$

Many test houses will deliberately or otherwise use the null hypothesis:

“If a defect is not identified – then none exists.”

In order to determine λ, one could assume the velocity through the good concrete to be:

Velocity = 4,000 m/s.

4.8.6. Rebound / Schmidt hammer Test

Rebound / Schmidt hammer test is conducted to assess the condition of cover concrete particularly to identify the presence of any delamination (Figure 4.31). This test is to be carried out by dividing the member into well-defined grids of spacing 300 mm × 300 mm. The weak surface of concrete can be identified using this rebound numbers and compressive strength of concrete can also be found. The guidelines for qualitative interpretation of rebound hammer test results with reference to corrosion are tabulated in Table 4.2.

Figure 4.31. Rebound / Schmidt hammer

Table 4.2. Quality of concrete cover form rebound numbers

Average rebound number	***Quality of concrete***
> 40	Very good hard layer
30 to 40	Good layer
20 to 30	Fair
< 20	Poor concrete
0	Delaminated

Equipment

The Schmidt rebound hammer is shown in Fig. 4.1. The hammer weighs about 1.8 kg and is suitable for use both in a laboratory and in the field. A schematic cutaway view of the rebound hammer is shown in Fig. 4.2. The main components include the outer body, the plunger, the hammer mass, and the main spring. Other features include a latching mechanism that locks the hammer mass to the plunger rod and a sliding rider to measure the rebound of the hammer mass. The rebound distance is measured on an arbitrary scale marked from 10 to 100. The rebound distance is recorded as a "rebound number" corresponding to the position of the rider on the scale.

General Procedure for Test

The method of using the hammer is explained using Figure 4.32. With the hammer pushed hard against the concrete, the body is allowed to move away from the concrete until the latch connects the hammer mass to the plunger, Figure 4.32a. The plunger is then held perpendicular to the concrete surface and the body pushed towards the concrete, Figure 4.32b. This movement extends the spring holding the mass to the body. When the maximum extension of the spring is reached, the latch releases and the mass is pulled towards the surface by the spring, Figure 4.32c. The mass hits the shoulder of the plunger rod and rebounds because the rod is pushed hard against the concrete, Figure 4.32d. During rebound the slide indicator travels with the hammer mass and stops at the maximum distance the mass reaches after rebounding. A button on the side of the body is pushed to lock the plunger into the retracted position and the rebound number is read from a scale on the body.

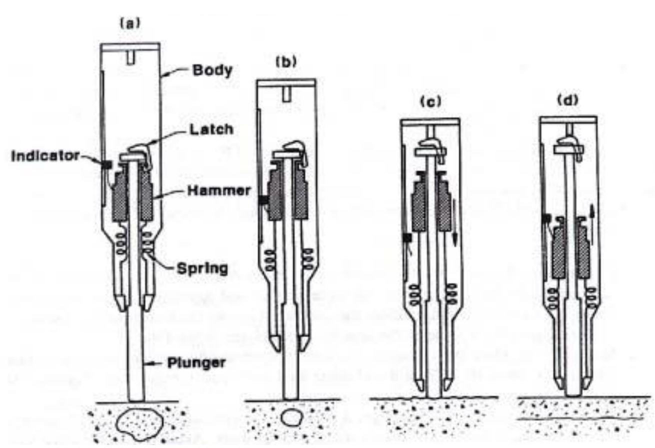


Figure 4.32. A cutaway schematic view of the Schmidt rebound hammer

Range and limitations

Although the rebound hammer does provide a quick, inexpensive method of checking the uniformity of concrete, it has some serious limitations. The results are affected by:

i. Smoothness of the test surface

Hammer has to be used against a smooth surface, preferably a formed one. Open textured concrete cannot therefore be tested. If the surface is rough, e.g. a trowelled surface, it should be rubbed smooth with a carborundum stone.

ii. Size, shape and rigidity of the specimen

If the concrete does not form part of a large mass any movement caused by the impact of the hammer will result in a reduction in the rebound number. In such cases the member has to be rigidly held or backed up by a heavy mass.

iii. Age of the specimen

For equal strengths, higher rebound numbers are obtained with a 7 day old concrete than with a 28 day old. Therefore, when old concrete is to be tested in a structure a direct correlation is necessary between the rebound numbers and compressive strengths of cores taken from the structure. Rebound testing should not be carried out on low strength concrete at early ages or when the concrete strength is less than 7 MPa since the concrete surface could be damaged by the hammer.

iv. Surface and internal moisture conditions of concrete

The rebound numbers are lower for well-cured air dried specimens than for the same specimens tested after being soaked in water and tested in the saturated surface dried conditions. Therefore, whenever the actual moisture condition of the field concrete or specimen is unknown, the surface should be pre-saturated for several hours before testing. A correlation curve for tests performed on saturated surface dried specimens should then be used to estimate the compressive strength.

v. Type of coarse aggregate

Even though the same aggregate type is used in the concrete mix, the correlation curves can be different if the source of the aggregate is different.

vi. Type of cement

High alumina cement can have a compressive strength 100% higher than the strength estimated using a correlation curve based on ordinary Portland cement. Also, super sulphated cement concrete can have strength 50% lower than ordinary Portland cement.

vii. Carbonation of the concrete surface

In older concrete the carbonation depth can be several millimeters thick and, in extreme cases, up to 20 mm thick. In such cases the rebound numbers can be up to 50% higher than those obtained on an uncarbonated concrete surface.

viii. Cement content

ix. Concrete compaction

x. Age, rate of hardening and curing type

4.8.7. Ultrasonic Pulse Velocity (UPV) test

Ultrasonic Pulse Velocity (UPV) test is used to assess the homogeneity and integrity of concrete. It consists of transmitting the ultrasonic pulse of 50 – 54 kHz frequency through a concrete medium and receiving at the other end(Figure 4.33). The time of travel of ultrasonic pulses is measured and the pulse velocity is calculated by dividing the thickness of concrete member, which is the length of travel, by measured time. The guidelines for qualitative assessment of concrete based on UPV test results are shown in Table 4.3. These tests can identify the quality at a particular point (local) in the member.

Figure 4.33. Ultrasonic Pulse Velocity Test setup

Table 4.3. Quality of concrete based on UPV

Velocity	***Concrete quality***
> 4.0 km/s	Very good to excellent
3.5 to 4.0 km/s	Good to very good, slight porosity may exist
3.0 to 3.5 km/s	Satisfactory but loss of integrity is suspected
< 3.0 km/s	Poor and loss of integrity exists

Fundamental principle

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete from the transducer using a liquid coupling material such as grease or cellulose paste, it undergoes multiple reflections at the boundaries of the different material phases within the concrete. A complex system of stress waves develops, which include both longitudinal and shear waves, and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by a second transducer. Electronic timing circuits enable the transit time T of the pulse to be measured. Longitudinal pulse velocity (in km/s or m/s) is given by:

$V = \frac{T}{L}$ where

V is the longitudinal pulse velocity,

L is the path length,

T is the time taken by the pulse to traverse that length.

Equipment for pulse velocity test

The equipment consists essentially of an electrical pulse generator, a pair of transducers, an amplifier and an electronic timing device for measuring the time interval between the initiation of a pulse generated at the transmitting transducer and its arrival at the receiving transducer. Two forms of electronic timing apparatus and display are available, one of which uses a cathode ray tube on which the received pulse is displayed in relation to a suitable time scale, the other uses an interval timer with a direct reading digital display.

Applications

Measurement of the velocity of ultrasonic pulses of longitudinal vibrations passing through concrete may be used for the following applications:

- determination of the uniformity of concrete in and between members measurement of changes occurring with time in the properties of concrete
- determination of areas of deteriorated concrete and detection of cracks
- correlation of pulse velocity and strength as a measure of concrete quality.
- determination of the modulus of elasticity and dynamic Poisson's ratio of the concrete.

Determination of pulse velocity

1. Transducer arrangement

The receiving transducer detects the arrival of that component of the pulse, which arrives earliest. This is generally the leading edge of the longitudinal vibration. Although the direction in which the maximum energy is propagated is at right angles to the face of the transmitting transducer, it is possible to detect pulses, which have travelled through the concrete in some other direction. It is possible, therefore, to make measurements of pulse velocity by placing the two transducers on either:

- opposite faces (direct transmission)
- adjacent faces (semi-direct transmission): or
- the same face (indirect or surface transmission)

These three arrangements are shown in Figures 4.34(a), 4.34(b) and 4.34(c).

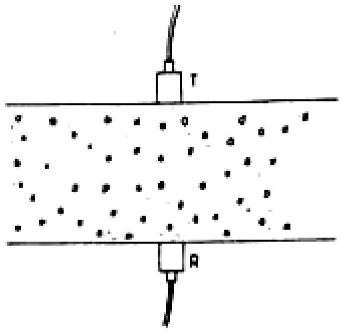


Figure 4.34 (a): Direct transmission

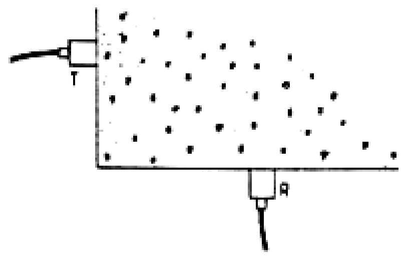
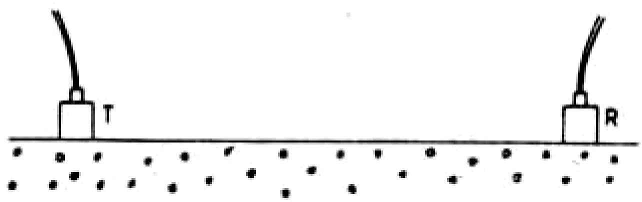


Figure 4.34 (b): Semi-direct transmission



T: Transmitter, R: Receiver

Figure 4.34(c): Indirect or surface transmission

Fig. 4.34(a) shows the transducers directly opposite to each other on opposite faces of the concrete. However, it is sometimes necessary to place the transducers on opposite faces but not directly opposite each other. Such an arrangement is regarded as semi-direct transmission, Fig 4.34(b).

2. Determination of pulse velocity by direct transmission

Where possible the direct transmission arrangement should be used since the transfer of energy between transducers is at its maximum and the accuracy of velocity determination is therefore governed principally by the accuracy of the path length measurement. The couplant used should be spread as thinly as possible to avoid any end effects resulting from the different velocities in couplant and concrete.

3. Determination of pulse velocity by semi-direct transmission

The semi-direct transmission arrangement has a sensitivity intermediate between those of the other two arrangements and, although there may be some reduction in the accuracy of measurement of the path length, it is generally found to be sufficiently accurate to take this as the distance measured from centre to centre of the transducer faces. This arrangement is otherwise similar to direct transmission.

Determination of pulse velocity by indirect or surface transmission

Indirect transmission should be used when only one face of the concrete is accessible, when the depth of a surface crack is to be determined or when the quality of the surface concrete relative to the overall quality is of interest. It is the least sensitive of the arrangements and, for a given path length, produces at the receiving transducer a signal which has an amplitude of only about 2% or 3% of that produced by direct transmission. Furthermore, this arrangement gives pulse velocity measurements which are usually influenced by the concrete near the surface. This region is often of different composition from that of the concrete within the body of a unit and the test results may be unrepresentative of that concrete. The indirect velocity is invariably lower than the direct velocity on the same concrete element. This difference may vary from 5% to 20% depending largely on the quality of the concrete under test. Where practicable site measurements should be made to determine this difference. With indirect transmission there is some uncertainty regarding the exact length of the transmission path because of the significant size of the areas of contact between the transducers and the concrete. It is therefore preferable to make a series of measurements with the transducers at different distances apart to eliminate this uncertainty. To do this, the transmitting transducer should be placed in contact with the concrete surface at a fixed point x and the receiving transducer should be placed at fixed increments xn along a chosen line on the surface. The transmission times recorded should be plotted as points on a graph showing their relation to the distance separating the transducers. An example of such a plot is shown as line (b) in Figure 11.2. The slope of the best straight line drawn through the points should be measured and recorded as the mean pulse velocity along the chosen line on the concrete surface. Where the points measured and recorded in this way indicate a discontinuity, it is likely that a surface crack or surface layer of inferior quality is present and a velocity measured in such an instance is unreliable.

Factors influencing pulse velocity measurements

1. Moisture content

The moisture content has two effects on the pulse velocity, one chemical the other physical. These effects are important in the production of correlations for the estimation of concrete strength. Between a properly cured standard cube and a structural element made from the same concrete, there may be a significant pulse velocity difference. Much of the difference is accounted for by the effect of different curing conditions on the hydration of the cement while some of the difference is due to the presence of free water in the voids. It is important that these effects are carefully considered when estimating strength.

2. Temperature of the concrete

Variations of the concrete temperature between 10°C and 30°C have been found to cause no significant change without the occurrence of corresponding changes in the strength or elastic properties. Corrections to pulse velocity measurements should be made only for temperatures outside this range.

3. Path length

The path length over which the pulse velocity is measured should be long enough not to be significantly influenced by the heterogeneous nature of the concrete. It is recommended that, the minimum path length should be 100 mm for concrete where nominal maximum size of aggregate is 20 mm or less and 150 mm for concrete where nominal maximum size of aggregate is between 20 mm and 40 mm. The pulse velocity is not generally influenced by changes in path length, although the electronic timing apparatus may indicate a tendency for velocity to reduce slightly with increasing path length. This is because the higher frequency components of the pulse are attenuated more than the lower frequency components and the shape of the onset of the pulse becomes more rounded with increased distance traveled. Thus, the apparent reduction of pulse velocity arises from the difficulty of defining exactly the onset of the pulse and this depends on the particular method used for its definition. This apparent reduction in velocity is usually small and well within the tolerance of time measurement accuracy for the equipment.

4. Shape and size of specimen

The velocity of short pulses of vibration is independent of the size and shape of the specimen in which they travel, unless its least lateral dimension is less than a certain minimum value. Below this value, the pulse velocity may be reduced appreciably. The extent of this reduction depends mainly on the ratio of the wavelength of the pulse vibrations to the least lateral dimension of the specimen but it is insignificant if the ratio is less than unity. If the minimum lateral dimension is less than the wavelength or if the indirect transmission arrangement is used, the mode of propagation changes and therefore the measured velocity will be different. This is particularly important in cases where concrete elements of significantly different sizes are being compared.

5. Effect of reinforcing bars

The pulse velocity measured in reinforced concrete in the vicinity of reinforcing bars is usually higher than in plain concrete of the same composition. This is because the pulse velocity in steel may be up to twice the velocity in plain concrete and, under certain conditions, the first pulse to arrive at the receiving transducer travels partly in concrete and partly in steel. The apparent increase in pulse velocity depends on the proximity of the measurements to the reinforcing bar, the diameter and number of bars and their orientation with respect to the propagation path. The frequency of the pulse and surface conditions of the bar may both also affect the degree to which the steel influences the velocity measurements. Corrections to measured values to allow for reinforcement will reduce the accuracy of estimated pulse velocity in the concrete so that, wherever possible, measurements should be made in such a way that steel does not lie in or close to the direct path between the transducers.

6. Determination of concrete uniformity

Heterogeneities in the concrete within or between members cause variations in pulse velocity, which in turn are related to variations in quality. Measurements of pulse velocity provide a means of studying the homogeneity and for this purpose a system of measuring points which covers uniformly the appropriate volume of concrete in the structure has to be chosen.

4.8.8. Acoustic emission Test

Acoustic emission technique is an effective method for early detection of rebars corrosion in concrete. When corrosion products are formed on a corroding rebar, they swell and apply pressure to the surrounding concrete. Mircocracks will be formed and stress waves will be generated during the expansion process when the pressure is high enough to break the interface layer. The growth of the microcracks is directly related to the amount of corrosion product of a corroding rebar. The degree of the corrosion can be interpreted by detecting the acoustic emission event rate and their amplitude.

Acoustic Emissions (AE) are microseismic activities originating from within the test specimen when subjected to an external load (Figure 4.35). Acoustic emissions are caused by local disturbances such as microcracking, dislocation movement, intergranular friction, etc. An acoustic signal travels to a number of piezoelectric transducers, which convert the acoustic signals (mechanical waveforms) to electric signals. A digital oscilloscope captures the electric signals. The time of arrival of the signal at each transducer depends on the distance of the transducer from the AE source. The source, frequency and amplitude of the AE events have been used to quantify the nature of microfracture in various materials. AE sources are determined by calculating the difference in time taken for the wave to arrive at the different transducers. The velocity of the waves in the specimen is determined using the ultrasonic pulse velocity method.

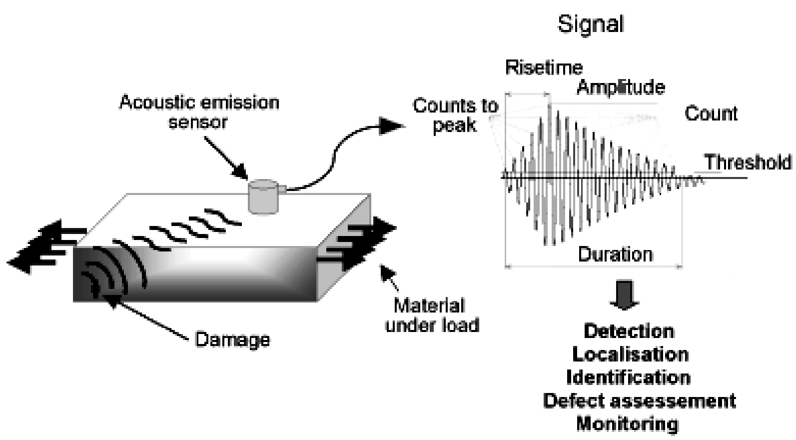


Figure 4.35. Acoustic emission correlated with the presence of rebar corrosion

Some of the NDT equipments including Rebound hammer, UPV tester, and acoustic emission device have been used to identify the localized quality of concrete in a structure. The surface hardness obtained using these tests will give the strength of concrete after making extensive calibration of the equipment. The strength of concrete is related to modulus of elasticity, durability, resistance to environmental attack, etc. But the strength obtained by conducting the above tests are local in nature and hence the quantities like load carrying capacity derived out of these properties are unreliable unless exhaustive measurements at close intervals are conducted over the entire structure.

4.8.9. CARBONATION DEPTH MEASUREMENT TEST

Fundamental Principle

Carbonation of concrete occurs when the carbon dioxide, in the atmosphere in the presence of moisture, reacts with hydrated cement minerals to produce carbonates, e.g. calcium carbonate. The carbonation process is also called depassivation. Carbonation penetrates below the exposed surface of concrete extremely slowly. The time required for carbonation can be estimated knowing the concrete grade and using the following equation:

$$t = (d / k)^2$$

where

t is the time for carbonation,

d is the concrete cover,

k is the permeability.

Typical permeability values are shown in Table 4.4.

Table 4.4. Permeability Values Versus Concrete Grade

Concrete Grade	***Permeability***
15	17
25	6
35	4
20	10
30	5
40	3.5

The significance of carbonation is that the usual protection of the reinforcing steel generally present in concrete due to the alkaline con ditions caused by hydrated cement paste is neutralized by carbonation. Thus, if the entire concrete cover over the reinforcing steel is carbonated, corrosion of the steel would occur if moisture and oxygen could reach the steel.

Equipment

If there is a need to physically measure the extent of carbonation it can be determined easily by spraying a freshly exposed surface of the concrete with a 1% phenolphthalein solution. The calcium hydroxide is coloured pink while the carbonated portion is uncoloured.

General Procedure

The 1% phenolthalein solution is made by dissolving 1gm of phenolthalein in 90 cc of ethanol. The solution is then made up to 100 cc by adding distilled water. On freshly extracted cores the core is sprayed with phenolphthalein solution, the depth of the uncoloured layer (the carbonated layer) from the external surface is measured to the nearest mm at 4 or 8 positions, and the average taken. If the test is to be done in a drilled hole, the dust is first removed from the hole using an air brush and again the depth of the uncoloured layer measured at 4 or 8 positions and the average taken. If the concrete still retains its alkaline characteristic, the colour of the concrete will change to purple. If carbonation has taken place the pH will have changed to 7 (i.e. neutral condition) and there will be no colour change. Another formula, which can be used to estimate the depth of carbonation, utilizes the age of the building, the water-to-cement ratio and a constant, which varies depending on the surface coating on the concrete.

$$y = \frac{7.2}{R^2(6.4x - 1.76)^2} \times C^2$$

where

y is age of building in years,

x is water-to-cement ratio,

C is carbonation depth,

R is a constant (R= αβ).

R varies depending on the surface coating on the concrete (β) and whether the concrete has been in external or internal service (α). This formula is contained in the Japanese Construction Ministry publication “Engineering for improving the durability of reinforced concrete structures.” α is 1.7 for indoor concrete and 1.0 for outdoor concrete. β values are shown in Table 4.5.

TABLE 4.5. Values of β

Finished condition	**Indoor**	**Outdoor**
no layer	1.7	1.0
plaster	0.79	
mortar + plaster	0.41	
mortar	0.29	0.28
mortar + paint	0.15	
tiles	0.21	0.07
paint	0.57	0.8

The carbonation depth is therefore given by:

$$C = \frac{Y^{1/2} R(4.6x - 1.76)}{(7.2)^{1/2}}$$

The phenolphthalein test is a simple and cheap method of determining the depth of carbonation in concrete and provides information on the risk of reinforcement corrosion taking place. The only limitation is the minor amount of damage done to the concrete surface by drilling or coring.

4.8.10. Windsor Probe Test- penetration resistance test

The Windsor Probe test measures the compressive strength of concrete accurately and effectively, on-site in the field. The Windsor Probe system rapidly and accurately determines the concrete compressive strength of a structure by driving a probe into the concrete with a known amount of force. Improved and enhanced over thirty years, this modern system is capable of measuring concrete with a maximum compressive strength of 17,000 PSI (110MPA). An electronic measuring unit has been added to help ensure proper test results which can be recorded for later review or uploading to a personal computer (Figure 4.36). Two probe styles are available: one for lightweight, low density concrete with air filled aggregate and the other probe for more standard mix designs. Also, two standard power settings facilitate testing fresh concrete as well as mature mixes.

Figure 4.36. Windsor Probe Apparatus

Equally accurate results are obtained on horizontal or vertical surfaces provided that the probe is perpendicular or at right angles to the test surface. A hardened steel alloy probe is propelled at high speed by an exactly measured explosive charge into the concrete and its penetration measured. Each power load is guaranteed to have an energy level to give an exit muzzle velocity tolerance within ±3%. The compressive strength of the concrete is directly related to the resistance to penetration of the crushed aggregate and cement matrix: this is determined by the distance required to absorb the specific amount of kinetic energy of the probe. The compressive strength of the concrete is empirically related to the penetration that varies with the hardness of the aggregate. This relationship is recognized by determining the Moh’s scale of hardness of the aggregate and applying a correction factor to the penetration.

For most accurate test results ASTM recommends so that a correlation be developed for the particular mix design being tested. Exact duplication of cylinder test results should not be expected. The probes measure the strength of the actual concrete in a structure rather than that of a sample compacted and cured under strict and somewhat artificial conditions which do not necessarily represent those of the structure itself.

Applications:

- Form Removal
- Structural Analysis
- Light-weight concrete strength determination
- Standard concrete strength determination
- High-strength concrete strength determination
- High-precision determination

Advanatages and Limitations

The advantages are:

- ❖ The test is relatively quick and the result is achieved immediately provided an appropriate correlation curve is available.
- ❖ The probe is simple to operate, requires little maintenance except cleaning the barrel and
- ❖ is not sensitive to operator technique.
- ❖ Access is only needed to one surface.
- ❖ The correlation with concrete strength is affected by a relatively small number of variables.
- ❖ The test result is likely to represent the concrete at a depth of from 25 mm to 75 mm from the surface rather than just the property of the surface layer as in the Schmidt rebound test.

The limitations are:

- ❖ The minimum acceptable distance from a test location to any edges of the concrete member or between two test locations is of the order of 150 mm to 200 mm.
- ❖ The minimum thickness of the member, which can be tested, is about three times the expected depth of probe penetration.
- ❖ The distance from reinforcement can also have an effect on the depth of probe penetration especially when the distance is less than about 100 mm.
- ❖ The test is limited to <40 MPa and if two different powder levels are used in an investigation to accommodate a larger range of concrete strengths, the correlation procedure becomes complicated.
- ❖ The test leaves an 8 mm hole in the concrete where the probe penetrated and, in older concrete, the area around the point of penetration is heavily fractured.
- ❖ On an exposed face the probes have to be removed and the damaged area repaired.

4.8.11. Cover thickness survey

A cover meter is an instrument to locate rebar and measure the exact concrete cover (Figure 4.37). Rebar detectors are less sophisticated devices that can only locate metallic objects below the surface. Due to the cost-effective design, the pulse-induction method is one of the most commonly used solutions. The pulse-induction method is based on electromagnetic pulse induction technology to detect rebars. Coils in the probe are periodically charged by current pulses and thus generate a magnetic field. On the surface of any electrically conductive material which is in the magnetic field eddy currents are produced. They induce a magnetic field in opposite directions. The resulting change in voltage can be utilized for the measurement. Rebars that are closer to the probe or of larger size produce a stronger magnetic field.

Advantages of this method are high accuracy, not influenced by moisture and inhomogeneities of the concrete, unaffected by environmental influences and its low costs. Disadvantage of this method are limited detection range and minimum bar spacing depends on cover depths.

Figure 4.37. Cover Meter

4.8.12. Chloride Testing

Chloride can be present in the concrete either as added calcium chloride used to accelerate the setting time of the concrete or as a contaminant of the aggregate. Post-construction contamination can come from chloride salts, used for de-icing, that have dissolved in water and subsequently permeated the concrete or from a marine environment. Chloride salts, in the presence of moisture, can cause the accelerated corrosion of the reinforcing steel within the concrete, resulting in an expansive rusting reaction of the reinforcement and subsequent spalling of the concrete. The aim of the chloride testing is to establish the level of chlorides within the concrete structure, either as an overall level or, via the taking of incremental samples, as a profile through the depth of the structure. Chloride testing in conjunction with a visual survey and a survey to establish depth of carbonation and cover to reinforcement can give an indication of the condition of the structural concrete within a building and the potential for future performance.

Testing for Chlorides in Concrete

Chloride content of concrete can be determined by collecting from core samples. The test consists of powdering the sample, obtaining the water extracts and conducting standard titration experiment for determining the water soluble chloride content which is expressed by weight of concrete or by weight of cement if the mix ratio is known. Rapid Chloride Test Kit is available and test is performed by drilling and collecting samples from different depths, mixing the sample with a special chloride extraction liquid, and measuring the electrical potential of the liquid by chloride ion sensitive electrode. With the help of a calibration chart relating electrical potential and chloride content, the chloride content of the sample can be directly determined. Table 4.6 gives a qualitative guidelines for identification of corrosion prone locations based on pH values and chloride content.

Table 4.6. Guidelines for identification of corrosion prone locations based on chemical test

Sl.No.	Corrosion prone	Corrosion prone
1	High pH values greater than 11.5 and very low chloride content	No corrosion
2	High pH values and high chloride content greater than threshold values (0.4% - 0.6% by weight of cement)	Corrosion prone
3	Low pH values and high chloride content greater than 0.4% - 0.6% by weight of cement)	Corrosion prone

4.8.13. Concrete's Ability to Resist Chloride Ion Penetration (Rapid Chloride Permeability Test)

Mechanisms of Chloride Ion Transport

Capillary absorption, hydrostatic pressure, and diffusion are the means by which chloride ions can penetrate concrete. The most familiar method is diffusion, the movement of chloride ions under a concentration gradient. For this to occur, the concrete must have a continuous liquid phase and there must be a chloride ion concentration gradient.

A second mechanism for chloride ingress is permeation, driven by pressure gradients. If there is an applied hydraulic head on one face of the concrete and chlorides are present, they may permeate into the concrete. A situation where a hydraulic head is maintained on a highway structure is rare, however. A more common transport method is absorption. As a concrete surface is exposed to the environment, it will undergo wetting and drying cycles. When water (possibly containing chlorides) encounters a dry surface, it will be drawn into the pore structure though capillary suction. Absorption is driven by moisture gradients. Typically, the depth of drying is small, however, and this transport mechanism will not, by itself, bring chlorides to the level of the reinforcing steel unless the concrete is of extremely poor quality and the reinforcing steel is shallow. It does serve to quickly bring chlorides to some depth in the concrete and reduce the distance that they must diffuse to reach the rebar.

Of the three transport mechanisms described above that can bring chlorides into the concrete to the level of the rebar, the principal method is that of diffusion. It is rare for a significant hydraulic head to be exerted on the structure, and the effect of absorption is typically limited to a shallow cover region. In the bulk of the concrete, the pores remain saturated and chloride ion movement is controlled by concentration gradients.

Test

In the ASTM C1202 test, a water-saturated, 50-mm thick, 100-mm diameter concrete specimen is subjected to a 60 V applied DC voltage for 6 hours using the apparatus shown in Figure 38. In one reservoir is a 3.0 % NaCl solution and in the other reservoir is a 0.3 M NaOH solution. The total charge passed is determined and this is used to rate the concrete according to the criteria included as Table 4.7. This test, originally developed by Whiting [1981], is commonly referred to as the "Rapid Chloride Permeability Test" (RCPT) Figure 4.38. This name is inaccurate as it is not the permeability that is being measured but ionic movement. In addition, the movement of *all* ions, not just chloride ions, affects the test result (the total charge passed). There have been a number of criticisms of this technique, although this test has been adopted as a standard test, is widely used. The main criticisms are: (i) the current passed is related to all ions in the pore solution not just chloride ions, (ii) the measurements are made before steady-state migration is achieved, and (iii) the high voltage applied leads to an increase in temperature, especially for low quality concretes, which further increases the charge passed. Lower quality concretes heat more as the temperature rise is related to the product of the current and the voltage. The lower the quality of concrete, the greater the current at a given voltage and thus the greater heat energy produced. This heating leads to a further increase in the charge passed, over what would be experienced if the temperature remained constant. Thus, poor quality concrete looks even worse than it would otherwise.

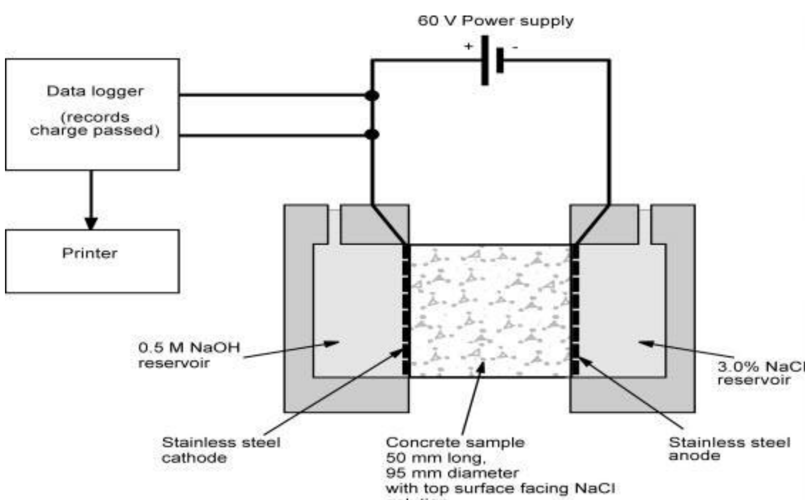


Figure 4.38. RCPT test setup

Table 4.7. RCPT ratings (per ASTM C1202)

Charge Passed (coulombs)	Chloride Ion Penetrability
> 4,000	High
2,000-4,000	Moderate
1,000-2,000	Low
100-1,000	Very Low
< 100	Negligible

Core Sampling and testing

Condition of concrete in the structure after construction can be assessed directly by core drilling at required location or at suspected location. The core samples obtained are then taken for visual inspection and compression test to determine maximum compressive strength. The core drilling and compression testing shall be done in accordance with ASTM Method C 42-87. The extracted cores can be subjected to a series of tests and serve multiple functions such as:

- confirming the findings of the non-destructive test
- identifying the presence of deleterious matter in the concrete
- ascertaining the strength of the concrete for design purposes
- predicting the potential durability of the concrete
- confirming the mix composition of the concrete for dispute resolution
- determining specific properties of the concrete not attainable by non-destructive methods such as intrinsic permeability.

Equipments and Apparatus

1. Core drilling machine, rotary drilling type
2. Diamond coring bit, single-tube core barrel type
3. Compression machine

Applications

The core samples can also be used for the following:

1. Standard and density determination
2. Depth of carbonation of concrete
3. Chemical analysis
4. Water/gas permeability
5. Petrographic analysis
6. Chloride permeability test

Precautions observed in core drilling:

1. Test a minimum of 3 cores for each section n of questionable concrete
2. Obtain 85mm minimum diameter cores. Obtain larger cores for concrete with 25mm size aggregate.
3. Try to obtain a length at least 1 ½ times the diameter (L/D ratio)
4. Trim to remove steel provided the minimum 1 ½ L / D ratio can be maintained
5. Trim ends square with an automatic feed diamond saw
6. When testing, keep can thickness under 3mm
7. Use high strength capping material; neoprene pad caps should not be used.
8. Check planeness of caps and bearing blocks
9. Do not drill cores from the top layers of columns, slabs, walls, or footings, which will be 10 to 20% weaker than cores from the mid or lower portions
10. Test cores after drying for 7 days if the structure is dry in service; otherwise soak cores 40 hours prior to testing.

7.13.4. Significance and Use

This test method provides standardized procedures for obtaining and testing specimens to determine the compressive, splitting tensile, and flexural strength of in-place concrete. Generally, test specimens are obtained when doubt exists about the in-place concrete quality due either to low strength test results during construction or signs of distress in the structure. Another use of this method is to provide strength information on older structures. Concrete strength is affected by the location of the concrete in a structural element, with the concrete at the bottom tending to be stronger than the concrete at the top. Core strength is also affected by core orientation relative to the horizontal plane of the concrete as placed, with strength tending to be lower when measured parallel to the horizontal plane. These factors shall be considered in planning the locations for obtaining concrete samples and in comparing strength test results. The strength of concrete measured by tests of cores is affected by the amount and distribution of moisture in the specimen at the time of test. There is no standard procedure to condition a specimen that will ensure that, at the time of test, it will be in the identical moisture condition as concrete in the structure. The moisture conditioning procedures in this test method are intended to provide reproducible moisture conditions that minimize within-laboratory and between-laboratory variations and to reduce the effects of moisture introduced during specimen preparation.

There is no universal relationship between the compressive strength of a core and the corresponding compressive strength of standard-cured moulded cylinders. The relationship is affected by many factors such as the strength level of the concrete, the in-place temperature and moisture history, and the strength gain characteristics of the concrete. Historically, it has been assumed that core strengths are generally 85 % of the corresponding standard-cured cylinder strengths, but this is not applicable to all situations. ACI 318 provides core strength acceptance criteria for new construction.

4.8.15. Pullout test

There are two options for the pullout test:

- DANISH LOK TEST which requires that the head be cast into the concrete at the time of construction. This test gives a good indication of near surface compressive strength.
- Building Research Establishment, UK (BRE) PULLOUT involves drilling a hole and inserting a “fixing” which is pulled out. The advantage of this test is that it does not require a head to be cast into the concrete during construction. The disadvantage is that the test really measures tensile strength and is then calibrated to compressive strength.

The pullout test is a test that falls in the transition area between a destructive test and a non-destructive test. It is destructive in the sense that a relatively large volume of the concrete is damaged but non-destructive because the damaged can be repaired. The pullout test measures the force required to pull an embedded metal insert with an enlarged head from a concrete specimen or a structure. The insert is pulled by a loading ram seated on a bearing ring that is concentric with the insert shaft. The bearing ring transmits the reaction force to the concrete. Frustum geometry is controlled by the inner diameter of the bearing ring (D), the diameter of the insert head (d), and the embedment depth (h). The apex angle (2α) of the idealized frustum is given by:

2α 2 $\tan^{-1}$[(D-d) / 2h]

The pullout test is widely used during construction to estimate the in-place strength of concrete to help decide whether critical activities such as form removal, application of post tensioning, or termination of cold weather protection can proceed. Since the compressive strength is usually required to evaluate structural safety, the ultimate pullout load measured during the in-place test is converted to an equivalent compressive strength by means of a previously established correlation relationship.

As the insert is pulled out, a conical shaped fragment of concrete is extracted from the concrete mass. The idealized shape of the extracted conic frustum is shown in Figure 4.39.

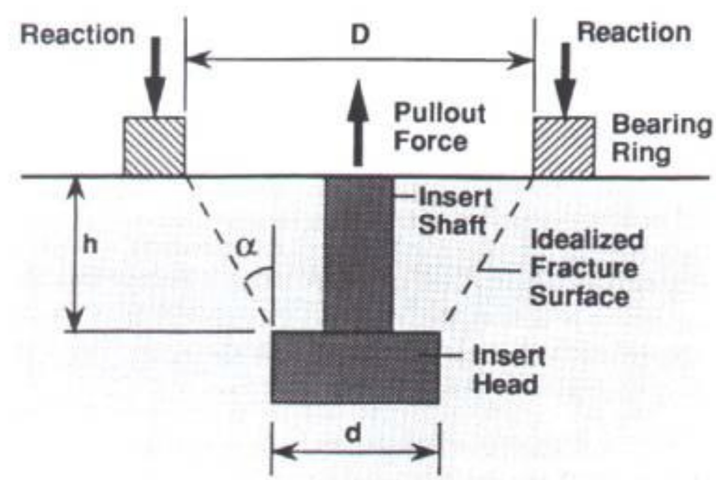


Figure 4.39. Schematic of the pullout test

Unlike some other tests that used to estimate the in-place strength of concrete, the pullout test subjects the concrete to a slowly applied load and measures an actual strength property of the concrete. However, the concrete is subjected to a complex three dimensional state of stress, and the pullout strength is not likely to be related simply to uniaxial strength properties. Nevertheless, by use of correlation curves the pullout test can be used to make reliable estimates of in-place strength. An important step in implementing the method is choosing the locations and number of pullout tests in a given placement of concrete. The inserts should be located in the most critical portions of the structure and there should be a sufficient number of tests to provide statistically significant results. Additional inserts are recommended in the event that testing begins too soon, and the concrete has not attained the required strength. The use of maturity meters along with the pullout tests is encouraged to assist in selecting the correct testing times and in interpreting possible low strength results. The BRE pullout test was developed to permit testing in an existing construction by drilling a hole and inserting some type of expansion anchor. The results of these tests are difficult to interpret if a correlation curve does not exist for the concrete used in the construction.

4.8.16. Pulloff test

This test involves attaching a plate to the concrete using epoxy resin and, after curing has taken place, measuring the force required to pull the plate off. This test scars the concrete but gives a measure of the near surface tensile strength which can be converted to the compressive strength provided a correlation exists between the compressive strength and tensile strength for the concrete mix being investigated.

4.8.17. Strength Tests- a Comparative Assessment

Test Method	***Cost***	***Speed of Operation***	***Damage to Concerete***	***Representativeness***	***Reliability of Strength Prediction***
Cores	Moderate to High	Slow	Moderate	Good	Good
Rebound Hammer	Very Low	Fast	Nil	Surface only	Poor to Fair
Ultrasonic	Low	Fast	Nil	Good to Moderate	Fair toGood
Pullout	Moderate	Fast	Minor	Near surface	Moderate
Breakoff/Pulloff	Moderate	Slow	Moderate	Moderate to Good	Moderate
Lok Test	Moderate	Moderate to Fast	Moderate	Moderate	Good to Moderate
Capo Test	Moderate	Slow	Moderate	Moderate to Good	Good to Moderate
Penetrateion	Moderate	Fast	Minor	Near Surface	Moderate

4.8.18. Damage assessment by vibration technique

A state of damage can be detected by a reduction in dynamic stiffness and an increase in damping. This damage might be localized, as in a crack, or distributed through the bulk of specimen as in many microcracks. Changes of stiffness lead to changes in the natural frequencies of the structure. The measurement of dynamic characteristics such as natural frequencies and damping of a structure is potentially a very attractive method of non-destructive testing, since these properties can be measured at one point of the structure and are independent of the position chosen (Adams et al., 1975).

The results obtained from dynamic tests at a low level of excitation are mainly natural frequencies, modal damping, and mode shape components. These are used to identify any decrease in stiffness of the structural elements and to get information that is sometimes considered sufficient to verify the occurrence of damage. For many civil and industrial engineering structures, damage to structural element is similar to those produced by a local reduction in stiffness. The monitoring of structures consists of comparing these results of test carried out at various intervals, following the evolution of damage (Capecci and Vestroni, 1999).

Vibration testing of structures provides a potential for a global technique for detecting damages based on vibration characteristics. The other conventional methods of diagnostic evaluation are local methods, as they require one to look directly at locations of suspected faults. A whole structure or a large component of a structure may be tested by imparting vibration causing driving force to the structure at one or more pre - selected points and these need not be the locations of suspected fault (Biswas et al., 1994).

Wu et al. (1992) developed some automatic monitoring methods for detection of structural damage by the use of the self – organization and learning capabilities of neural networks in structural damage assessment. Durgaprasad et al. (1996) studied the applicability of artificial neural networks for damage assessment of structures. Zhao et al. (1998) studied the neural network to locate structural damage for a beam, a frame, and support movements of a beam in its axial direction. Flood et al. (2001) demonstrated the viability of using neural networks to predict deflection in externally reinforced concrete beams. Akkurt et al. (2004) developed a fuzzy logic prediction model for 28 days compressive strength of cement mortar under standard curing conditions.

The existing damage assessment methods are used to investigate the quality and to ascertain the degree of damage in the structures. It is not conclusively concentrated on the residual strength and ultimate load carrying capacity of concrete structures involving damage parameters such as peak load effects, deficiencies in materials and environmental effects.

4.9. Summary

The composition and properties and testing of concrete are discussed in detail. The various demage assessment procedures are explained in detail.

4.10. Keywords

Composition – properties – tests on concrete – durability – damage assessment procedures.

4.11. Intext questions

1. What do you understand the term ‘durability of concrete’? Explain in detail.
2. Explain any one of the test on concrete.
3. Explain any one of the NDT procedure to assess the quality of concrete.

❑

LESSON – 5

METHODS OF SURVEY

Objective

- ❖ To assess the strength of structures, surveys to be conducted are studied.

Contents

5.1. Introduction and fundamental principles
5.2. Methods and inspection techniques required
5.2.1. First Survey
5.2.2. Second Survey
5.3. Carbonation test
5.4. Corrosion of reinforcing bars
5.5. Assessment of cracks
5.6. Assessment of evidence of water leakage
5.7. Deterioration of concrete strength
5.8. Assessment of a large deflection
5.9. Assessment of surface deterioration
5.10. Third Survey
5.10.1. Corrosion of beam
5.10.2. Cracking
5.10.3. Water leckage
5.10.4. Large deflection
5.10.5. Surface deterioration
5.11. Summary
5.12. Keywords
5.13. Intext Question

5.1. INTRODUCTION AND FUNDAMENTAL PRINCIPLES

Civil engineers or building surveyors may be asked to carry out an inspection of a concrete structure to assess its condition. The request may be caused by doubts about the safety of the structure, because of damage to the structure or age of the structure. On other occasions there may be a proposal to carry out alterations to the structure, for instance by making new doorways or windows in a building, or extending the building. In such situations it is necessary to tackle the inspection of the structure in a planned manner. It is usual in such inspections to begin by gathering as much information about the structure as possible and then by visually inspecting the structure. This is followed if considered necessary, by further investigation of any areas of severe deterioration. This investigation focuses on the extent of carbonation of the structure, the extent of corrosion of the reinforcement and theoretical remaining concrete life. The final investigation, again if considered necessary, examines in greater detail the extent of any cracking in the structure, the compressive strength of the concrete, extent of corrosion, etc. Finally an assessment can be made of the condition of the building and whether repair is an option.

5.2. METHODS AND INSPECTION TECHNIQUES REQUIRED

5.2.1. First survey (regular inspection)

It is important to find out as much information as possible about the structure. A typical list of information, which should be gathered as follows:

- date of survey
- name and address of building
- building's use
- date of construction
- no. of floors
- area of each floor
- type of construction
- span between beams
- kind of foundation
- designer
- building supervisor
- builder
- name of maintenance personnel
- environmental conditions (tropical, temperate etc}
- presence of vibration
- presence of chemicals
- presence of air conditioning
- distance from sea
- prevailing wind direction
- side of building closest to the sea
- average wind speed
- history of building use
- extensions or rebuilding carried out
- any repairs necessary
- any accident
- type of concrete used(cement, sand, aggregate, use of admixture)
- design strength
- fabrication method.

Armed with this information the building can then be visually inspected. This may require the use of binoculars to view more inaccessible parts of the building. The visual inspection needs to concentrate on those areas that are most likely to show damage, namely column, beam and floor areas, and particularly those areas where tension occurs. This is the corner areas if the floor is inspected from above and the centre of the floor if the floor is inspected from below. Any cracks identified during this process are recorded. At the end of this assessment one can give a rough estimate of the condition of the building. If cracks, lifting, exfoliation, deteriorated surface and water leaking are found it may be necessary to carry out a second more detailed survey. This decision is usually made by grading the degree of deterioration. The degree of deterioration criteria given is shown in Table-5.1.

5.2.2. Second survey (specific/particular inspection)

A second survey is carried out if the degree of deterioration reaches Grade III. This survey determines the depth of carbonation of the concrete, extent of corrosion of the reinforcing bars, extent of any cracking, severity of water leakage, any deterioration of concrete strength, identification of any areas of excessive deflection and the identification of any areas of surface deterioration.

5.3. Carbonation Test

Using the carbonation test, the depth of carbonation is determined at 4 or 8 points and classified, see Table 5.2, where D is distance from surface to first layer of reinforcing bars and MCD is the measured carbonation depth. As shown in Section 5 theoretical carbonation depth (TCD) can be calculated, as follows:

$$TCD = \frac{Y^{½} R(4.6x - 1.76)}{(7.2)^{½}}$$

A classification of the theoretical depth of carbonation is shown in Table 5.3. The degree of deterioration due to carbonation can then be classified as shown in Table 5.4.

TABLE 5.1. Criteria for Assessment of Degree of Deterioration

Kind of deterioration	Unit for classification	Grade I	Grade II	Grade III
Cracks along main bars	No. of 1 m crack lengths per 100m²	0	1-2	3 and over
Cracks along supplementary bars	No. of 1 m crack lengths per 100m²	0-2	3-4	5 and over
Cracks around openings	Number of cracks for 10 openings	0-2	3-4	5 and over
Mesh cracks	Area of meshed cracks as a %	less than 5%	5-10%	10% and over
Other cracks	No. of 1m crack Lengths/100m²	0-4	5-9	10% and over
Exfoliation - only on finished layer	(Exfoliated area/area of side) %	0%	0-1%	1% and over
No explosion of bars	Number per 100m²	0	0	1% and over
Explosion of bars	Number per 100m²	0	0	1% and over
Deteriorated surface ❖ stain on surface ❖ efflorescence ❖ pop out ❖ weakened surface ❖ other stain	 no. per 100 m² no. per 100 m² no. per 100 m² % of weakened	 0 0 0 less than 1%	 Less than 2	 2 and over 4 and over 1 and over 3% and over 5% and over

	% of weakened	less than 1%		
Water leaking		no	no	yes
Abnormal structural Movement or deflection		no	no	yes

Table 5.2. Classification table for measured carbonation depth

Classification of carbonation	***Outdoors or contact with soil***	***Indoors***
A1	MCD<0.5D	MCD<0.7D
A2	0.5D<MCD>D	0.7D<MCD<D + 20mm
A3	D<MCD	D+20mm<MCD

Table 5.3. Classification of the theoretical depth of carbonation for the purpose of a second survey

Classification Depth of carbonation	***Classification Depth of carbonation***
B1	MCD<0.5TCD
B2	0.5TCD≤MCD<1.5TCD
B3	MCD≥1.5TCD

Table 5.4. Classification of degree of deterioration due to carbonation

Degree of deterioration	***Classification***
I-Minor	A1 and B1, A2 and B1, A1 and B2
II-Mild	A1 and B3, A2 and B2
III-Severe	A2 and B3, A3 and B1, A3 and B2, A3 and B3

5.4. Corrosion of reinforcing bars

An assessment of the extent of corrosion of the reinforcing bars is carried out by selecting a number of representative areas to survey. The intent of the survey is to determine the amount of concrete cover over the bar at the positions selected, establishing the type, diameter and direction of the reinforcing bar, assessing the condition of the reinforcing bar and checking the depth of carbonation and surface condition. The corrosion of the bars can be classified as in Table 5.5.

Table 5.5. Classification of corrosion of the bars

Classification	Points	Condition of bar
I	0	Surface with mill scale, slightly rusted without stain on concrete
II	1	Bar has begun to rust, mill scale has begun to flake and there is some pitting
III	3	Whole bar surface has rusted and flaked
IV	6	There is a reduction of bar area.

Table 6 shows a method of assessing the degree of deterioration based on the points calculated.

Table 5.6. Method of assessing degree of deterioration based on the points calculated

Degree of deterioration	***Points calculated***
I (good)	0 < 1
II (slight)	1< 3
III (medium)	3 < 4.5
IV (serious)	4.5 < 6

Table 5.7 gives guidance on whether the corrosion of the bars needs to be repaired and whether a third survey is required.

Table 5.7. Guidance to determine the need to repair the bars or conduct a third survey

Degree of deterioration	Needs repair	Third survey required
I (good)	No	No
II (slight)	No	If necessary
III (medium)	Yes	If necessary
IV (serious)	Yes	If necessary

5.5. Assessment of cracks

If cracks have been classified as Grade III during the first survey, a second survey is required to determine the pattern, width and depth of the cracks and their cause. A hammer is used to sound the area around the crack. Dense concrete will produce a different sound from concrete containing a void underneath. If the crack is only in the coating on top of the concrete no further assessment is required. However, if the crack runs into the concrete and is not just a surface crack, the crack width is measured. If necessary, evidence of the location of the crack can be recorded on a photograph. The grade of the crack can be assessed using Table 5.8.

Table 5.8. Guide to assess grade of crack

Crack severity	***Crack width in mm Outdoor crack***	***Crack width in mm Indoor crack***
I	<0.05	<0.2
II	0.05~0.5	0.2~1.0
III	>0.5	>1.0

The need to repair the crack or the requirement to proceed to a third survey depends upon the degree of its severity as indicated in Table 5.9.

Table 5.9. Guide to determine need to repair crack and conduct third survey

Degree of deterioration	Characteristics of crack	Need to repair	Need for third survey
I	crack not growing crack growing	no	no yes
II	crack not growing crack growing	yes	no yes
III	crack not growing crack growing	yes	no yes

5.6. Assessment of evidence of water leakage

The second survey of water leakage requires the surface coating to be removed and the area containing the watermark to be measured. The area is then inspected both on a wet day and a fine day to establish the cause of the watermark. The degree of deterioration due to water leakages is classified as sown in Table 5.10. Furthermore, the decision whether repair is required or a third survey is necessary depends on the degree of deterioration as indicated in Table 5.11.

Table 5.10. Classification of deterioration due to water leakage

Degree of deterioration	*Outdoor area (under cover)*	*Inside room (water used in area)*	*Inside room (no water used in area)*	*Exposed outdoor area (no scaffolding requested)*
I (good)	-	-	Mark dried	-
II (slight)	Mark dried	Mark dried	-	Mark dried
III (medium)	Mark wet	Mark wet	Mark wet	Mark wet
IV (serious)	Water leaking	Water leaking	Water leaking	Water leaking

Table 5.11. Guide to determine if water leakage areas need to be repaired and third survey required

Degree of deterioration	*Need for repair*				
	Outd oor area(under cover)	*Inside room (water used in area)*	*Inside room (no water used in area)*	*Exposed outdoor area (no scaffolding requested)*	*Need for third survey*
I (good)	-	-	no	-	No
II (slight)	no	no	-	yes	No
III(medium)	yes	yes	yes	yes	yes
IV (serious)	yes	yes	yes	yes	yes

5.7. Deterioration of concrete strength

NDT is used during the second survey to assess the deterioration of concrete strength. The areas surveyed are those assessed during the first survey as being suspect. For comparison some sound areas are also selected. Three NDT techniques are used:

- rebound hammer test
- pulse velocity measurement
- pullout test.

Rebound Hammer Test

The surface layer on the area to be checked is first removed then the concrete is smoothed using a carborundum whetstone. About 20 points are marked in the test area with a minimum of 25 mm spacing. No point should be marked any closer than 30 mm from a corner and the test area should be greater than 100 mm × 100 mm. The average of all 20 test points is taken and any reading greater than ± 20% of the average is discarded. The test is repeated until all 20 tests are less than ± 20% of the average.

Pulse Velocity Measurement

UPV test is performed to assess the homogeneity and integrity of concrete (Sec.4.8.7).

Pullout Test Procedure

Drill the area to be tested using a 15 mm diameter drill and 35mm depth. Install the plug. The pullout strength and compressive strength of the concrete is estimated using the following formula:

$$F_P = \frac{P}{A}(kg/cm^2)$$

where,

Fp is pullout strength,

P is proof stress of pullout,

A is side area of hole.

Concrete can be graded based on its compressive strength as shown in Table 5.12.

Table 5.12. Grade of concrete based on its compressive strength

Degree of deterioration	% of the design strength
I (no deterioration)	100 and over
II (deteriorated)	75-100
III (severe deterioration)	Less than 75%

Assessment on whether repair is necessary and whether a third survey is necessary is shown in Table 5.13.

Table 5.13. Guide to assess the need for repair and a third survey

Degree of deterioration	*Need for repair*	*Need for third survey*
I	No	Depends on necessity
II	Yes	Yes
III	Yes	Yes

5.8. Assessment of a large deflection

To assess an apparent large deflection it is necessary to quantify the deflection. This is done by measuring the bend in a beam or floor by determining the difference in level between either end of beam (or edge of the floor) and centre of the beam (or floor). This can be done with a surveying instrument (or a string line) and a measuring tape, or a straight edge and a measuring tape. The length of span of the beam or floor slab is also measured. If cracking is present in the beam or slab, the width of crack is measured with a suitable scale. The length of crack is also measured and recorded. If possible the time when cracking was first noticed should be obtained from people familiar with the building. This should be combined with questions about the building’s loading history to determine whether the building has ever been overloaded. Also, the building’s current loading should be estimated to assess whether it exceeds the design parameters. The result of the deflection survey can be classified as shown in Table 5.14.

Table 5.14. Degree of deterioration due to deflection and crack

Degree of deterioration	Deflection/span	Width (mm) and total length of crack (m)
I (good)	Less than 1/300	<0.5mm and <6m
II (slight)	1/300 to less than 1/200	<1.5mm and <15m
III (medium)	1/200 to less than 1/100	<3mm and <20m
IV (serious)	1/100 and over	>3mm and >20m

In using Table 5.14 it is important to note that if a beam is being assessed, the width of the crack is the most important aspect and, if a slab is being assessed, the total length of crack is the most important aspect. In a situation where there is a difference in the assessed degree of deterioration between the rating with deflection/span and either the crack width or the crack length, the deflection/span ratio is the more important parameter. Further course of action depends upon the severity of defect as graded in Table 5.15.

Table 5.15. Guide to determine need for repair and a third survey

Degree of deterioration	***Need for repair***	***Need for third survey***
I (good)	No	No
II (slight)	Yes	No
III (medium)	Yes	Depends on necessity
IV (serious)	Yes	Yes

5.9. Assessment of surface deterioration

The second survey of surface deterioration is carried out by selecting a few representative areas of each kind of deterioration present. These include:

- efflorescence
- stains (water and rust)
- lifting, separation and exfoliation
- rub off
- pop out
- weakness including disintegration.

By visually assessing the selected areas, a record is made of the area, depth and degree of deterioration. A classification of the deterioration is shown in Table 5.16, whereas the need to repair or for a third survey depends on its grade as indicated in Table 5.17.

Table 5.16. Classification of deterioration

Degree of deterioration	***Description***
I (good)	Deterioration is noticeable but it is only a small area and there is no danger of the area falling.
II (medium)	Area of deterioration is large, however, in only some areas is the depth of penetration up to 20 mm.
III (serious)	Loss of cross-sectional area is large with the depth of deterioration reaching the reinforcing bars. Rate of progress of deterioration is estimated to be fast.

Table 5.17. Guide to determine need for repair or third survey

Grade	***Assessment of future progress of deterioration***	***Need for repair***	***Need for third survey***
I	Dormant – will not progress furth	No, except if required to improve appearance	No
I	Will progress further	Yes	Depends on necessity
II	- do -	Yes	- do -
III	- do -	Yes	- do -

5.10. Third survey

A third survey is necessary if the damage of concrete condition is very severe.

5.10.1. Corrosion of bars

If it is decided after the second survey that a third survey of the corrosion of reinforcing bars is necessary a selection is made of areas of normal concrete as well as defective areas containing cracks, construction joints, cold joints, honeycombing, rust staining and exfoliation. Ten positions are selected for each condition or four to six bars in an area of about 500 mm × 500 mm are surveyed. As for the second survey the following are assessed using the same tables for corroded bar: classification of deterioration, presumption of cause, remaining life and need for repair. In addition, salt content of the concrete is measured.

5.10.2. Cracking

Areas are selected containing large growing cracks discovered during the second survey stage. Investigation involved the following:

- ❖ width and growing status of the cracks are checked at intervals of 6 months to 1 year
- ❖ degree of corrosion of the bars in the cracked area is established using the same tables as for the second survey
- ❖ extent of carbonation is determined
- ❖ depth of the crack is established using an ultrasonic technique or by extracting a core from the cracked area.

A core is extracted from an area where Schmidt Hammer tests have been carried out to establish the compressive strength of the concrete. The core is analysed to establish as much information as possible about the concrete used, e.g. aggregate type, mix proportions, degree of compaction, etc. A proof load test is applied on the floor slab and the width of the crack monitored during the test. The structure is checked for differential settlement by checking the floor, windows etc.

5.10.3. Water leakage

If the second survey into water leakage showed that a third survey was necessary or if it was not possible to reach the area where water leakage is occurring without scaffolding, the third survey is carried out. This may require scaffolding to be erected if previous access was not possible. Essentially the third survey requires the design drawings to be checked for the possible source of the water. From a practical point of view it may be possible to detect the source of the water by colouring the water in the possible sources. The concrete in the area of the leakage can also be removed to determine if the reinforcing is being corroded. Once the reason for the leakage is determined the concrete can be repaired.

5.10.4. Large deflection

In this case the third survey is carried out to either

- determine the percentage of residual deflection, or
- determine the ratio of measured to calculated frequency of vibration of the member.

If the percentage of residual deflection is to be measured, the deflection of the member in the existing condition is first measured. The member is then put under tension by loading with for instance a known load. When the load is removed the deflection is again measured and the residual additional deflection established. A core needs to be taken from the member before the load test to determine the compressive strength of the concrete. If the ratio of measured to calculated frequency of vibration of the member is to be established, free vibration wave is measured by generating a shock wave and using a vibrometer to determine frequency. Knowing the dynamic force applied and the dimensions of the member the theoretical frequency of vibration can be calculated. The result of those investigations can be assessed using Table 5.18. Whether the results justify repair or not can be judged using Table 5.19.

Table 5.18. Guide to assess the result of investigations

Degree of deterioration	***Ratio = frequency measured/frequency calculated***	***% of residual deflection***
I	0.90 and over	Less than 15
II	0.75 and over	15 and over
III	Less than 0.75	,,

Table 5.19. Guide to determine if results justify repair

Degree of deterioration	***Need for structural analysis***	***Need for repair***
I	No	No
II	Yes	Yes
III	Yes	Yes

5.10.5. Surface deterioration

If the need for a third survey has been established, additional information is required

- On the weakness of the surface of the concrete
- On any deterioration of the compressive strength of the concrete
- On the depth of carbonation of the concrete
- On the depth of deterioration.

Assessment of the deterioration after further investigation can be obtained using the Table 5.20.

Table 5.20. Guide to assess deterioration after further investigation

Degree of deterioration	***Description***
I (good)	Deterioration is noticeable but it is only a small area and the depth is 10 mm or less.
II (medium)	Area of deterioration is large, however, depth of penetration up to 20mm is true only in some areas.
III (serious)	Loss of cross-sectional area is large with the depth of deterioration reaching the reinforcing bars or else the depth of penetration is 20 mm or less. However, the reason for deterioration is unknown or the progress of deterioration estimated to be fast and is progressing quickly.

5.11. Summary

One of the most important parameters that determine the safety of a building is its strength. In all cases, if the investigation finds the strength of concrete is less than the design strength, the result needs to be presented to the engineer in charge (civil engineer/structural engineer) who must make a decision based on the results presented as well as other considerations.

5.12. Keyword

Survey – assessment – detoriation – concrete structure.

5.13. Intext Questions

1. What are the informations to be collected during survey / impaction of structure?
2. How will you find the deterioration process of concrete structure?
3. Explain about the third survey to be conducted to assess the quality of structure.

❑

LESSON - 6

REPAIR METHODS

OBJECTIVE

❖ To study the different repair methods used in structures.

Contents

6.1. Repair
6.2. Rehabilitation
6.3. Retrofitting
6.4. Repair materials
 6.4.1. Criteria for selection of repair materials
 6.4.2. Methodology for selection of repair materials
 6.4.3. Material properties
 6.4.4. Compatibility
6.5. Factors affecting the selection of a repair material
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 6.8.1. Cement patching mortar and concrete
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6.21. Epoxy Bonding Agents
6.22. Surface Coatings
6.23. Sealents
6.24. Summary
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6.26. Intext Questions
6.27. References

6.1. REPAIR

Action taken to reinstate to an acceptable level the current functionality of a structure or its components which are defective or deteriorated, degraded or damaged in some way is called repair. The action may not be intended to bring the structure or its components so treated back to its original level or functionality or durability. The work may be intended simply to reduce the rate of deterioration or degradation without significantly enhancing the current level of functionality.

The main purpose of repairs is to bring back the architectural shape of the building so that all services start working and the functioning of building is resumed quickly. Repair does not pretend to improve the structural strength of the building and can be very deceptive for meeting the strength requirements of the next earthquake. The actions will include the following:

i. Patching up of defects such as cracks and fall of plaster
ii. Repairing doors, windows, replacement of glass panes
iii. Checking and repairing electric wiring
iv. Checking and repairing gas pipes, water pipes and plumbing services
v. Re-building non-structural walls, smoke chimneys, boundary walls, etc.
vi. Re-plastering of walls as required
vii. Rearranging disturbed roofing tiles
viii. Relaying cracked flooring at ground level
ix. Redecoration, whitewashing, painting, etc.

The architectural repairs as stated above do not restore the original structural strength of cracked walls or columns and may sometimes be very illusive, since the redecorates building will hide all the weaknesses and the building will suffer even more severe damage if shaken again by an equal shock since the original energy absorbing capacity will not be available.

6.2. REHABILITATION

It is the process of bringing the structure to its original level of function including durability and strength. The main purpose of restoration is to carry out structural repairs to load bearing elements. It may involve cutting portions of the elements and rebuilding them or simply adding more structural material so that the original strength is more or less restored. The process may involve inserting temporary supports, underpinning, etc. Some of the approaches are stated below:

- Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of nonshrinking mortar will be preferable.
- Addition of reinforcing mesh on both -faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it suitably. Several alternatives have been used.
- Injecting epoxy like material, which is strong in tension, into the cracks in walls, columns, beams, etc.

Where structural repairs are considered necessary, these should be carried out prior to or simultaneously with the architectural repairs so that total planning of work could be done in a coordinated manner and wastage is avoided.

6.3. RETROFITTING

Action to modify the functionality of a structure and to improve future performance in terms of load carrying capacity is called retrofitting. It relates to the strengthening of a structure against additional loading such as earthquake, etc. Commonly, strengthening procedures should aim at one or more of the following objectives:

(i) Increasing the lateral strength in one or both directions, by reinforcement or by increasing wall areas or the number of walls and columns.

(ii) Giving unity to the structure by providing a proper connection between its resisting elements, in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have the ability to resist them. Typical important aspects are the connections between roofs or floors and walls, between intersecting walls and between walls and foundations.

(iii) Eliminating features that are sources of weakness or that produce concentrations of stresses in some members. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses, large openings in walls without a proper peripheral reinforcement are examples of defect of this kind.

(iv) Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members. Since its cost may go to as high as 50 to 60% of the cost of rebuilding, the justification of such strengthening must be fully considered.

The effects of the above three operations on strength and deformation capacity can be shown in Figure 6.1.

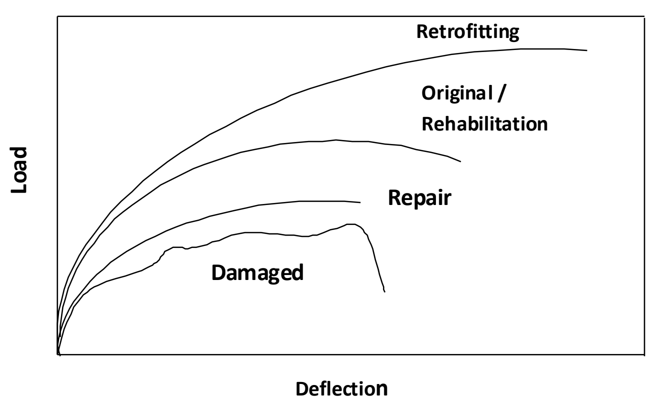

Figure 6.1. Effects of repair, rehabilitation and retrofitting

Repair of concrete structures is decided upon depending on factors such as the cause of damage, type, shape and function of the structure, the type and extent of damage, the capabilities and facilities available with builders, the availability of repair materials. The repair of concrete structures will involve treating the deteriorated material for extended durability, and / or strengthening of weak structural members to restore the load carrying capacity.

6.4. REPAIR MATERIALS

6.4.1. Criteria for selection of repair materials

A careful selection of repair material is necessary for the following reasons (Shan Somayaji, 1995 and ACI. 1980)

- ❖ Almost every repair job has unique condition and special requirements
- ❖ The composition and properties of repair materials have a profound effect and cured on the performance and durability of a repair.
- ❖ The repair materials perform adequately only if they are prepared, applied and cured as per the specified procedures, which may necessitate the use of appropriate tools and considerable skill.
- ❖ Repair materials, being generally proprietary in nature, are very costly.

6.4.2. Methodology for the selection of repair materials

The basic emphasis of the selection methodology is to maximize the performance of a repair material and ensure durability. The step-by-step procedure for selection of repair materials for chemical process industry is given below (Baaza, 1996):

Definition of service condition: Identification of major and minor chemicals, traces, spillage, cleaning chemicals, slurries and abrasives, their characteristics and their interaction with the environment under stagnant conditions, operating temperature range, dilute conditions, alternate wetting and drying effect.

Determination of appropriate application condition: Viscosity, flow characteristics, pot life, curing requirements, coverage, film / layer thickness and size of repair.

- ❖ Tools and equipments required: Pump, sprayer, injecting / grouting machine, dryer, mixer, batching plant, heater, etc.
- ❖ Repair and maintenance schedule: Anticipated durability of repair, maintenance requirements, and replacement / renewal / reinforcement requirements.
- ❖ Product performance record/history: durability, functionality, environment friendliness.
- ❖ Material testing and assessment for quality assurance and quality control
- ❖ Selection of applicator / contractor
- ❖ Material and job specifications

6.4.3. Material Properties

These differences in the mechanical properties of repair material and concrete substrate can be categorized as follows:

- ❖ Curing shrinkage of repair material relative to drying shrinkage of the concrete substrate.

- ❖ Differential thermal expansion/contraction between the repair material and concrete substrate.
- ❖ Differences in stiffness and Poisson's ratio causing unequal load sharing and strains resulting in interface stresses.
- ❖ Creep of repair material under sustained load as compared with that of the concrete.
- ❖ Relative fatigue performance of the components in the composite steel-concrete repair structures.

Such differences in properties may result in either initial tensile strains induced in the repair or cracking at or adjacent to the repair substrate interface. Both of these may reduce long-term structural capacity. Stresses that may be generated by relative volume changes between the repair material and the existing concrete substrate and service loads carried by the repair are shown in Figure 6.2. During service, incompatibilities in the form of differing elastic moduli and differential thermal movement between repair and substrate can cause problems.

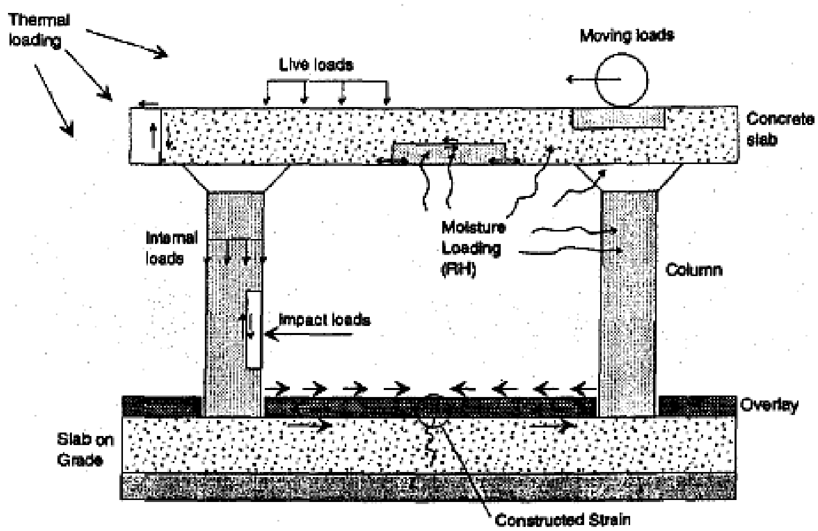

Figure 6.2. Possible loads acting on a repair

Also, creep of the repair material under sustained stress may render the repair less effective with time. The effects of carrying out repairs while the existing structure is under load and the influence of cyclic and impact loading may also be significant and failure to support some of the load temporarily before and during the repair process will result in stresses being transferred to undamaged parts of the member. Little load will be subsequently transmitted through repaired areas making the repair nonstructural. Figure 6.3 shows how load relief during the repair operation may enable the repair material to carry its share of stress.

Pre-repair considerations are as important as the repairs and consequently proper materials selection and surface preparation are essential to high quality, durable, and functional repair. Materials selected for use in concrete repair must meet specification requirements for the particular application or intended use. Engineers therefore need to know the mechanical and physical characteristics of available products and their proposed substrates before an assessment of structural compatibility can be made and suitable repair systems chosen.

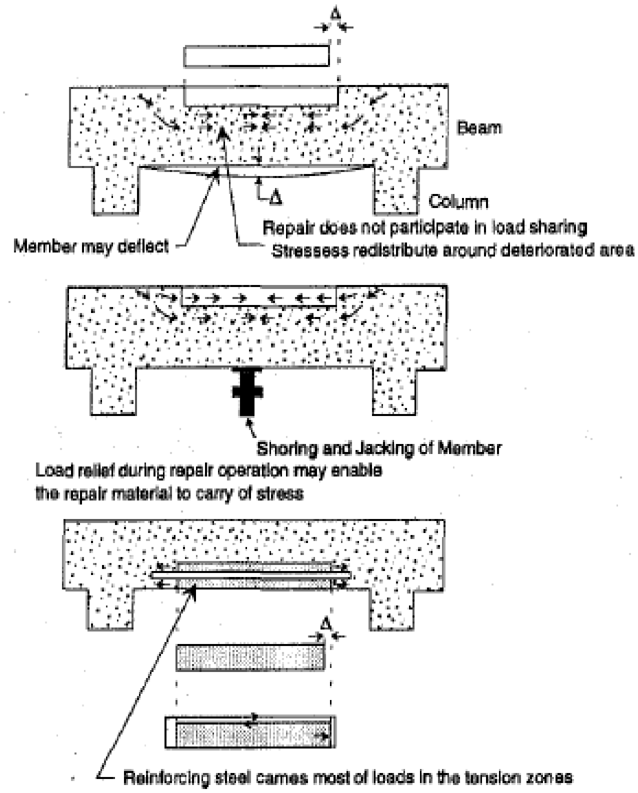

Figure 6.3. Repair in Tension and Compression Zones

6.4.4. Compatibility

Compatibility for a structural repair may be defined as that combination of properties and dimensions which ensures that interface bond strength is not exceeded and that the repair material carries its design load. This definition involves knowledge of repair dimensions in conjunction with a variety of material properties of both the repair material and substrate and as well, knowledge of the environmental influences and applied structural loads and resulting deformation. Table 6.1 suggests the properties generally required of repair materials as compared with the concrete substrate, to produce long-term structurally efficient repairs.

Table 6.1. General Requirements of Patch Repair Materials for Structural Compatibility

Property	***Relationship of repair mortar (R) to concrete substrate (C)***
Strength in compression, tension and flexure	$R \geq C$
Modulus in compression, tension and flexure	R~C
	Dependent on modulus and type of repair

Property	Relationship of repair mortar (R) to concrete substrate (C)
Poisson’s ratio	R~C
Coefficient of thermal expansion	R≥C
Adhesion in tension and shear	R≤C
Curing and long term shrinkage	R≥C
Strain capacity	Dependent on whether creep causes desirable or undesirable effects
Creep	
Fatigue performance	R≥C
Chemical reactivity	Should not promote alk/agg reaction, sulphate attack or corrosion of embedments in substrate
Electrochemical stability	Dependent on permeability of patch material and Chloride ion content of substrate

Repair materials can be formulated to provide a very wide range of properties from brittle to ductile and impermeable to porous. A wide variety of combinations of these properties is possible, with values selected to meet specific requirements of the application at hand. Many products particularly polymer-based materials are influenced by environmental conditions in service. In the repair situation environmental conditions can range from freezing to refractory temperatures and from very dry to full saturation. While cement-based materials are slightly affected by these conditions, the polymer-based materials are significantly affected. Therefore, the selection of the appropriate material is imperative to the intended purpose.

To understand how various factors affect the performance of repair systems it is necessary to consider the repair and substrate as components of a composite system, which includes dissimilar materials. The meaning of compatibility in such a system relates to a balance of physical, chemical and electrochemical properties and dimensions between repair materials and substrates. These ensure that a repair withstands stresses induced by volume changes, chemical and electrochemical effects without distress and deterioration in a specified environment over a designated period of time.

6.5. Factors affecting the selection of a repair material

Figure 6.4 presents the various factors affecting the selection of repair materials and these are discussed in detail below.

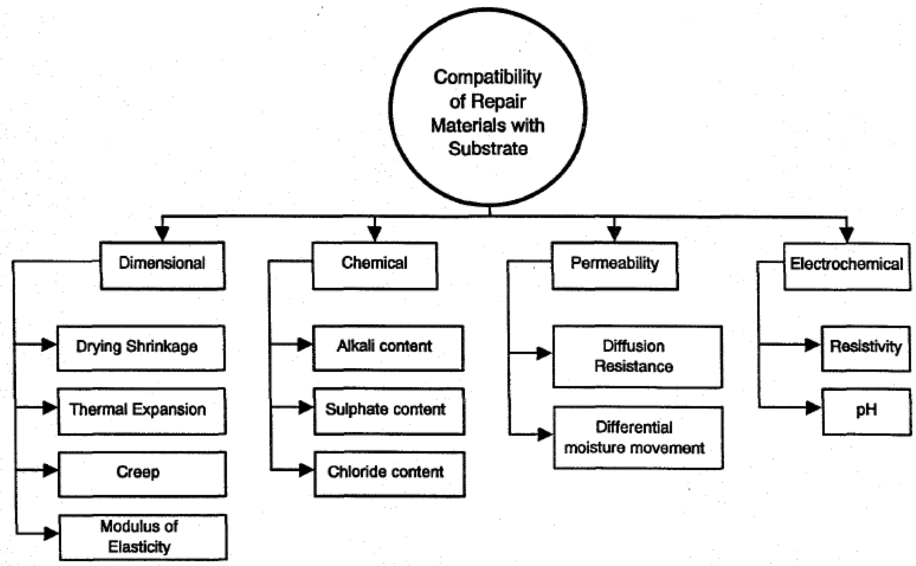

Figure 6.4. Various factors affecting the compatibility of repair materials

(i) Dimensional Stability

Dimensional incompatibility adversely affects the load carrying capacity of structural repairs. It may lead to the inability to carry the expected portion of the load and overstressing in the existing structure. The two volume-change properties that affect dimensional compatibility are drying shrinkage and thermal expansion. When making large thick patches or when placing an overlay, it is important to closely match the coefficient of thermal expansion of the repair material with the concrete being repaired. The differences in volume change that arise when a composite of two materials with quite different thermal coefficients undergo a significant temperature change, often cause failure at the bond interface or within the section of lower strength material.

(ii) Modulus of Elasticity

When materials with widely differing moduli are in contact with each other, the significant difference in deformability will cause problems under specific loading conditions. For example when the external load is perpendicular to the bond line (Figure 6.5-a) as in the case of pavement repair, a difference in modulus of elasticity between the repair material and concrete is usually not a problem. In repairs where the service load is parallel to the bond line however, the deformation of the lower modulus materials transfers the load to the higher modulus material which may then fracture. (Figure 6.5-b). Not all failures of bonded materials with widely differing modulus of elasticity are caused by external loads. Shrinkage or thermal expansion and contraction can cause loss of bond unless the modulus of the repair material is low enough to permit movement without excessive stress at the bond line.

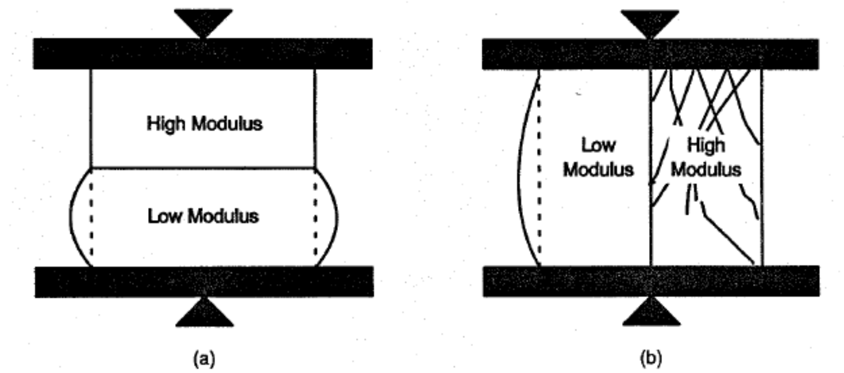

Figure 6.5. Materials with low modulus of elasticity deform more under given unit load

(iii) Chemical Reactivity

The reactivity of the patching material to steel reinforcement and other embedded metals, to the aggregate in the concrete or specific sealers or protective coatings applied over the patch must also be considered. Patching materials with low to moderate pH provide little protection to concrete while highly alkaline material may attack potentially reactive aggregates in the concrete. Therefore reactivity of patching materials with both the substrate and the surface protection product should be checked.

(iv) Electrochemical Compatibility

The resistivity of the patching material may also affect the durability of the patch and the concrete in the members undergoing repair. Materials that are highly resistive or non conductive have a tendency to isolate the repaired area from the adjacent undamaged areas.

Consequently, if there is a large permeability or chloride content differential between the patched area and the rest of the concrete, the corrosion current becomes concentrated in a restricted area and the rate of corrosion may then be accelerated, causing premature failure in either the patch or adjoining concrete. Figure 6.6 highlights the critical factors that largely govern the effectiveness and durability of concrete repairs in practice and must be considered in the design and specification process.

Compatibility cannot however, be tackled purely in material terms. It must factor in aspects of design detailing and construction. Several interrelated items such as surface preparation, method of application and inspection need to be considered to ensure long-term performance.

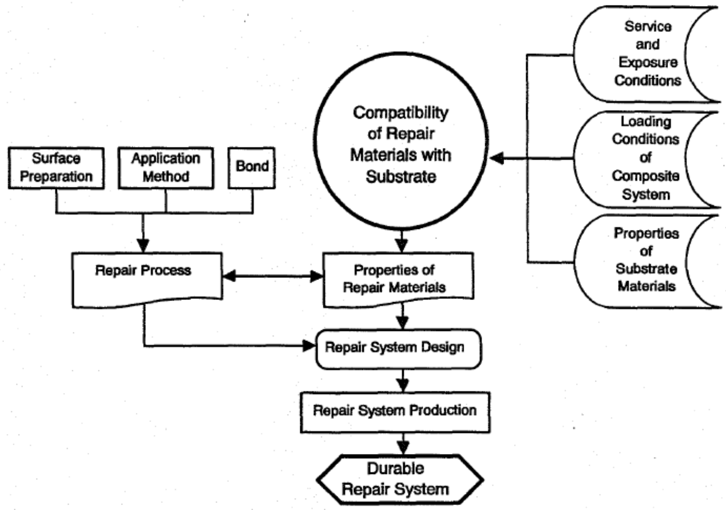

Figure 6.6. Factors affecting durability of concrete repair systems

6.6. Essential parameters for repair materials

Besides being of compatible properties, repair materials for cement concrete / mortar shall also be easy to apply and require no attention after the repair has been applied. The essential parameters for deciding upon a repair material for concrete are as follows:

- ❖ Low shrinkage properties
- ❖ Requisite setting / hardening properties
- ❖ Workability
- ❖ Good bond strength with existing sub-strate
- ❖ Compatible coefficient of thermal expansion
- ❖ Compatible mechanical properties and strength to that of the sub-strate
- ❖ Should allow relative movement, if expected, particularly in case of sealing of cracks or dealing with expansion joints
- ❖ Minimal or no curing requirement
- ❖ Alkaline character
- ❖ Low air and water permeability
- ❖ Aesthetics to match with surroundings
- ❖ Cost
- ❖ Durable, non degradable or non-biodegradable due to various forms of energy, life, UV rays, heat, etc.
- ❖ Non-hazardous / non – polluting

6.7. Classification of repair materials

The materials used for concrete repairs on the basis of type of application into following groups(Allen & Edwards, 1987 and ACI.1980):

(i) Patch Repair Materials

- ❖ Cementitious mortar / concrete
- ❖ Polymer modified cementitious mortar / concrete

- Polymer mortar / concrete
- Quick setting compounds
- High alumina cement based
- Calcium sulphate based
- Magnesium phosphates
- Sulphur concrete

(ii) Injection Grouts

- Cementitious grouts (with or without fibres)
- Gas forming grouts
- Sulpho-aluminate grouts
- Polymer grouts

(iii) Bonding Aids

- Polymer emulsion type
- Polymer resin type

(iv) Resurfacing Materials

- Protective coatings and membranes
- Impregnants and hydrophobic sealers
- Toppings / screeds
- Overlays
- Gunite / shotcrete

(v) Other Repair Materials

- Corrosion inhibitors
- Rebar protective coatings
- Cathodic protection
- Re-alkalization
- Materials for surface preparation
- Chemical rust removers for corroded reinforcement
- Joint sealers
- Surface coatings for protection of RCC

Products available in the market are generally in pre-proportioned and in pre-weighed packs together with accompanying instructions regarding mixing procedure, pot life, dosage and application procedure, etc. It is desirable that the manufacturer indicates the generic name and proportion of the components in the products on the packs. Table 6.2 shows the various types of deterioration in concrete, repair techniques and materials.

Table 6.2 Various types of deterioration in concrete, repair techniques and materials

Sl.No.	***Concrete damage***	***Repair technique***	***Repair materials***
1.	Alkali-Aggregate	Coatings	Bituminous Coatings
2.	Expansion	Concrete replacement Jacketing Total replacement	Epoxies Jacketing materials Latex-modified concrete Linseed oil Portland cement concrete
3.	Cavitation	Coatings Concrete replacement Pneumatically applied mortar Prepacked concrete	Bituminous Coatings Epoxies Jacketing materials Latex-modified concrete Portland cement concrete Portland cement mortar
4.	Cracks-Active	Caulking, Jacketing Stitching, Stressing	Elastic sealants Jacketing materials
5.	Cracks-Dormant	Acid etching Caulking Coatings Concrete replacement Dry pack Grinding Grouting Jacketing Pneumatically applied mortar Thin bonded or unbounded resurfacing Sand blasting, Stressing	Dry pack Epoxies High-speed setting materials Latex-modified concrete Portland cement concrete Portland cement grout Portland cement mortar
6.	Crazing	Coatings Grinding Pneumatically applied mortar Thin bonded or unbounded resurfacing Sack rub Sand blasting	Epoxies High-speed setting materials Latex-modified concrete Linseed oil Portland cement concrete Portland cement grout Portland cement mortar
7.	Dusting	Acid etching Coatings Grinding Jacketing Thin bonded or unbounded resurfacing Total replacement	Bituminous Coatings Epoxies High-speed setting materials Jacketing materials Latex-modified concrete Linseed oil Special floor aggregates Surface hardeners

Sl.No.	Concrete damage	Repair technique	Repair materials
8.	Fire damage	Acid etching Caulking Coatings Concrete replacement Dry pack Grinding Grouting Jacketing Mortar replacement Pneumatically applied mortar Prepacked concrete Thin bonded or unbounded resurfacing Sand blasting Stitching, Stressing Total replacement	Dry pack Elastic sealents Epoxies Expanding mortars Latex-modified concrete Portland cement concrete Portland cement grout Portland cement mortar
9.	Foam Scabbing	Coatings Concrete replacement Dry pack Mortar replacement Pneumatically applied mortar Thin bonded or unbounded resurfacing Total replacement	Dry pack Epoxies Expanding mortars Latex-modified concrete Portland cement concrete Portland cement grout Portland cement mortar

6.8. Patch Repairing

Once the deterioration process is initiated, repair is an important factor in extending the life span of structures. . The replacement of defective and spalled concrete to reintroduce a protective and durable environment around reinforcement is great importance. Therefore, deteriorated, reinforced concrete should be repaired with impermeable, highly alkaline cement-based materials, closely matched in properties to the parent concrete. Patch repair consists of removal of the damaged concrete, cleaning of rust, and restitution of the original geometry with a patch material. Patch repairing is one of the common concrete repair technologies, especially when a localized corrosion occurs

The repair material has the tendency to shrink after placement and hence the bond to substrate creates problem again. The following are the requirements of a good patching material:

1. Durable as the surrounding material
2. Require minimum site preparation
3. Resist wide range of temperature and moisture content
4. Chemically compatible with the substrate
5. Possess a similar colour and surface texture to the surrounding material.

The three major types of patch materials available in the market are Cement mortars and concretes, Polymer mortar and concrete and Epoxy-resin mortar and concrete. The plain cement mortar repairing is not suitable for structural repair works because of their dimensional instability, weak adhesion, and durability. The resin mortars including acrylics, polyurethanes, polyesters, and epoxies have superior properties as repair mortars. But the use of these mortars is restricted because of their cost and incompatibility with most of the substrate concretes. The cement-polymer mortar has better adhesive properties, crack resistibility and compatibility. Styrene Butadiene Rubber (SBR) latex is being effectively used to modify cement mortar to be used as a repair system in practical application. Some additional reinforcements are added partially or totally to restore the original area of bars.

6.8.1. Cement patching mortars and concretes:

A variety of cements such as ordinary Portland, rapid hardening, sulphate resistant and high alumina type can be used to produce mortar and concrete. Mortar can be used for small cavities up to 40mm depth while the concrete is often used for complete replacement of sections of deep cavities extending beyond reinforcing bars. The expansion agents such as aluminum powder, coke powder, anhydrous calcium sulfoaluminate, calcium oxide can be added in cement and causes the mortar and concrete to expand either in plastic state or after it has hardened. The expansion produced in the plastic stage creates an intimate contact with the substrate before it hardens, thus completely filling in space and promoting a good bond.

It is very popular repair material because of its advantages such as easy to use, cheaper in cost, similar appearance, readily available and familiarity. But it requires careful curing to prevent drying shrinkage. Chemical admixtures can be added to enhance the density and workability, accelerate hardening, control shrinkage and prevent aggregate reaction.

(i) Pre-packed concrete: Now-a-days, factory blended and packed cement concrete mixes incorporating chemical admixtures are readily available. For relatively large areas of repair, particularly on arches, soffits, etc., repair by sprayed concrete technique is the most effective. Dry guniting involving blowing of pre-blended and carefully graded cement and sand into a nozzle and spraying on to the surface under pressure after gauging with water is a very common and effective technique for thickness more than 40mm.

(ii) Preplaced concrete: It is technique in which the aggregates are placed first and then cemented together by introduction of intruded grout (cement –sand grout). It requires some special skills to carry out effectively. Pre-placed aggregate concrete (PAC) is concrete that is made by forcing into the voids of a mass of clean, graded coarse aggregate densely pre-packed in formwork (Figure 6.7). PAC is used where placing conventional concrete is extremely difficult, such as where massive reinforcement steel and embedded items are present, in underwater repairs, concrete and masonry repairs, or where shrinkage of concrete must be kept to a minimum.

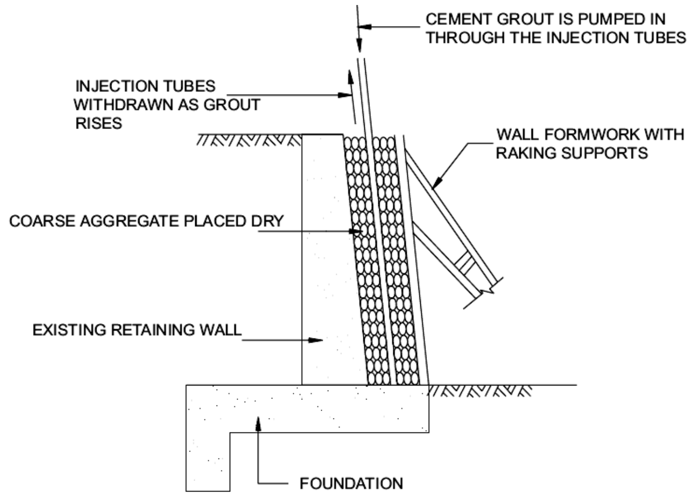

Figure 6.7(a): Pre-Placed Aggregate Concrete Repair to Concrete Wall

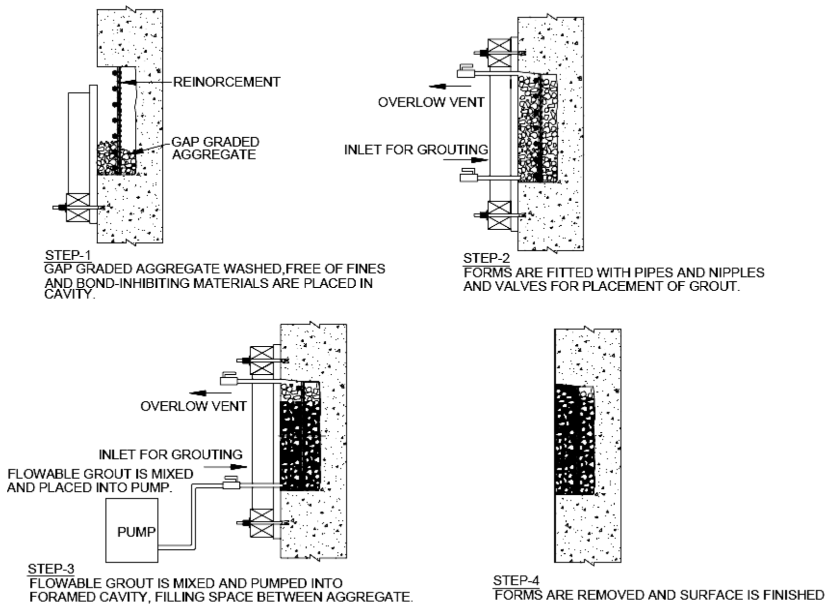

Figure 6.7(b) Preplaced Aggregate Concrete

For the purpose of this repair method, grout typically consists of sand, cement, pozzolana, plasticizer / super-plasticizer and air entraining agents (for anticipated freeze and thaw problem, if required). The pozzolana and the plasticizer/super-plasticizer are used to impart flowability to the grout. The coarse aggregate is washed to remove all fines and screened just prior to placement. Grout is then injected through forms to provide the cementing matrix. Grouting is begun at the bottom of the pre-placed aggregates.

Characteristics of the grout are affected by the water content, sand grading, cement, pozzolana and the types and amount of admixtures. For each design of grout mixture, there are optimum amounts of fillers and admixtures to produce the best pumpability or consistency. Proper proportioning of the structural grout mix components is necessary to get the required strength and durability of the finished pre-placed aggregate concrete. Trial mix design is necessary for each job.

In underwater repair, injection of grout at the bottom of the PAC displaces water, leaving a homogeneous mass of concrete with minimum of paste wash out. In such applications, addition of anti-wash admixtures minimizes the paste wash out. For underwater PAC, the quality of underwater should also be tested to determine its influence on PAC over a period of time for taking appropriate corrective action.

(iv) Dry Pack: Dry pack mortar is a stiff sand-cement mortar that is typically used to repair small areas that are deeper than they are wide. Dry pack mortar contains (by dry volume or weight) one part cement, 2½ parts sand, and enough water to produce a mortar that will just stick together while being molded into a ball with the hands (Figure 6.8). The ball should neither slump when placed on a flat surface, nor crumble due to lack of moisture. Place dry pack mortar immediately after mixing it. Compact the mortar in the hole by striking a hardwood dowel or stick with a hammer. The sticks are usually about eight to 12 inches long and no more than one inch in diameter. Use a wooden stick instead of a metal one because metal tends to polish the surface of the mortar, making bonding less certain and filling less uniform. Place and pack the mortar in layers to a compacted thickness of about 3/8 inch. Direct the tamping at a slight angle toward the sides of the hole to ensure maximum compaction in these areas. Overfill the hole slightly, then place the flat side of a hardwood piece against the hole and strike it several times with a hammer. If necessary, a few light strokes with a rag may improve its appearance.

Figure 6.8. Dry Pack mortar

In the market, the factory batched product is readily available to use and only requires the addition of water on site. The incorporation of shrinkage compensating additives ensures that the placed mortar maintains contact within the filled section. Dry Pack can be used for placing in horizontal and vertical gaps and may be rammed into place within gaps of 5mm to 100mm. Curing of exposed surfaces should be undertaken in accordance with good concrete practice. Suitable methods of curing include water spray, polythene sheeting and spray applied concrete curing membrane such as Polycure.

Dry Pack is alkaline when mixed with water and should not come into contact with skin or eyes.

Avoid inhalation of dust during mixing and wear safety glasses, dust mask and gloves. If skin contact occurs wash thoroughly with clean water. Should eye contact occur rinse immediately with plenty of clean water and seek medical advice.

(v) Fibre Reinforced Concrete (FRC): FRC is a concrete containing small embedded reinforcements called fibres which ensures the concrete to delay the formation and propagation of internal micro cracks. Fibres include steel fibres, glass fibres, synthetic and natural fibres.

Fibers are usually used in concrete to control cracking due to both plastic shrinkage and drying shrinkage. They also reduce the permeability of concrete and thus reduce bleeding of water. Some types of fibers produce greater impact, abrasion and shatter resistance in concrete. Generally fibers do not increase the flexural strength of concrete, and so cannot replace moment resisting or structural steel reinforcement. Indeed, some fibers actually reduce the strength of concrete. The amount of fibers added to a concrete mix is expressed as a percentage of the total volume of the composite (concrete and fibers), termed volume fraction (V_f). V_f typically ranges from 0.1 to 3%. Aspect ratio (l/d) is calculated by dividing fiber length (l) by its diameter (d). Fibers with a non-circular cross section use an equivalent diameter for the calculation of aspect ratio. If the modulus of elasticity of the fiber is higher than the matrix (concrete or mortar), they help to carry the load by increasing the tensile strength of the material. Increase in the aspect ratio of the fiber usually segments the flexural strength and toughness of the matrix. However, fibers which are too long tend to “ball” in the mix and create workability problems. Polypropylene and Nylon fibers can:

- Improve mix cohesion, improving pumpability over long distances
- Improve freeze-thaw resistance
- Improve resistance to explosive spalling in case of a severe fire
- Improve impact resistance
- Increase resistance to plastic shrinkage during curing

Steel fibers can

- Improve structural strength
- Reduce steel reinforcement requirements
- Improve ductility
- Reduce crack widths and control the crack widths tightly thus improve durability
- Improve impact & abrasion resistance
- Improve freeze-thaw resistance

Blends of both steel and polymeric fibers are often used in construction projects in order to combine the benefits of both products; structural improvements provided by steel fibers and the resistance to explosive spalling and plastic shrinkage improvements provided by polymeric fibers.

In certain specific circumstances, steel fiber can entirely replace traditional steel reinforcement bar in reinforced concrete. This is most common in industrial flooring but also in some other precasting applications. FRC is effectively used in tunneling projects, overlays on roads and runways using precast lining segments reinforced only with steel fibers.

6.8.2. Polymer concrete and mortar

Although its physical properties and relatively low cost make it the most widely used construction material, conventional Portland cement concrete has a number of limitations, such as low flexural strength, low failure strain, susceptibility to frost damage and low resistance to chemicals. These drawbacks are well recognized by the engineer and can usually be allowed for in most applications. In certain situations, these problems can be solved by using materials which contain an organic polymer or resin (commercial polymer) instead of or in conjunction with Portland cement. These relatively new materials offer the advantages of higher strength, improved durability, good resistance to corrosion, reduced water permeability and greater resistance to damage from freeze-thaw cycles. Polymer concrete is part of group of concretes that use polymers to supplement or replace cement as a binder. The types include polymer-impregnated concrete, polymer concrete, and polymer-Portland-cement concrete.

There are three principal classes of composite materials containing polymers: polymer impregnated concrete; polymer cement concrete and polymer concrete. The first is produced by impregnation of precast hardened Portland cement concrete with a monomer that is subsequently converted to solid polymer. To produce the second, part of the cement binder of the concrete mix is replaced by polymer (often in latex form).

Polymer Impregnated Concrete (PIC)

Polymer impregnated concrete is made by impregnation of precast hardened Portland cement concrete with low viscosity monomers (in either liquid or gaseous form) that are converted to solid polymer under the influence of physical agents (ultraviolet radiation or heat) or chemical agents (catalysts). It is produced by drying conventional concrete; displacing the air from the open pores (by vacuum or monomer displacement and pressure); saturating the open pore structure by diffusion of low viscosity monomers or a prepolymer-monomer mixture and in-situ polymerization of the monomer or prepolymer-monomer mixture. The important feature of this material is that a large proportion of the void volume is filled with polymer, which forms a continuous reinforcing network. The concrete structure may be impregnated to varying depths or in the surface layer only, depending on whether increased strength and/or durability is sought. The main disadvantages of polymer impregnated concrete products are their relatively high cost, as the monomers used in impregnation are expensive and the fabrication process is more complicated than for unmodified concrete.

Impregnation of concrete results in a remarkable improvement in tensile, compressive and impact strength, enhanced durability and reduced permeability to water and aqueous salt solutions such as sulfates and chlorides. The compressive strength can be increased from 35 MPa to 140 MPa, the water sorption can be reduced significantly. and the freeze-thaw resistance is considerably enhanced. The greatest strength can be achieved by impregnation of auto-claved concrete. This material can have a compressive-strength-to-density ratio nearly three times that of steel. Although its modulus of elasticity is only moderately greater than that of non-autoclaved polymer impregnated concrete, the maximum strain at break is significantly higher.

The monomers most widely used in the impregnation of concrete are the vinyl type, such as methyl methacrylate (MMA), styrene, acrylonitrile, t-butyl styrene and vinyl acetate. Acrylic monomer systems such as methyl methacrylate or its mixtures with acrylonitrile are the preferred impregnating materials, because they have low viscosity, good wetting properties, high reactivity, relatively low cost and result in products with superior properties. By using appropriate bifunctional or polyfunctional monomers (cross-linking agents) in conjunction with MMA, a cross-linked network is formed within the pores, resulting in products with greatly increased mechanical strength and higher thermal and chemical resistance. Improvement of these properties will depend on the degree of cross-linking. A cross-linking agent commonly used with vinyl monomers such as MMA and styrene is trimethylolpropane trimethacrylate.

Thermosetting monomers and prepolymers are also used to produce polymer impregnated concrete with greatly increased thermal stability (i.e. resistance to deterioration by heat). These include epoxy prepolymers and unsaturated polyester-styrene. These monomers and prepolymers are relatively viscous and, therefore, their use results in reduced impregnation. Their viscosity can be reduced by mixing them with low-viscosity monomers such as MMA.

Applications of polymer concrete impregnated include structural floors, high performance structures, food processing buildings, sewer pipes, storage tanks for seawater, desalination plants and distilled water plants, marine structures, wall panels, tunnel liners, prefabricated tunnel sections and swimming pools. Partially impregnated concrete is used for the protection of bridges and concrete structures against deterioration and repair of deteriorated building structures, such as ceiling slabs, underground garage decks and bridge decks.

Polymer modified Cement Concrete (PMC)

Polymer cement concrete is a modified concrete in which part (10 to 15% by weight) of the cement binder is replaced by a synthetic organic polymer. It is produced by incorporating a monomer, prepolymer-monomer mixture, or a dispersed polymer (latex) into a cement-concrete mix. To effect the polymerization of the monomer or prepolymer-monomer, a catalyst is added to the mixture. The process technology used is very similar to that of conventional concrete. Therefore, polymer cement concrete can be cast-in-place in field applications, whereas polymer impregnated concrete has to be used as a precast structure.

The properties of polymer cement concrete produced by modifying concrete with various polymers range from poor to quite favorable. Poor properties of certain products have been attributed to the incompatibility of most organic polymers and monomers with some of the concrete mix ingredients. Better properties are produced by using prepolymers, such as unsaturated polyester cross-linked with styrene or epoxies. To achieve a substantial improvement over unmodified concrete, fairly large proportions of these polymers are required. The improvement does not always justify the additional cost.

Modification of concrete with a polymer latex (colloidal dispersion of polymer particles in water) results in greatly improved properties, at a reasonable cost. Therefore, a great variety of latexes is now available for use in polymer cement concrete products and mortars. The most common latexes are based on poly (methyl methacrylate) also called acrylic latex, poly (vinyl acetate), vinyl chloride copolymers, poly (vinylidene chloride), (styrene-butadiene) copolymer, nitrile rubber and natural rubber. Each polymer produces characteristic physical properties. The acrylic latex provides a very good water-resistant bond between the modifying polymer and the concrete components, whereas use of latexes of styrene-based polymers results in a high compressive strength.

Curing of latex polymer cement concrete is different from that of conventional concrete, because the polymer forms a film on the surface of the product retaining some of the internal moisture needed for continuous cement hydration. Because of the film-forming feature, moist curing of the latex product is generally shorter than for conventional concrete.

Generally, polymer cement concrete made with polymer latex exhibits excellent bonding to steel reinforcement and to old concrete, good ductility, resistance to penetration of water and aqueous salt solutions, and resistance to freeze-thaw damage. Its flexural strength and toughness are usually higher than those of unmodified concrete.

The drying shrinkage of polymer cement concrete is generally lower than that of conventional concrete; the amount of shrinkage depends on the water-to-cement ratio, cement content, polymer content and curing conditions. It is more susceptible to higher temperatures than ordinary cement concrete. For example, creep increases with temperature to a greater extent than in ordinary cement concrete, whereas flexural strength, flexural modulus and modulus of elasticity decrease. These effects are greater in materials made with elastomeric latex (e.g., styrene-butadiene rubber) than in those made with thermoplastic polymers (e.g., acrylic).

The main application of latex-containing polymer cement concrete is in floor surfacing, as it is non-dusting and relatively cheap. Because of lower shrinkage, good resistance to permeation by various liquids such as water and salt solutions, and good bonding properties to old concrete, it is particularly suitable for thin (25 mm) floor toppings, concrete bridge deck overlays, anti-corrosive overlays, concrete repairs and patching.

Polymer Concrete (PC)

Polymer concrete (PC) is a composite material formed by combining mineral aggregates such as sand or gravel with a monomer. Cement and water are completely eliminated in this of concrete. Due to its rapid setting, high strength properties and ability to withstand a corrosive environment, PC is increasingly being used as an alternate to cement concrete in many applications, construction and repair of structures, highway pavements, bridge decks, waste water pipes and even structural and decorative construction panels.

There are three classes of polyester polyester used in polymer concrete mixtures Class I resins, resist mild corrodents and non oxidizing mineral acids. Class II resins, isophthalic type, are more resistant as compared to class I. Class III resins are based on bisphenol-A and have the best overall resistance to corrosive solutions. Increasing polymer content, resulted in increasing flexural strength and flexural modulus while the compressive strength decreased. In general, the lowest polymer content at which compressive strength/modulus was maximum represented the optimum polymer content for polymer concrete.

Aggregates used must be usually dry and free of dirt to get the best bond between aggregates and resin. Blasting sand aggregate systems showed an increase in the flexural modulus with the increase in the polymer content. Different types of carbon, glass and steel fibers can also be used to increase in compressive, flexural and impact strengths.

Table 6.3 gives the comparison of different polymer concretes and Portland cement concrete. Table 6.4 gives the general applications of polymer impregnated concrete and polymer cement concrete.

Table 6.3. Typical Properties of Polymer Concrete and Portland Cement Concrete

Material	Tensile Strength, MPa	Modulus of Elasticity, GPa	Compressive Strength, MPa	Shear Bond Strength, KPa	Water Sorption, %	Freeze-thaw Resistance, No. of Cycles / % Wt. Loss	Acid Resistance
Polymer impregnated concrete	10.5	42	140	-	0.6	3,500/2	10
Polymer impregnated concrete*	14.7	49	273	-	≤ 0.6	-	≥10
Polymer cement concrete	5.6	14	38	≥4,550	-	-	4
Portland cement concrete	2.5	24.5	35	875	5.5	700/25	-

* Concrete autoclaved before impregnation

Table 6.4. General Characteristics and Applications of Polymer-Modified Concretes

Material	***General Characteristics***	***Principal Applications***	***Remarks***
Polymer impregnated concrete	Consists generally of a precast concrete, which has been dried(and evacuated) then impregnated with a low viscosity monomer(or mixture of monomers) that polymerizes in situ to form a network within the pores. Impregnation results in markedly improved strength and durability (e.g., resistance to freeze-thaw damage and corrosion) in comparison with conventional concrete.	Principal applications include use in structural steel floors, food processing buildings, sewer pipes, storage tanks for seawater, desalination plants and distilled water plants, wall panels, tunnel liners and swimming pools.	The disadvantage is the relatively high cost, as the polymer is more expensive than cement and the production process is more complicated.
Polymer cement concrete	Products made with thermosetting(cross-linked) polymers and polymer latex have greater mechanical strength, markedly better resistance to penetration by water and salt, and greater resistance to freeze-thaw damage than Portland cement concrete; excellent bonding to steel reinforcing and to old concrete	Major applications are in floors, bridge decks, road surfacing and compounds for repair of concrete structures,(e.g. parking garage decks). Because of good adhesive properties, latex modified mortar is used for laying bricks, in prefabricated panels and in stone and ceramic tiles.	The mixing and handling are similar to Portland cement concrete. However, in the production process, air entrainment occurs without the use of an admixture, and prolonged moist curing is not required.

Advantages of polymer concrete include the following:

- Rapid curing at ambient temperatures
- High tensile, flexural, and compressive strength
- Good adhesion to most surfaces
- Good long-term durability with respect to freeze and thaw cycles
- Low permeability to water and aggressive solutions
- Good chemical resistance
- Good resistance against corrosion
- Lightweight
- May be used in regular wood and steel formwork
- May be vibrated to fill voids in forms
- Allows use of regular form-release agents
- Dielectric

6.8.3. Epoxy-resin mortar and concrete

Epoxies also come in the category of polymer but in the case the of epoxies, the polymerization process take place when tow materials called the epoxy resin and hardener come in contact by thoroughly mixing in specified proportion. The epoxy resin materials have good mechanical strength, chemical resistance and ease of working. These are being used in civil engineering structures for high performance coatings, adhesives, injection grouting, high performance systems, industrial flooring, etc. Epoxy patching materials are superior than cementitious materials because of their rapid cure and faster rate of strength development, high resistance to aggressive chemicals, high bond strength, good resistance and longterm durability. Epoxies are available as two-part systems consisting of a resin and a curing agent or hardener. When the two products combine together, it changes from a liquid to solid. The process of setting together is shown in Figure 6.9.

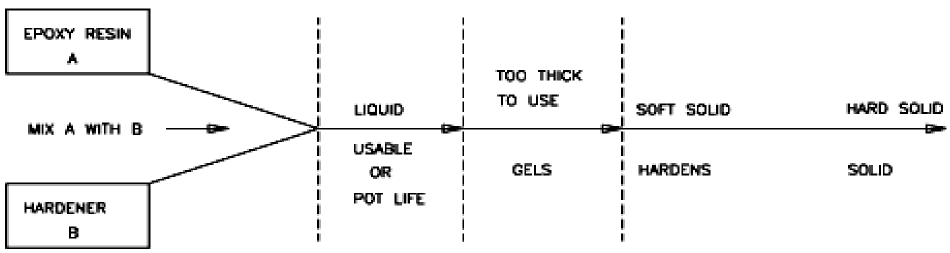

Figure 6.9. Process of setting

Field of applications:

Anti corrosive and water proofing coatings: Fusion bonded epoxy powder (FBEP) coatings are being used for protection of reinforcing bars against corrosion in RC structures located in highly aggressive areas. FBEP process provides a tough film, which can withstand bar bending without cracking.

Bond coats and grouts: Epoxies are used as bond coats and grouts due to their excellent adhesive qualities on cementitious as well as metallic surface. An epoxy film is brushed or sprayed onto the surface of the cleaned substrate and the new concrete is placed as the epoxy becomes tacky but before it hardens.

Structural repair to concrete: Due to their excellent mechanical properties and bond characteristics with most of the materials, epoxy mortars / concretes are used to make up the damaged or lost cover concrete, etc.

Epoxy resins

Epoxy resins are excellent binding agents with high tensile strength. There are chemical preparations the compositions of which can be changed as per requirements. The epoxy components are mixed just prior to application. The product is of low viscosity and can be injected in small cracks too. The higher viscosity epoxy resin can be used for surface coating or filling larger cracks or holes. The epoxy mixture strength is dependent upon the temperature of curing (lower strength for higher temperature) and method of application.

Epoxy mortar

For larger void spaces, it is possible to combine epoxy resins of either low viscosity or higher viscosity, with sand aggregate to form epoxy mortar. Epoxy mortar mixture has higher compressive strength, higher tensile strength and a lower modulus of elasticity than Portland cement concrete. Thus the mortar is not a stiff material for replacing reinforced concrete. It is also reported that epoxy is a combustible material. Therefore it is not used alone. The sand aggregate mixed to form the epoxy mortar provides a heat sink for heat generated and it provides increased modulus of elasticity too.

Reasons of concrete floors cracks

1. The sub-base wasn't properly compacted and the concrete settled in one area causing the floor to sink and crack.
2. The concrete floor isn't heated in the winter and the dirt under the concrete freezes causing it to expand and lift, or heave, the concrete. This upward pressure will make the concrete crack.
3. Newly poured concrete shrinks when it dries. If no expansion joints are cut into the concrete floor then shrinkage cracks will appear at some point in the future. These are quite common.

There are some other reasons like; pouring the concrete too wet, or too much calcium chloride was added to the mix and it dried too fast, or the concrete wasn't properly cured. All these will lead to shrinkage cracking at some point.

If you determine the concrete floor cracks are due to settling or heaving, then that is a sub-base issue and the concrete may have to be removed to correct that problem. Repairing these cracks will only be temporary if the concrete continues to move up or down.

If you think the concrete floor is done moving, you can repair those cracks and the shrinkage cracks with an epoxy concrete repair material you inject into the crack and add silica sand for filler. This will weld the crack together creating an excellent concrete floor repair.

Step by step procedure of epoxy concrete repair:

Step **1.** Clean out the crack with a vacuum, remove any loose cement or aggregates.

Step **2.** Inject the concrete crack repair resin into the crack to wet it, it will soon get tacky (Figure 6.10).

Step **3.** Push the dry silica sand into the crack to fill it.

Step **4.** Thoroughly saturate the sand with the concrete floor crack repair resin and fill it to the surface. A typical mortar mix consists of 1 part of mixed epoxy resin and hardener to 3 parts of sand.

Step **5.** In 10 - 15 minutes scrape the surface level with a putty knife.

Step **6.** Sand the surface smooth or lightly grind it with a hand grinder for a neat appearance.

Figure 6.10. Epoxy concrete repair

Use the moist material you just scraped off with the putty knife to patch any holes or chips in the concrete floor.

Hairline cracks can be done without the sand. Wide cracks can first be filled partially with the sand then start with step 2. This epoxy crack repair material sets up very quickly, only do 10 - 15 feet at one time.

6.9. POLYESTER RESINS

A two-part polyester resin (unsaturated polyester such as methylmethacrylates) based material suitable for the repair, surfacing, jointing and bedding of concrete, brickwork and masonry. Available in summer and winter grades, the mixed mortar will harden to give rapid strength gain and can be placed in section thicknesses of 5mm to 50mm in single layers. Thicker sections can be achieved by placing multiple layers. The bond strength developed is stronger than the tensile strength of most construction materials. Typical uses include bedding and fixing of precast concrete units, fixing of concrete barrier units, patch repairs to concrete floors, bedding of coping stones and infills where rapid service use is required.

Polyester patches cure faster than epoxy materials and are less sensitive to lower temperatures. The shrinkage that occurs on curing is significantly larger than that observed with epoxies. Therefore, the patch size is limited to thin sections and small areas. Vinyl ester patching compositions combine resiliency, impact resistance, and excellent chemical resistance.

6.10. ACRYLIC CONCRETE AND MORTAR

In this type, aggregates are mixed together with acrylic polymer to make concrete / mortar. Methylmethacrylate (MMA) and high molecular weight methacrylate (HMWM) monomers are used to make concrete. All components of the product can be mixed together and placed over a patch area filled with preplaced aggregates. Due to its rapid strength gain and high ultimate strength, it is widely used in bridge slabs, parking garage decks, industrial warehouse floors and tanks.

6.11. QUICK SETTING COMPOUNDS

Repairs in heavy trafficked areas are possible using faster setting and curing materials such as high alumina cement containing compounds, magnesium phosphates, molten sulfur and calcium sulphate based materials.

6.12. BITUMINUOUS MATERIALS

Hot mixed, densely graded asphalt concrete is widely used for patch repair works. It is cheaper material and easy to apply. Care should be taken against the poorly graded aggregate and unconsolidated patches that are prone to water pick up, which accelerate deterioration of surrounding concrete.

6.13. FERROCEMENT

Ferrocement is a technical term, not to be confused with ordinary reinforced concrete. It might be defined as a composite material consisting of a matrix made from hydraulic cement mortar and a number of layers of continuous steel mesh reinforcement distributed throughout the matrix. It is relatively cheap, strong and durable, and the basic technique is easily acquired. The basic parameters which characterise ferrocement are the specific surface area of reinforcement, the volume fraction of the reinforcement, the surface cover of the mortar over the reinforcement and the relatively high quality of the mortar. The thickness of ferrocement is in the range between 12 to 30mm. The wire mesh is mechanically connected to the parent surface by U shaped nails fixed with suitable epoxy bonding system.

Ferrocement behaves like reinforced concrete in its load bearing characteristics, with the essential difference being that crack development is retarded by the dispersion of the reinforcement in fine form through the mortar. The main advantages of ferrocement are low cost, the low level of skills required for hull construction, and reduced maintenance with increased resistance to rot and corrosion when compared to wood and steel. Very high cracking resistance ductility, flexibility and fatigue resistance are obtained using Ferrocement. Ferrocement is impermeable, light and tough. It can be easily cast. It is ideally suited for repair of large pop outs, craters and for laying of overlays, especially in view of the simplicity of technique involved.

6.14. SLURRY INFILTRATED FIBROUS CONCRETE (SIFCON)

Slurry-infiltrated fibrous concrete (SIFCON) can be considered as a special type of fiber concrete with high fiber content. It is also sometimes termed as 'high-volume fibrous concrete'. The matrix usually consists of cement slurry or flowing mortar. SIFCON has excellent potential for application in areas where high ductility and resistance to impact are needed.

While in conventional steel fibre reinforced concrete, the steel fibre content usually varies from 1 to 3 percent by volume, it varies from 4 to 20 percent in SIFCON depending on the geometry of the fibres and the type of application. The process of making SIFCON is also different, because of its high steel fibre content. While in SFRC, the steel fibres are mixed intimately with the wet or dry mix of concrete, prior to the mix being poured into the forms, SIFCON is made by infiltrating a low-viscosity cement slurry into a bed of steel fibres 'pre-packed' in forms/moulds (Figure 6.11). The matrix in SIFCON has no coarse aggregates, but a high cementitious content. However, it may contain fine or coarse sand and additives such as fly ash, micro silica and latex emulsions. The matrix fineness must be designed so as to properly penetrate (infiltrate) the fibre network placed in the moulds, since otherwise, large pores may form leading to a substantial reduction in properties. A controlled quantity of high-range water-reducing admixture (super plasticizer) may be used for improving the flowing characteristics of SIFCON. All types of steel fibres, namely, straight, hooked, or crimped can be used.

Proportions of cement and sand generally used for making SIFCON are 1: 1, 1:1.5, or 1:2. Cement slurry alone can also be used for some applications. Generally, fly ash or silica fume equal to 10 to 15% by weight of cement is used in the mix. The water-cement ratio varies between 0.3 and 0.4, while the percentage of the super plasticizer varies from 2 to 5% by weight of cement. The percentage of fibres by volume can be varying from 4 to 20%, even though the current practical range ranges only from 4 to 12%.

(i) Mould filled with fibres **(ii) Fibre pack well rammed** **(iii) Pouring of slurry**

Figure 6.11. Making of SIFCON

Applications of SIFCON

- Pavement rehabilitation and precast concrete products
- Overlays, bridge decks and protective revetments
- Seismic and explosive-resistant structures
- Security concrete applications (safety vaults, strong rooms etc)
- Refractory applications (soak-pit covers, furnace lintels, saddle piers)
- Military applications such as anti-missile hangers, under-ground shelters
- Sea-protective works
- Primary nuclear containment shielding
- Aerospace launching platforms
- Repair, rehabilitation and strengthening of structures
- Rapid air-field repair work
- Concrete mega-structures like offshore and long-span structures, solar towers etc.

6.15. SLURRY INFILTRATED MAT CONCRETE (SIMCON)

SIMCON can also be considered a pre-placed fibre concrete, similar to SIFCON. However, in the making of SIMCON, the fibres are placed in a “mat form” rather than as discrete fibres. The advantage of using steel fibre mats over a large volume of discrete fibres is that the mat configuration provides inherent strength and utilizes the fibres contained in it with very much higher aspect ratios. The fibre volume can, hence, be substantially less than that required for making of SIFCON, still achieving identical flexural strength and energy absorbing toughness. SIMCON is made using a non-woven “steel fibre mats” that are infiltrated with a concrete slurry. Steel fibres produced directly from molten metal using a chilled wheel concept are interwoven into a 0.5 to 2 inches thick mat. This mat is then rolled and coiled into weights and sizes convenient to a customer’s application (normally up to 120 cm wide and weighing around 200 kg). Since the mat is already in a preformed shape, handling problems are significantly minimised resulting in savings in labour cost. Besides this, “balling” of fibres does not become a factor at all in the production of SIMCON. A reinforcement level in SIMCON of only 25% of that of conventional SIFCON is found to provide as much as 75% of the latter’s ultimate flexural strength. SIMCON offers the designer a premium building material to meet the specialised niche applications, such as military structures or industrial applications requiring high strength and ductility.

6.16. GROUTS

Grout is a type of mortar used to fill joints, cracks, and cavities in tiles, masonry, and brickwork. It typically consists of water, cement, and sand; or cement and water. Used in semi-liquid form, it may be pumped, spread, or poured into cavities and allowed to harden, creating a tight, water-resistant seal.

The three main types of grout are epoxy, Portland cement - based, and furan resin. The epoxy type is strong and water resistant. It is available in 100 percent epoxy resin and modified epoxy emulsion form. Epoxy grout is generally more expensive than other types and can be difficult to find. However, it considered highly effective when a high level of water and stain resistance is desired. Portland cement-based grout is available in sanded, unsanded, pre-mixed, or powdered form. Furan resin grout is available in sanded or unsanded form. Instead of water, the furan resin type contains alcohol. This type of grout is considered extremely resistant to chemicals. It is best used when working with strong acids.

Grouting formulations are comprised of three basic elements: binder (clay types and properties, lime, synthetic), aggregate (sand, synthetic materials) and dispersant (water). Occasionally, additional elements such us additives (organic, inorganic) or thickeners are necessary. The formulation of a grout is a balancing of these elements in the correct proportions to achieve the desirable properties as established by the context and the critical performance properties. The following properties essential in better defining the grout composition and predicting its future performance:

- Viscosity
- Fluidity
- Penetration/Injectability
- Set time (initial and final)
- Stability

- ❖ Shrinkage
- ❖ Dilation
- ❖ Cohesion
- ❖ Bonding
- ❖ Crack formation within grout
- ❖ Compatibility (similar material if possible)
- ❖ Durability

6.17. Shotcrete [GUNITE]

Shotcrete is a method of applying a combination of sand and Portland cement which mixed pneumatically and conveyed in dry state to the nozzle of a pressure gun, where water is mixed and hydration takes place just prior to expulsion. A maximum coarse aggregate size of 10mm is used. It can be applied by either a 'dry mix' or 'wet mix' process.

The **dry process** involves premixing of the cement and sand, and transfer through a hose in a stream of compressed air (Figure 6.12). The end of the hose is equipped with a suitable nozzle at which point water is injected and mixed with the material as it exists at high velocity. The water can be adjusted at the nozzle and is restricted to approximately that required for proper hydration. Set accelerating admixtures in powder form are introduced into the premix, whereas a liquid accelerator is added to the water at the discharge nozzle, or as a separate injection at the nozzle. The material bonds perfectly to properly prepared surface of masonry and steel. The high impact force, at which the material is applied compacts it to form a dense concrete possessing very high bond strength. This process is particularly suited to restoration work that requires the replacement of concrete that has been lost or cut away and to insure against future damage by adding a further layer of concrete. Since low water-cement ratios are used, the no-slump characteristic affords it to be placed in layers of limited thickness on vertical and overhead surfaces.

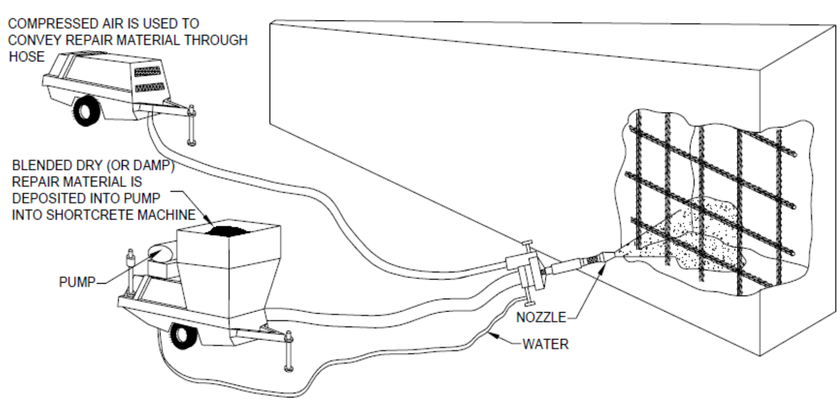

Figure 6.12. Dry mix Shotcrete

- ❖ The dry process involves the following steps:
- ❖ Thoroughly mixing the dry materials

- ❖ Feeding of these materials into mechanical feeder or gun
- ❖ Carrying the materials by compressed air through a hose to a special nozzle
- ❖ Introducing water at nozzle point and intimately mixing it with other ingredients at the nozzle
- ❖ Jetting the mixture from the nozzle at high velocity on to the surface to receive the shotcrete

In the **wet process**, a predetermined ratio of cement, aggregate and water is batched, mixed and transferred to a pump (Figure 6.13). The concrete is pumped along a flexible hose to a discharge nozzle from where it is projected at high velocity on to the surface to be coated. A rapid setting admixture like sodium aluminate or metasilicate solution is commonly added at the nozzle to enable buildup of thick layers. The wet process is similar to the use of mix design procedure suited for pumping. The cement content and aggregate to cement ratio, maximum aggregate size and grading is limited to what will give a pumpable mix with the equipment used. A higher water cement ratio than in dry process is usually necessary to provide pumpable mixes. It is more suited to the application of large quantities of material, typically in new construction. It is less suited for restoration works.

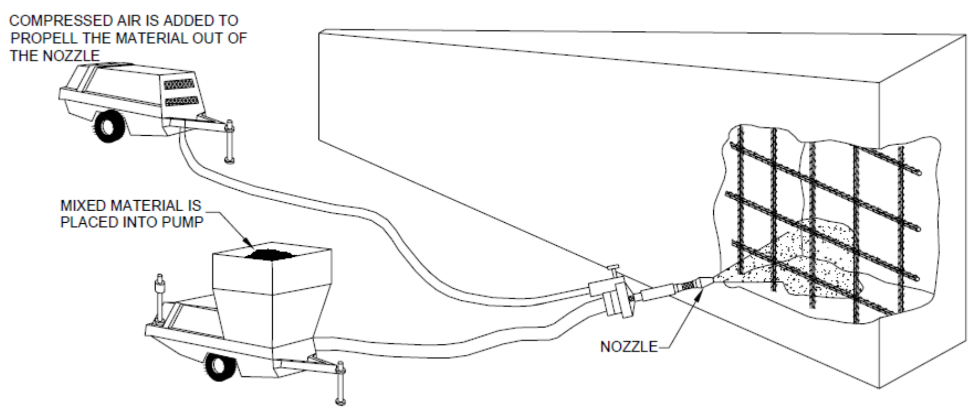

Figure 6.13. Wet mix Shotcrete

The wet process involves the following steps:

- ❖ Thoroughly mixing all the materials except accelerating agents (if used)
- ❖ Feeding of these materials into the delivery equipment
- ❖ Delivering the mixture by positive displacement or compressed air through a nozzle
- ❖ Jetting the mixture from the nozzle at high velocity on to the surface to receive the shotcrete

Moist curing is essential for shotcrete and protection from freezing or quick drying must be provided. In versatility of application to curved or irregular surfaces, its high strength after application and good physical characteristics, make for an ideal means to achieve added structural capability in walls and other elements.

There are some minor restrictions of clearance, thickness, direction of application, etc. It requires specially trained workers to achieve better results.

6.18. BONDING AGENTS

When applying conventional concrete, sprayed concrete, cement mortar, polymer modified mortar or epoxy mortar, bonding of repair material to the existing concrete is often a problem. Bonding agents are usually used to make perfect bond of new concrete to old concrete. Two of the critical factors affecting the bonding between new and old concrete are (i) the strength and integrity of the old surface and (ii) the cleanliness of the old surface. The use of epoxy resin or polymer latex bonding agent can assist in achieving a reliable bond.

6.19. Polymer Latex Emulsions

The latex emulsions generally used in cementitious compositions are of the oil-in-water type, and sometimes contain more than 50% water. They are generally stable in the cement/water system. However, not all emulsions are compatible with cement, and the selection of an appropriate product for a given application requires an understanding of its chemistry or, alternatively, consultation with the manufacturer.

Three methods can be used to modify a latex to make it a useful bonding agent:

(i) Prepare a neat cement slurry utilizing the latex as part of the mixing water;

(ii) Use a 1:1 water : latex diluted material;

(iii) Use re-emulsifiable latex, which can be softened and re-tackified upon contact with water.

The use of method (ii) is now discouraged because of the lack of bonding encountered in field applications. The use of the latex without any cement in the mix produces a failure plane because of the lack of film formation at the bond interface.

Styrene Butadiene (SBR) latex, which is compatible with cementitious compounds, is a copolymer. This type of latex shows good stability and is unaffected by the addition of relatively large amounts of electrolytes. SBR latex may coagulate if subjected to high temperatures, freezing temperatures, or severe mechanical action for prolonged periods of time.

Polyvinyl Acetate Latex (PVA)

Two main types of PVAs are used in repair: non-re-emulsifiable and emulsifiable. Non-re-emulsifiable PVA forms a film that offers good water resistance, ultraviolet stability, and aging characteristics. Because of its compatibility with cement, it is widely used as a bonding agent and as a binder for cementitious water-based paints and waterproofing coatings. Emulsifiable PVA produces a film that can be softened and re-tackified with water. This type of latex permits the application of a film to a surface long before the subsequent application of a water-based overlay. Its use is limited to specific applications where the possible infiltration of moisture to the bond line is precluded. It is most widely used as a bonding agent for plaster, and to bond finisher base-coat gypsum, or Portland cement plaster, to interior surfaces of cured cast in- place concrete.

Acrylic latex

Acrylic ester resins are polymers and copolymers of the esters of acrylic and methacrylic acids. Their physical properties range from soft elastomers to hard plastics. This type of emulsion is used in cementitious compounds in much the same manner as SBR latex.

6.20. Epoxy latex

Epoxy emulsions are produced from liquid epoxy resin mixed with the curing agent. In addition to serving as an emulsifying agent, the curing agent also serves as a wetting agent. From the time of mixing until gellation occurs, the emulsions are stable and can be diluted with water. Pot life can be varied from 1 to 6 hours depending on the curing agent selected and on the amount of water added. Most epoxy emulsions are prepared on the job site just before use because phase separation occurs in prepackaged emulsions. Equal parts of epoxy and curing agent are mixed, then blended for 2 to 5 minutes and allowed to set for 15 minutes to enable polymerization to begin. While the mixture is being mechanically agitated, water is added slowly to form the emulsion.

3.21. Epoxy Bonding Agents

Various epoxy products are available for the bonding of freshly placed concrete to cured concrete and of concrete to steel. Most products contain resins that are 100% solids. In severe drying conditions, the open time for bonding coats may be too short to ensure a good bond and such situations epoxy resin bonding is preferable. They may or may not contain fillers, such as calcium carbonate or silica flour, and other additives to enhance a particular property or reduce cost.

3.22. SURFACE COATINGS

The protective coatings of concrete surface generally improve the durability and greatly help to protect concrete deterioration due to environment effects. The protective coatings over concrete should possess the following properties:

1. Possess excellent bond to substrate
2. Be durable with a long useful life normally 5 years
3. Little or no colour change with time
4. Little or no chalking
5. Should have maximum permeability to allow water vapour escape from concrete substrate
6. Should sufficient impermeability against the passage of oxygen and carbon dioxide from air to concrete
7. Should be available in a reasoning range of attractive colours.
8. The properties of concrete which affect the successful application and performance of a coating are (i) porosity (ii) moisture content (iii) presence of contaminants on the surface. Most of the protective coatings used are (i)

Bituminous coatings and mastics (ii) Polyesters and Vinylesters (iii) Urethanes (iv) Epoxies (v) Neoprene (vi) Coal Tar Epoxy (vii) Acrylics.

Resin based toppings are used to protect the industrial floors subjected to heavy loadings. It consists of three components such as resin, curing agent (hardener) and aggregate fillers. Epoxies, polyurethane, polyester, polyacrylate and phenolic materials are used as resins. Epoxy mortar and glass fibre reinforced multicoat can also be used as toppings.

Surface Hardeners and Overlays: These are used to upgrade the floor's wear resistance, reduce dusting and increase chemical resistance.

Application fields: (i) Pavements of garages, parking, shopping malls, sport installations, schools, hospitals, subjected to moderate or medium traffic. (ii) Dock slabs in warehouses, industries, fuel stations with moderate erosion.

Advantages:

i. Increases the durability of the pavement
ii. Reduction of formation of superficial dust
iii. Improves the resistance against impacts
iv. Provides colour to pavement
v. It bonds structurally into the surface becoming part of the slab
vi. Easy application by powdering over the fresh concrete
vii. Low installation costs. Without maintenance
viii. Easily cleaning of the treated surface

Surface Hardener: Surface hardeners are used to repair and upgrading of industrial floors. Two types of surface hardeners are generally used. One is shake hardener and the other is liquid hardener. In shake hardener, high aggregate (mineral and metallic aggregates) to cement ratio (2:1) are used. In liquid hardener, sodium silicate or silicofluorides are used. Recently, dilute solutions of both solvent and water based emulsions of resins such as epoxies, urethanes and methylmethacrylates are used for this purpose. Three to four coats applied on successive days.

Overlays: It is applied as a second stage of construction on a new floor or deck, as preventive maintenance on a deck that has been open to traffic for a short time. The commonly used materials are (i) High early strength OPC concretes and mortars using superplasticizers (ii) Polymer latex concrete (iii) Epoxy mortars (iv) Fibre reinforced concrete incorporating steel or polypropylene fibres (v) Silica fume concrete.

Thin Polymer Overlays: These are used to improve the abrasion resistance and for creating waterproofing barriers on the surface and act as protective coatings. These are applied in less than 10mm thickness. It is quite suitable for improving surface characteristics and also it acts as protective coatings. It comprises of one coat of primer and one or more coats of sealent. The primer coat shall consist of the same material as in the primer but with the addition of silica filler, titanium dioxide pigment and carbon black pigment.

Thin Epoxy Overlays: These are used to improve the abrasion resistance of surface and for creating waterproofing / protective coating. Thin epoxy overlays are applied in 2 to 3 mm thickness. It consists of resin and hardener. The strength gain is much faster than polymer overlays. There are several types for epoxy overlays available to suite different performance requirement. Epoxy overlays require protective coating in exposed locations subjected to ultra violet exposure.

6.23. SEALANTS

Sealants are used to seal the concrete surfaces and joints to prevent the ingress of moisture, solid matters such as dust and sand into the structures. Concrete sealers are finish coatings used to protect the concrete and to accommodate joint movements. Sealers act to prevent damage from traffic, water and chemical agents. Sealers reduce the porosity of concrete and prevent water and dirt from getting into the concrete.

Types of Sealers:

Film Formers:

It creates a barrier on the concrete's surface. It is used for enhancing exposed aggregate colored concrete since they are shiny.

Penetrates:

These sealers penetrate into the concrete (Figure 6.14). They get 1 to 4 millimeters into the concrete to increase water repellency. Unlike the film formers, they do not leave a sheen or gloss.

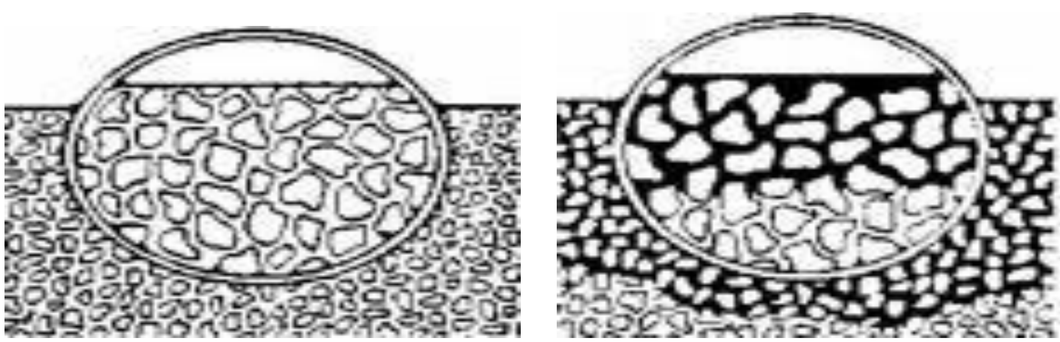

Figure 6.14. Application of penetrate

Which Type of Sealer to Use?

It depends on: (i) Surface finish and (ii) Type of maintenance

film formers ⟶ glossy finish

penetrate ⟶ matte finish OR slip-free surface

Examples on sealers:

- Acrylic
- Silicone
- Mastics

- Latex
- Urethane
- silane
- siloxane
- Epoxy Resin
- wax forms

Sealer Application:

- Surface Prepration:
  - clean the concrete surface before sealing.
  - coat the floor for 10 to 15 minutes.

Apply Sealer:

Clean Up:

Wash up with warm soap and water immediately.

6.24. Summary

Different methods of repair, rehabilitation and retrofitting are studied. The quality of repair materials is also discussed.

6.25. Keywords

Repair – Rehabilitation – Retrofitting – Grout – Shotcrete – SIFCON – SIMCON – Ferrocement – Polymer – Epoxy

6.26. Intext Questions

1. Distinguish between repair, rehabilitation and retrofitting.
2. What are the factors affecting the selection of repair materials?
3. Explain the technique called “Grouting”.
4. Explain about Polymer Conceréte.
5. Explain the difference between SIFCON and SIMCON.

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STRENGTHENING OF RC MEMBERS

Objective

- To study the strengthening techniques for RC elements using different methods.

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7.1. Introduction

7.2. Crack injection repair to concrete structures

7.2.1. Epoxy resins

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7.1. Introduction

The strengthening of reinforced concrete members is a task that should be carried out by a structural engineer according to calculations. Only a few suggestions are included to illustrate the ways in which the strengthening could be done.

7.2. Crack injection repair to concrete structures

Resin Injection — Resin injection is used to repair concrete that is cracked or delaminated and to seal cracks in concrete to water leakage. Two basic types of resin and injection techniques are used to repair Reclamation concrete.

7.2.1. Epoxy Resins – Epoxy resins cure to form solids with high strength and relatively high moduli of elasticity. These materials bond readily to concrete and are capable, when properly applied, of restoring the original structural strength to cracked concrete. The high modulus of elasticity causes epoxy resin systems to be unsuitable for rebonding cracked concrete that will undergo subsequent movement. Epoxy resin has been used to seal cracks in concrete to waterflow. The epoxies, however, do not cure very quickly, particularly at low temperatures, and using them to stop large flows of water may not be practical. Cracks to be injected with epoxy resins should be between 0.005 inch and 0.25 inch in width. It is difficult or impossible to inject resin into cracks less than 0.005 inch in width, while it is very difficult to retain injected epoxy resin in cracks greater than 0.25 inch in width, although high viscosity epoxies have been used with some success.

Epoxy resins cure to form relatively brittle materials with bond strengths exceeding the shear or tensile strength of the concrete. If these materials are used to rebond cracked concrete that is subsequently exposed to loads exceeding the tensile or shear strength of the concrete, it should be expected that the cracks will recur adjacent to the epoxy bond line. In other words, epoxy resin should not be used to rebond "working" cracks. Epoxy resins will bond with varying degrees of success to wet concrete, and there are a number of special techniques that have been developed and used to rebond and seal water leaking cracks with epoxy resins. These special techniques and procedures are highly technical and, in most cases, are proprietary in nature. They may have application on Reclamation projects, but only after a thorough analysis has been performed to ensure that the more standard repair procedures will not be successful or cost effective.

7.2.2 Polyurethane Resins—Polyurethane resins are used to seal and eliminate or reduce water leakage from concrete cracks and joints. They can also be injected into cracks that experience some small degree of movement. Such systems, with the exception of the two-part solid polyurethanes, have relatively low strengths and should not be used to structurally rebond cracked concrete. Cracks to be injected with polyurethane resin should not be less than 0.005 inch in width. No upper limit on crack width has been established for the polyurethane resins at the time this is being written. Polyurethane resins are available with substantial variation in their physical properties. Some of the polyurethanes cure into flexible foams. Other polyurethane systems cure to semiflexible, high density solids that can be used to rebond concrete cracks subject to movement. Most of the foaming polyurethane resins require some form of water to initiate the curing reaction and are, thus, a natural selection for use in repairing concrete exposed to water or in wet environments.

(a) Preparation—Cracks, joints, or lift lines to be injected with resin should be cleaned to remove all the contained debris and organic matter possible. Several techniques have been used, with varying degrees of success, for cleaning such cracks. Once injection holes have been drilled, repeated cycles of alternately injecting compressed air followed by water have been very useful in flushing and cleaning cracks subject to water leakage. The successful use of soaps in the flushing water has been reported by some practitioners. Complete removal of such materials once injected into cracks is troublesome and may create more problems than it is worth. The use of acids to flush and clean cracks is not allowed by Reclamation. Cracks subject to epoxy injection for purposes of structural rebonding should not normally be injected with water. The epoxy resins will bond to wet concrete, but they develop higher bond strength when bonding to dry concrete.

(b) Materials—Epoxy resin used for crack injection should be a 100-percent solids resin meeting the requirements of specification ASTM C-881 for type I or IV, grade 1, class B or C. If the purpose of injection is to restore the concrete to its original design load bearing capabilities, a type IV epoxy should be specified and used. If the purpose does not involve restoration of load bearing capabilities, a type I epoxy is sufficient. Polyurethane resin used for crack injection should be a two-part system composed of 100-percent polyurethane resin as one part and water as the second part. The polyurethane resin, when mixed with water, should be capable of forming either a closed cell flexible foam or a cured gel, dependent on the water to resin mixing ratio.

(c) Injection Equipment—Resins can be injected with several types of equipment. A commercial polyurethane injection pump is shown in figure 7.1. Small repair jobs employing epoxy resin can use any system that will successfully deposit the epoxy in the required zones. Such systems could use a prebatch arrangement in which the two components of the epoxy are batched together prior to initiating the injection phase with equipment such as small paint pressure pots. The relatively short pot life of the epoxy makes this technique rather critical as far as timing is concerned.

Figure 7.1. Commercial polyurethane injection pump.

(d) Application—The success of resin injection repair projects is directly related to the experience and knowledge of the injection contractor. Reclamation requires that an injection contractor have a minimum of 3 years' experience in performing injection work similar to that being contracted for and that a minimum of five projects be included in that experience.

(1) Application of Epoxy Resin by Pressure Injection—The objective of epoxy resin injection is to completely fill the crack or delamination being injected and retain the resin in the filled voids until cure is complete. The first step in the resin injection process is to thoroughly clean the concrete surface in the vicinity of the cracks of all loose or deteriorated concrete and debris. The area of injection is then inspected and the injection port location pattern established. Several different types of injection patterns can be used:

7.3. Jacketing

(i) RC columns can best be strengthened by jacketing, and by providing additional cage of longitudinal and lateral tie reinforcement around the columns and casting a concrete ring, Figure 7.2, the desired strength and ductility can thus be built-up. It is necessary to ensure perfect bond between the old and new concrete by providing shear keys (Figure 7.2a) and effective bond coat with the use of epoxy of polymer modified cement slurry giving strength not less than that of new concrete.

(ii) Jacketing a reinforced concrete beam can also be done in the above manner. For holding the stirrup in this case, holes will have to be drilled through the slab, Figure 7.3.

(iii) Similar technique could tie used for strengthening RC shear walls.

(iv) Inadequate sections of RC column and beams can also be strengthened by removing the cover to old steel, welding new steel to old steel and replacing the cover. In all cases of adding new concrete to old concrete, the original surface should be roughened, groves made in the appropriate direction for providing shear transfer. The ends of the additional steel are to be anchored in the adjacent beams or columns as the case may be.

(v) RC beams can also be strengthened by applying prestress to it so that opposite moments are caused to those applied. The wires will run on both sides of the web outside and anchored against the end of the beam through a steel plate.

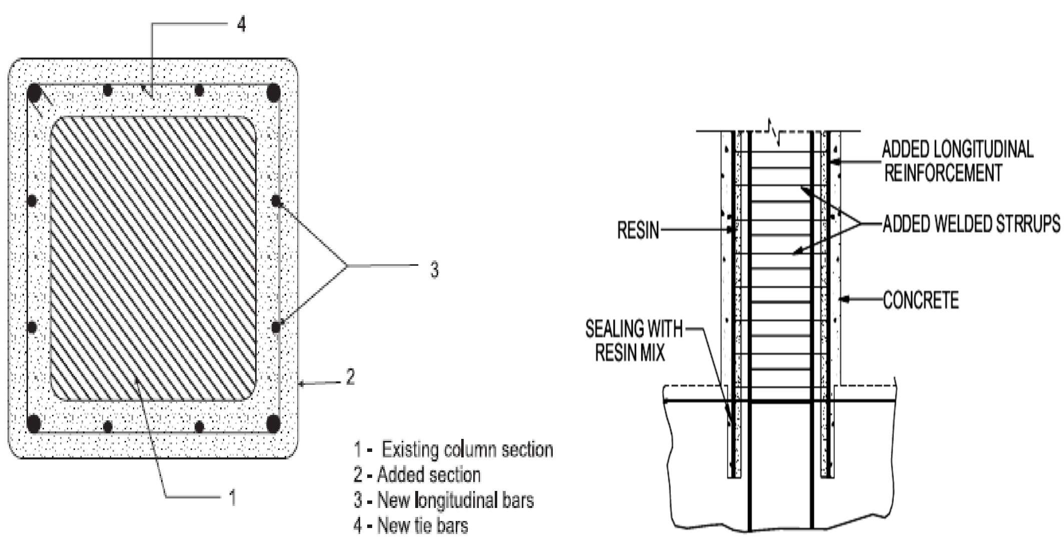
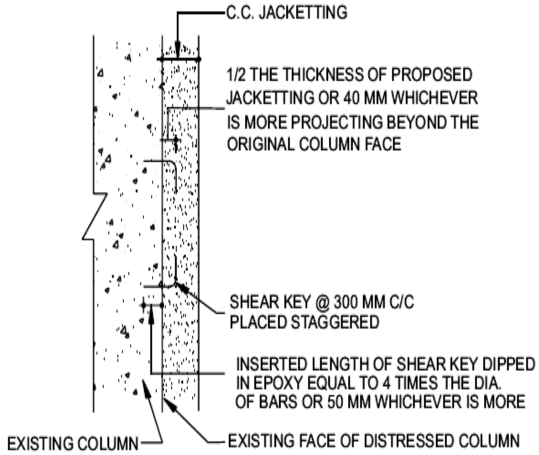


Figure 7.2. Jacketing a concrete column



Note: Shear key shall be provided in location where it is to function as such ie. Side of columns and beams etc. They shall not be provided in the soffit of the beams or ceiling of slab

Figure 7.2a. Jacketing a concrete column

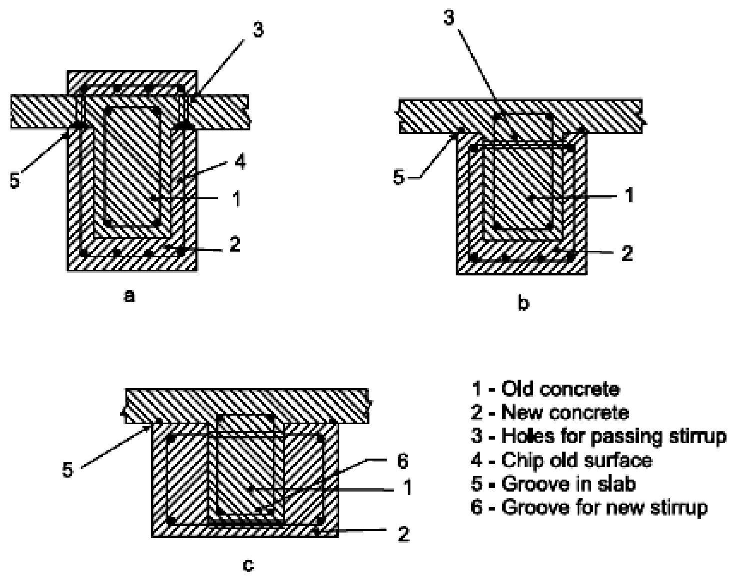


Figure 7.3. Increasing the section and reinforcement of existing beams

7.4. PLATE BONDING

Plate bonding is an inexpensive, and versatile and advanced technique for rehabilitation, up gradation of concrete structures by mechanically connecting MS plates by bolting and gluing to their surfaces with epoxy as shown in Figure 7.4. Plate bonding can substantially increase strength, stiffness, ductility and stability of the reinforced concrete elements and can be used effectively for seismic retrofitting. The bolts, which are first used to hold plates in position during construction, act as permanent shear connectors and integral restraints. The bolts are also designed to resist interface forces assuming the epoxy glue used as non-existent assuming it as destroyed by fire, chemical break down, rusting or simply bad workmanship. Since epoxy is prone to premature debonding, use of mechanical anchorage along with epoxy binding is considered more reliable.

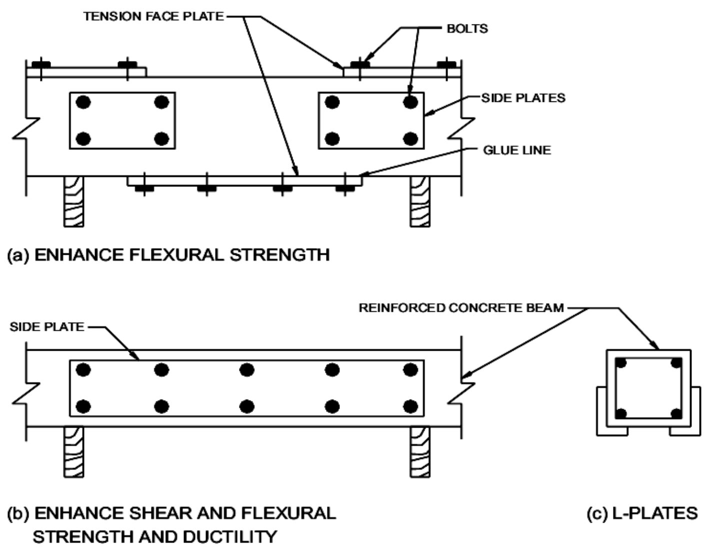


Figure 7.4. Increasing the section and reinforcement of existing beams

7.5. STRENGTHENING OF FOUNDATIONS

Seismic strengthening of foundations before or after the earthquake is the most involved task since it may require careful underpinning operations. Some alternatives are discussed for preliminary consideration of the strengthening scheme.

i. Introducing new load bearing members including foundations to relieve the already loaded members. Jacking operations may be needed in this process.

ii. Improving the drainage of the area to prevent saturation of foundation soil to obviate any problems of liquefaction which may occur because of poor drainage.

iii. Providing apron around the building to prevent soaking of foundation directly and draining off the water.

iv. Adding strong elements in the form of reinforced concrete strips attached to the existing foundation part of the building. These will also bind the various wall footings and may be provided on both sides of the wall, Figure 7.5. To avoid digging the floor inside the building, the extra width could be provided only on the outside of external walls. The extra width may be provided above the existing footing or at the level of the existing footing. In any case the reinforced concrete strips and the walls have to be linked by a number of keys, inserted into the existing footing.

   Note: To avoid disturbance to the integrity of the existing wall during the foundation strengthening process, proper investigation and design is called for.

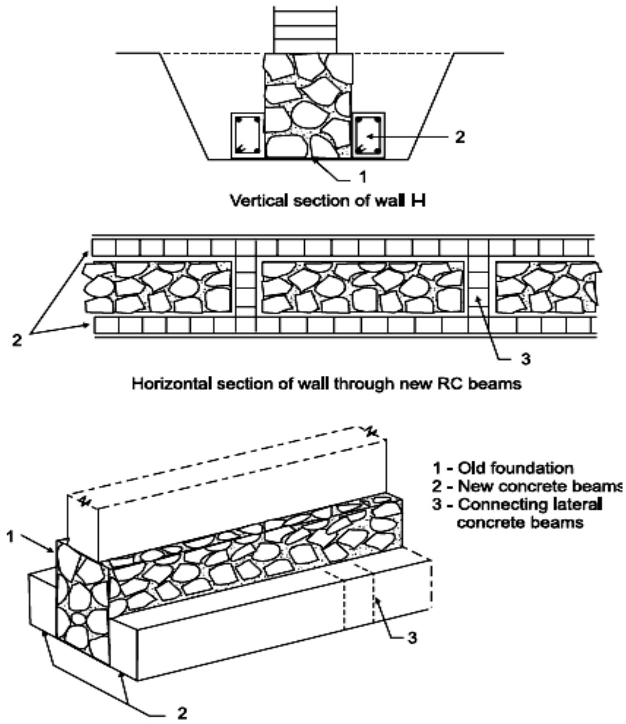


Figure 7.5. Improving a foundation by inserting lateral concrete beams strengthening existing walls

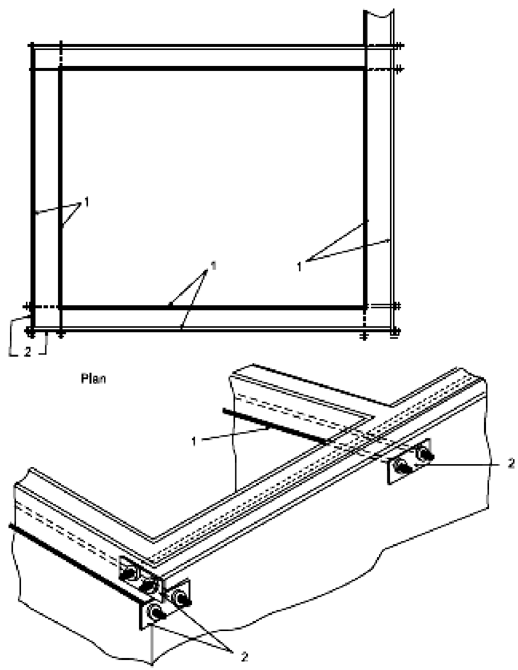
The lateral strength of buildings can be improved by increasing the strength and stiffness of existing individual walls whether they are cracked or uncracked. This can be achieved (a) by grouting; (b) by addition of vertical reinforced concrete coverings on the two sides of the wall (c) by pre-stressing walls.

(a) Grouting: A number of holes are drilled in the wall. First water is injected in order to wash the wall inside and to improve the cohesion between the grouted mixture and the wall elements. Secondly a cement water mixture (1:1) is grouted at low pressure (0.1 to 0.25 MPa) in the holes starting from the lower holes and going up. Alternatively, polymeric mortars may be used for grouting. The increase of shear strength which can be achieved in this way is considerable. However grouting cannot be relied on as far as the improving or connection between orthogonal walls is concerned. Note that pressure needed for grouting can be obtained by gravity flow from super-elevated tanks.

(b) Strengthening with wire mesh two steel meshes (welded wire fabric with an elementary mesh) are placed on the two sides of the wall, they are connected by passing steel each 500 to 750 mm apart. A 20 to 40 mm thick cement mortar or microconcrete layer is then applied on the two networks thus giving rise to two interconnected vertical plates. This system can also be used to improve connection of orthogonal walls.

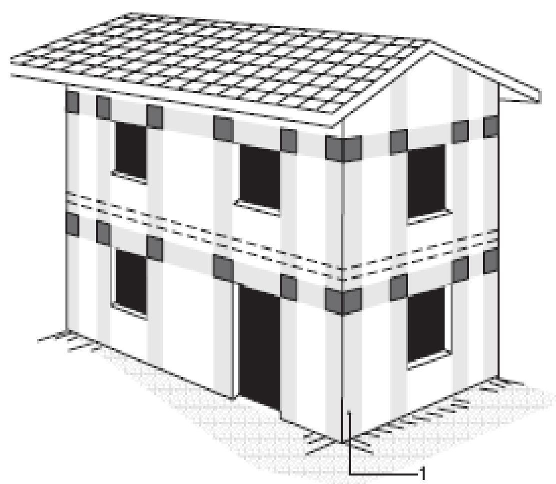
(c) Connection between existing stone walls In stone buildings of historic importance consisting of fully dressed stone masonry in good mortar effective sewing of perpendicular walls can be done by drilling inclined holes through them, inserting steel rods and injecting cement grout.

(d) Prestressing: A Horizontal compression state induced by horizontal tendons can be used to increase the shear strength of walls. Moreover this will also improve considerably the connections of orthogonal walls, Figure 7.6. The easiest way of affecting the precompression is to place two steel rods on the two sides of the wall and strengthening them by turnbuckles. Note that good effects can be obtained by slight horizontal prestressing (about 0.1 MPa) on the vertical section of the wall. Prestressing is also useful to strengthen spandrel beam between two rows of openings in the case no rigid stab exists.



1. Steel rod for prestressing **2. Anchor plates**

Fig. 7.6. Strengthening of walls by prestressing



1. Wire mesh with width > 400mm

Fig. 7.7. Splint and bandage strengthening

External binding

Opposite parallel walls can be held to internal cross walls by prestressing bars as illustrated above, the anchoring being done against horizontal steel channels instead of small steel plates. The steel channels running from one cross wall to the other will hold the walls together and improve the integral box like action of the walls. The technique of covering the wall with steel mesh and mortar or micro-concrete may be used only on the outside surface of external walls but maintaining continuity of steel at the corners. This would strengthen the walls as well as bind them together. As a variation and for economy in the use of materials, the covering may be in the form of vertical splints between openings and horizontal bandages over spandrel walls at suitable number of points only, Figure 7.7.

Other points

(i) Masonry arches: If the walls have large arched openings in them, it will be necessary to install tie rods across them at springing levels or slightly above it by drilling holes on both sides and grouting steel rods in them, Figure 7.8(a). Alternatively, a lintel consisting of steel channels or I shapes, could be inserted just above the arch to take the load and relieve the arch as shown at Figure 7.8(b). In jack-arch roofs, flat iron bars or rods may be provided to connect the bottom flanges of I-beams, connected by bolting or welding.

(ii) Random rubble masonry walls are most vulnerable to complete collapse and must be strengthened by internal impregnation by rich cement mortar grout in the ratio of 1:1 or better still covered with steel mesh and mortar. Damaged portions of the wall, if any, should be reconstructed using richer mortar.

(iii) For bracing the longitudinal walls of long barrack type buildings, a portal type framework can be inserted transverse to the walls and connected to them. Alternatively, masonry buttresses or, pillasters may be added externally as shown in Figure 7.9.

(iv) In framed buildings, the lateral resistance can be improved by inserting knee braces or full diagonal braces or inserting infill walls.

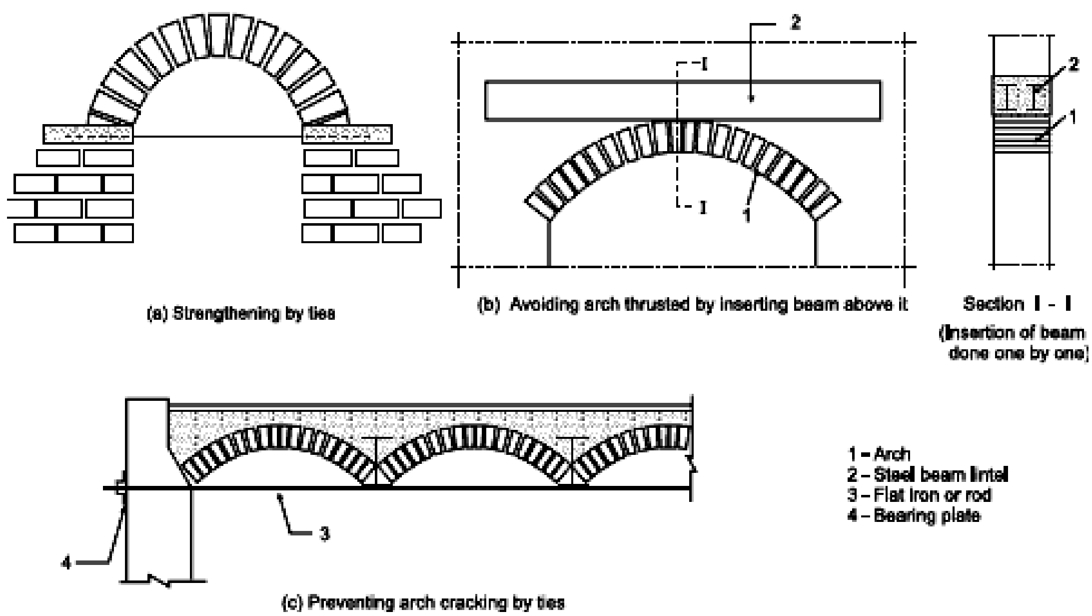
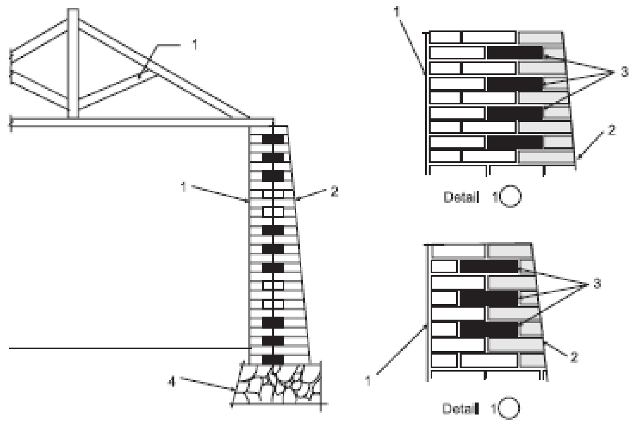


Figure 7.8. Strengthening an arched opening in masonry wall



1.Exisiting wall 2. Buttress of plaster 3. Key stone 4. Foundation

Figure 7.9. Strengthening of long walls by buttresses

7.6. TECHNIQUES TO RESTORE ORIGINAL STRENGTH

While considering restoration work, it is important to realize that even fine cracks in load bearing members which are unreinforced, like masonry and plain concrete reduce their resistance very largely. Therefore all cracks must be located and marked carefully and the critical ones fully repaired either by injecting strong cement or chemical grout or by providing external bandage. The techniques are described below along with other restoration measures.

Small cracks

If the cracks are reasonably small (opening width = 0.075 cm), the technique to restore the original tensile strength of the cracked element is by pressure injection of epoxy. The external surfaces are cleaned of non-structural materials and plastic injection ports are placed along the surface of the cracks on both sides of the member and are secured in place with an epoxy sealant. The centre to centre spacing of these ports may be approximately equal to the thickness of the element. After the sealant has cured, a low viscosity epoxy resin is injected into one port at a time, beginning at the lowest part of the crack in case it is vertical or at one end of the crack in case it is horizontal. The resin is injected till it is seen flowing from the opposite sides of the member at the corresponding port or from the next higher port on the same side of member. The injection port should be closed at this stage and injection equipment moved to the next port and so on. The smaller the crack, higher is the pressure or more closely spaced should be the ports so as to obtain complete penetration of the epoxy material throughout the depth and width of member. Larger cracks will permit larger port spacing, depending upon width of the member. This technique is appropriate for all types of structural elements, beams, columns, walls and floor units in masonry as well as concrete structures. Two items should however be taken care of in such type of repair:

(i) In the case of loss of bond between reinforcing bar and concrete, if the concrete adjacent to the bar has been pulverised to a very fine powder, this powder will dam the epoxy from saturating the region. So it should be cleaned properly by air or water pressure prior to injection of epoxy.

(ii) It has been stated that cracks smaller than about 0.75 mm may be difficult to pressure inject. So cracks smaller than this should not be repaired by this method.

Large cracks and crushed concrete

For cracks wider than about 6 mm or for regions in which the concrete or masonry has crushed, a treatment other than injection is indicated. The following procedure may be adopted.

(i) The loose material is removed and replaced with any of the materials mentioned earlier, i.e., expansive cement mortar, quick setting cement or gypsum cement mortar. (ii) Where found necessary, additional shear or flexural reinforcement is provided in the region of repairs. This reinforcement could be covered by mortar to give further strength as well as protection to the reinforcement. (iii) In areas of very severe damage, replacement of the member or portion of member can be carried out. (iv) In the case of damage to walls and floor diaphragms, steel mesh could be provided on the outside of the surface and nailed or bolted to the wall. Then it may cover with plaster or micro-concrete.

Repair Strategies

A number of options are available for giving a relief to a distressed structure, which could cover any of the following:

- Reduction of dead / live loads
- Repair / strengthening of columns, beams and slabs
- Improving the compressive strength of concrete\
- Attending to cracks and joints
- Improving the masonry structure to be able to resist earthquake forces
- Providing protective cover against the aggressive deteriorating chemicals

Stress Reduction: The reduction is another method of providing relief to the structure. This can be achieved by the following:

- Reducing dead load and live loads
- Replacing heavy solid partitions with lightweight partitions
- Enlarging openings by removing filler walls
- Reducing numbers of stories
- Changing the building use to a lower classification of building
- Span reduction of beams by providing struts, etc.

Repairing columns, beams and slabs: These form the basic structural elements in most of the building structural systems, which are deteriorated and require attention to improve the load carrying capacity. Their structural modification or strengthening would give the required relief to the structure and enhance its performance as under:

7.6.1. Columns: The strengthening of columns may be required for the following:

- ❖ Capacity: The load carrying capacity of the column can be enhanced by section enlargement. Different types of arrangement for section enlargement are shown in Figure 7.10.
- ❖ Ductility / confinement: The ductility of the column can be enhanced by providing additional tiles, steel tiles, steel plate bonding, and fibre wrap.
- ❖ Joints: The joints play crucial role for resisting earthquake forces. The joints can be strengthening by enlargement, jacketing by steel collar and fibre wrap.

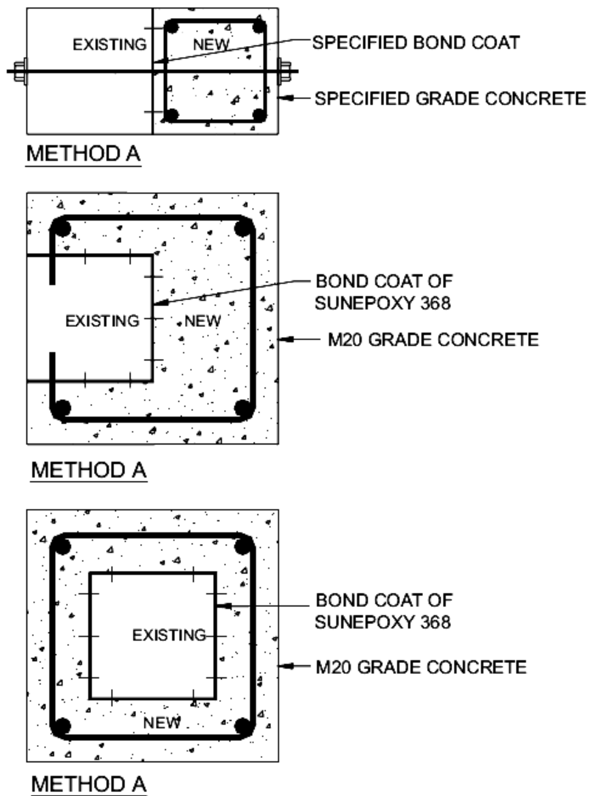


Figure 7.10. Column compressive Strengthening by section enlargement

7.6.2. BEAMS: These can be strengthened for:

i. Flexural strength: The flexural strength of the bean can be enhanced by (a) Section enlargement in compression. (b) Additional reinforcement in the tension (Figure 7.11. Caution shall be exercised to ensure that section is not over reinforced while providing additional reinforcement to compensate loss of reinforcement due to corrosion etc. (c) The provisioning for enhanced tensile strength if being undertaken, this should be accompanied with corresponding increase in compression as well. Due to such increased flexural capacities extra shear capacities required to ensure ductile behavior during earthquake shall also considered for provision. (d) MS plate bonding. (e) High strength fibre fabric wrap technique (without section enlargement)

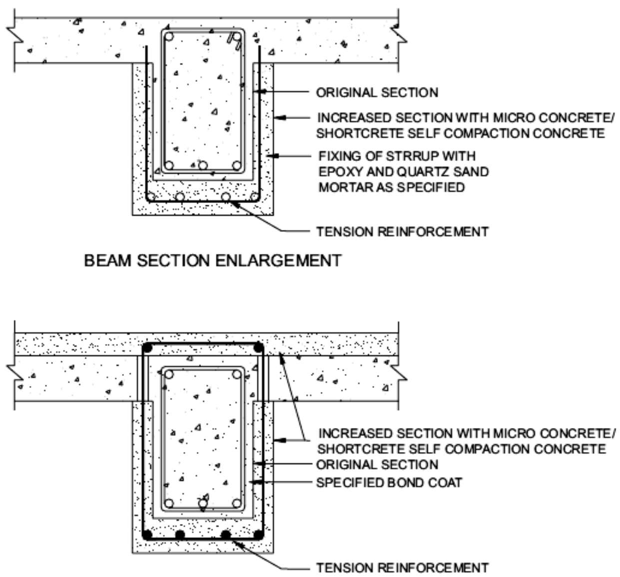


Figure 7.11. Beam Strengthening by concrete overlay and section enlargement

ii. Shear strength: The shear strength of the beam can be enhanced by any of the following:

(a) Section enlargement (b) Shear ties anchored in compression zone of beam (c) Post tension strap around the section (d) Diagonally anchored bolts (the holes are drilled perpendicular to the possible shear cracks) (e) MS steel plate bonding (f) Fibre wraps.

7.6.3. SLABS: The performance of the slab can be improved by providing overlays (in case of negative moment deficiency) or underlay (in case of positive moment deficiency) (Figure 7.12). The additional overlay/underlay will also increase the stiffness of the slabs and control the excessive deflections problems. The slabs are generally safe in shear and as such no need is likely to occur for shear strengthening except flat slabs near column capital.

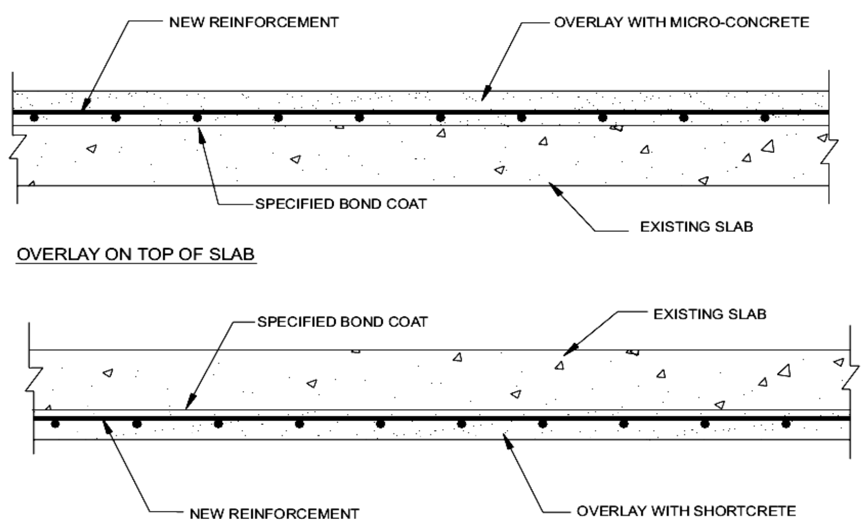


Figure 7.12. Slab strengthening: Concrete Overlay

7.7. STITCHING: The concrete and masonry are weak in tension. The cracks indicate the tensile failure of the material. The tensile strength of a cracked section can be restored by stitching in a manner analogous to sewing cloths. The crack is bridged with U shaped metal units called “stitching dogs” before being repaired with a rigid resin material. A nonshrink grout or an epoxy resin based adhesive should be used to anchor the legs of the dogs. The cracks should be sealed watertight before stitching. Stitching dogs should be of variable length and / or orientation and so located that the tension transmitted across the crack is not applied to a single plane within the section but is spread over an area. Since stress concentrations occur at the ends of crack, the spacing of stitching dogs should be reduced at such locations (Figures 7.13 and 7.14).

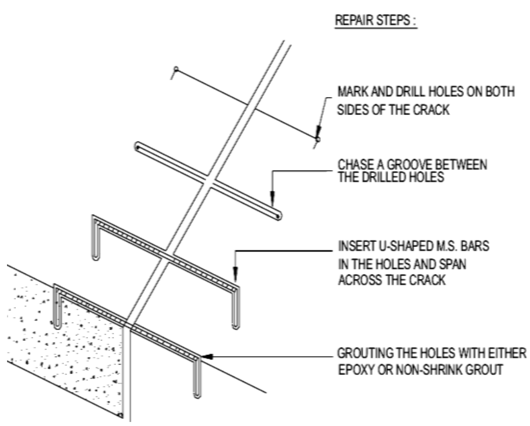


Figure 7.13. Stitching of wall / Slab

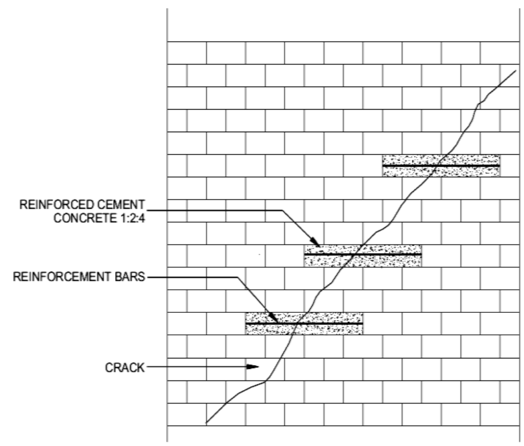


Figure 7.14. Stitching of brick masonry

7.8. Repair procedure for corrosion damaged elements

Step by step procedure for repairing the damaged columns / beams / slabs affected corrosion:

(A) Shotcreting: (Figure 7.15)

- ❖ Prop and support the structure in order to relieve the RC column of stresses due to load coming over it.
- ❖ Remove plaster and finishes all around the distressed RCC columns. Remove loose, cracked and spalled concrete to expose the rusted reinforcement.
- ❖ Remove concrete all around the reinforcement in order to get average 25mm air gap all around including behind the reinforcement and clean the reinforcement of concrete and rust by appropriate methods.
- ❖ Put additional reinforcement wherever the reinforcement diameter has been reduced by more than 15% with necessary overlap or welding with the existing reinforcement.
- ❖ Fix shear key bars of appropriate diameter at specified spacing in both directions over the surface to be covered with repair materials.
- ❖ Apply appropriate passivating and bond coat over the reinforcement and prepared RCC surface. Shotcrete the RCC column within the time limit

specified as pot life of the epoxy or tacking period of slurry. The necessary shuttering should be used for ensuring the desired thickness and shape of the columns.

- ❖ 6mm thick of finishing coat with cement sand plaster 1:3 if felt necessary, shall be applied within 48 hours of application of shotcrete repair.
- ❖ Wet curing shall be done over the finished surface of the shotcrete for a minimum period of 7 days.
- ❖ After RCC columns/beams are cured completely dried, a protective coating shall be applied over it for protecting the reinforcement and concrete.

(B) RCC Jacketing:

Step no. 1 to 5, same as above.

6. Appropriate passivating and bond coat shall be applied over the prepared surface.

7. Within the tacky period of bond coat, shuttering and concreting shall be done with specified grade of concrete with minimum cement content and water cement ratio more than 0.45. The consistency of this concrete shall be flowing and self-compacting, which shall be achieved by using super plasticizer.

8. Follow steps 6 to 8 as above.

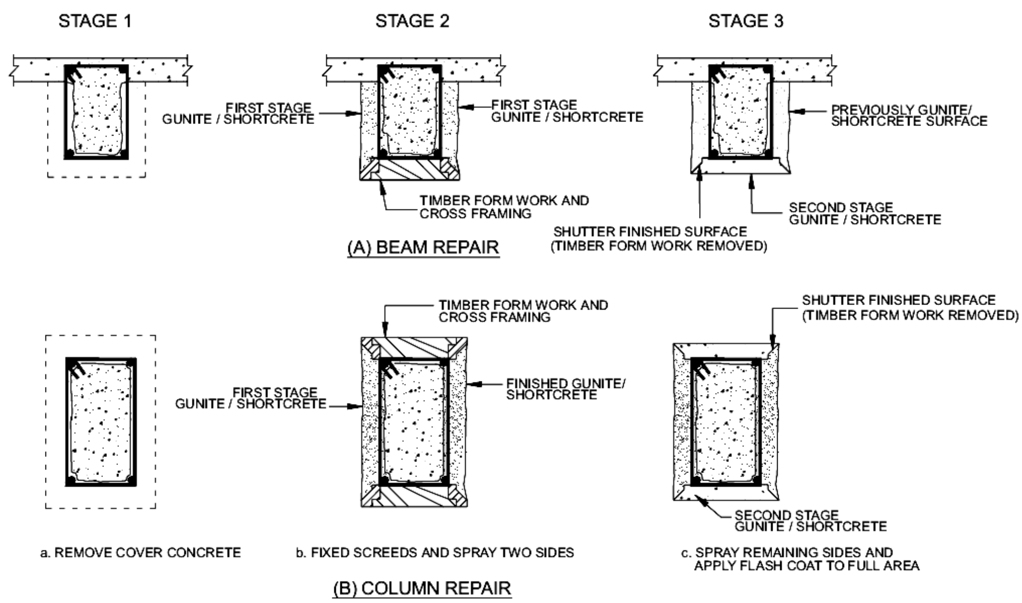


Figure 7.15. Column Repair by Guniting

Step by step procedure for repairing the damaged RCC slabs affected corrosion:

A. Repair with Polymer Modified Cement Mortar: (Figure 7.16)

1. Propping and supporting of RCC slab under distress.
2. All loose and spalled cover concrete shall be removed including finishing plaster wherever found loose by tapping.
3. The rusted reinforcement shall be cleaned of concrete preferably by using sand blasting to give a minimum 15mm clear air gap all around including behind the reinforcement.

4. Additional reinforcement wherever necessary shall be added and tied to the RCC slab with necessary binding wires and nails.
5. Fix shear key bars of appropriate diameter at specified spacing in both directions over the surface to be covered with repair materials.
6. The rusted reinforcement shall be cleaned of rust and passivated and applied bond coating.
7. The prepared concrete surface shall be covered with appropriate mix of polymer modified cement sand mortar in layers including behind reinforcement over a bond coat with polymer modified cement slurry. The mortar cover thickness shall be not less than 15mm over the reinforcement. The maximum thickness shall be not more than 30mm with each layer not exceeding 10mm.
8. Water curing shall be carried out for a minimum period of 7 days.

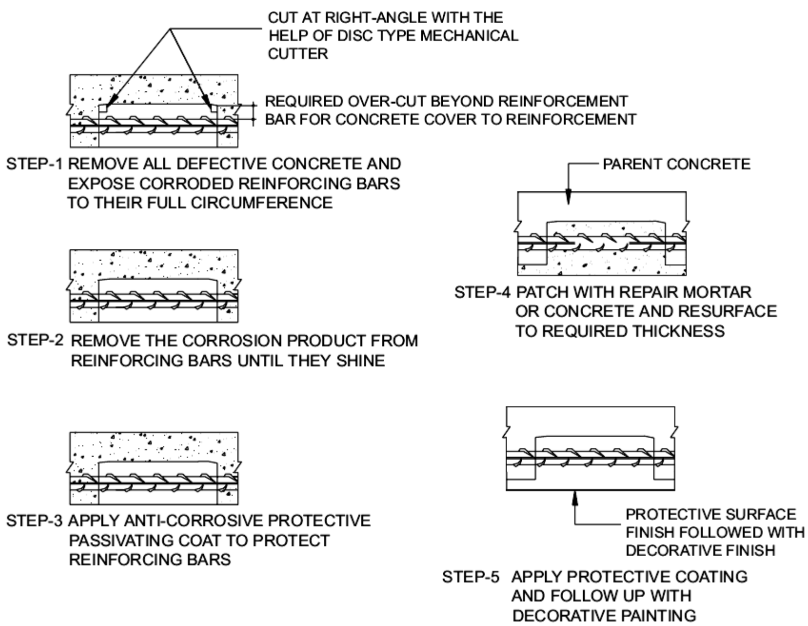


Figure 7.16. Repair of corrosion damaged slab

9. Necessary white washing/painting of the surface may be carried out after the curing period is over and the surface gets dried up.

(B) Shotcreting:

1. Step no. 1 to 6 shall be same as above.

7. Shotcreting with average thickness of 50mm shall be done within the tacking period of epoxy bond coat to be applied over the prepared surface of concrete.

8. Finishing plaster if necessary may be provided within 48 hours of shotcreting without allowing the RCC slab to become dry during the intervening period.

9. Water curing shall be carried out for a minimum period of 7 days.

7.9. TREATMENT OF DISTRESSED FLOOR IN TOILETS/KITCHEN: (Figure 7.17)

1. Remove all materials / flooring from the sunken floors and expose the drainage pipes/GI water supply lines.
2. Test the GI water supply lines less than 6 kg/m^2.
3. Test the drainage pipes and other joints for leakages, if any, by plugging of horizontal pipe at tee junction with vertical stack and filling with water upto the finished floor level for 48 hours.
4. Provide 40mm dia GI pipe spout and CC flooring with water proofing compound laid in slope (1:48 minimum) for draining out leaking water, if any, form the sunken portion in the shaft.
5. Provide 12mm thick cement plaster 1:3 mixed with water proofing compound on the vertical walls of the sunken portion including providing necessary repair around the drainage spout provided.
6. Provide two coats of bitumen coating in the sunken portion to ensure that the entire surface is properly covered and drainage pipes are also painted with bitumen.
7. Provide a 100 mm thick dry stone aggregate in the sunken portion. The mouth of the drainage spout provided earlier shall have graded filter so as not to get chocked.
8. Rest of the depth of sunken portion shall be filled with lean concrete 1:5:10 with stone aggregate and flooring laid to slope.
9. The rusted/leaking GI pipes wherever noticed during carrying out the repairs shall be replaced or otherwise be cleaned of rust and coated with polymer modified cement slurry and tow coats of bitumen painting.

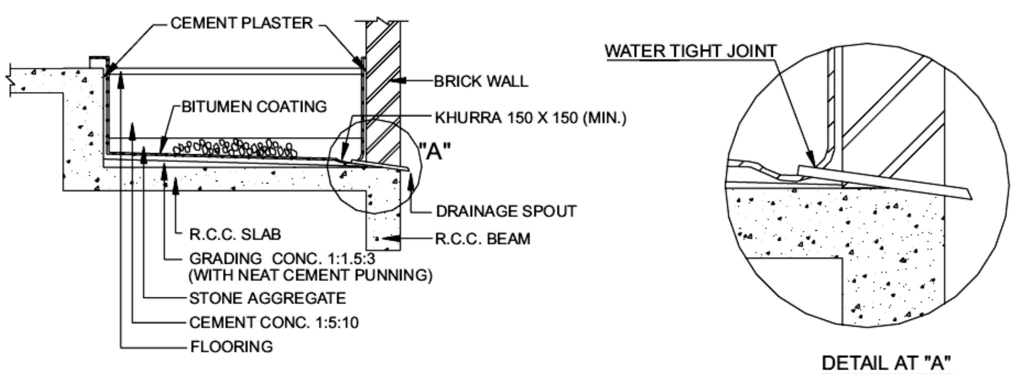


Figure 7.17. Repair of sunken slab of Toilets

7.10. Strengthening Solutions using FRP Plates:

Flexural Strengthening

The flexural capacity of member need to be enhanced either to withstand the additional increments of load beyond those for which the structures were originally designed, or to compensate the loss of capacity due to corrosion of the embedded steel reinforcement. Historically, RC members have been repaired by post - tensioning or jacking with new concrete in conjunction with a surface adhesive.

Since mid -1960s epoxy – bonded steel plates are being used to retrofit the flexural members. But corrosion may occur along the adhesive interface and affects the bond at the steel plate – concrete interface.

In the 1980s, fibre reinforced polymers (FRP) were developed and used in the form of thin laminates. They are constructed of high performance fibres such as carbon, aramid or glass, which are placed in a resin matrix. Selecting these fibres for particular application can alter the mechanical and durability properties.

The FRP laminates are being widely used for flexural strengthening because of their excellent properties including the following:

- High strength–to–weight ratio
- Low weight (making them much easier to handle on site)
- Immunity to corrosion
- Excellent mechanical strength and stiffness
- Unlimited availability in length
- Easy fabrication
- Possibility of bonding to non - flat surfaces
- Durability in adverse environments
- High fatigue strength

Moreover, bonding of FRP plating does not need expensive scaffolding. Many researchers have done experiments on RC beams strengthened with externally bonded FRP laminates to the tension face to exhibit ultimate flexural strength greater than their original/damaged beams. They indicated that the ductility of RC beams using externally bonded carbon fibre reinforced polymer (CFRP) and glass fibre reinforced polymer (GFRP) laminates gets reduced and the extent of reduction in ductility is dependent upon the characteristics of original.

One of the conventional methods for external strengthening implies the addition of adhesive-bonded steel plates on the tension side of the RC beams. The use of epoxy-bonded steel plates is very frequent in Europe and the United States but it suffers from a number of disadvantages:

Steel plates are heavy and difficult to transport, handle and install; the length of individual steel plates is restricted to 8-10m to enable handling and even at these lengths it may be difficult to erect them due to pre-existing service facilities; durability and corrosion effects remain uncertain; contaminants on structural members prior to bonding; surface preparation including the priming systems; steel plate thickness at least 5 mm to prevent distortion during blasting operation; complex profiles are difficult to be shaped with steel plates; expensive false work is required to maintain steel plates in position during bonding.

Composites fabricated either through wet processes on-site or prefabricated in plates (Figure.7.18) and then adhesively bonded to the concrete surface provide an efficient means of strengthening, that can be carried out with no or little disruption in use. The efficacy of the method depends mainly on the appropriate selection of the composite material and on the efficiency and integrity of the bond between the composite and the concrete surface.

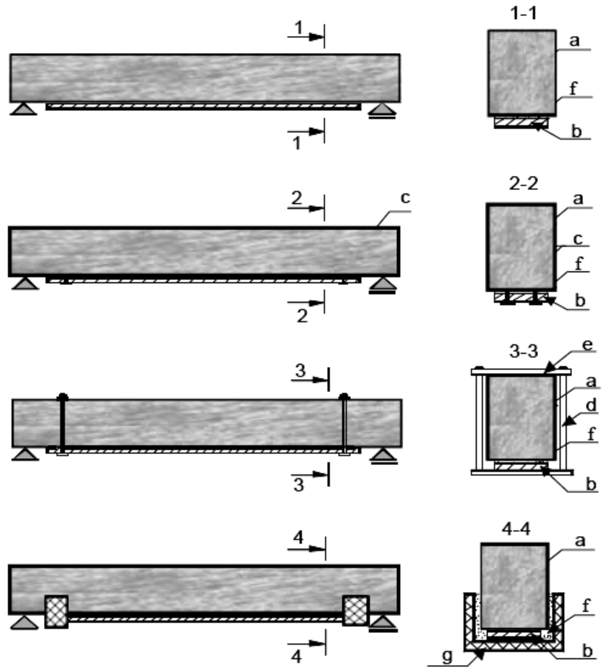
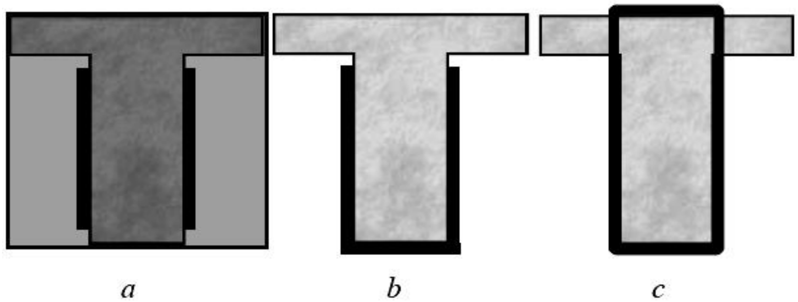


Figure 7.18. Flexural strengthening schemes with FRP composites

Shear Strengthening of beams

When a RC beam is deficient in shear, or when its shear capacity is less than the flexural capacity after flexural strengthening, the shear strengthening of the respective beam has to be considered. It has been realized that the FRP bonded to the soffit of a RC beam does not modify significantly the shear behaviour from that of the unstrengthened beams. Therefore, the influence of FRP strips bonded to the soffit for flexural strengthening may be ignored in predicting the shear strength of the beam. Various bonding schemes of FRP strips have been utilized to improve the shear capacity of reinforced concrete beams. The shear effect of FRP external reinforcement is maximized when the fibre direction coincides to that of maximum principal tensile stress. For the most common case of structural members subjected to transverse loads the maximum principal stress trajectories in the shear-critical zones form an angle with the member axis which may be taken about 450. However, sometimes it is more practical to attach the external FRP reinforcement with the principal fibre direction, perpendicular to the axis direction (Figure 7.19).

Because FRPs are strong in the direction of fibers only their orientation is recommended to control the shear cracks best. Shear forces in a beam may be reversed under reversed cyclic loading and fibers may be thus arranged at two different directions to satisfy the requirement of shear strengthening in both directions.



a **– FRP bonded to the web sides only;** ***b*** **– U jacketing;** ***c*** **– complete wrapping**
Figure 7.19. Shear strengthening schemes with FRP composites

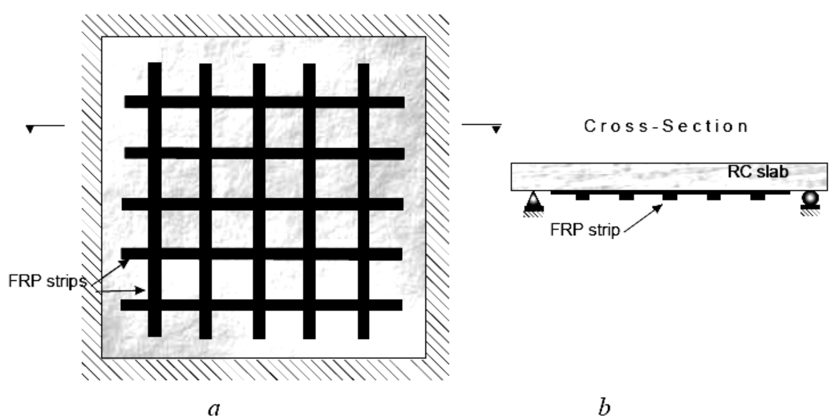
Strengthening of RC Slabs

When the RC slabs are simply supported the one-way slabs are strengthened by bonding FRP strips to the soffit along the required direction, Figure 7.20. For two-way slabs strengthening must be applied for both directions, by bonding FRP strips in both directions, Figure 7.21.

a – elevation; *b* – cross section

Figure 7.20. FRP strengthening of one-way simply supported slab:

The possible collapse mechanism of a two-way slab suggests that the strengthening of such a slab can be concentrated in the central region and the FRP strips can be terminated far away from the edges. The load capacity of such strengthened slabs can be predicted by a yield line analysis, as the part of the slab without bonded FRP strips has enough ductility for the formation of yield lines.

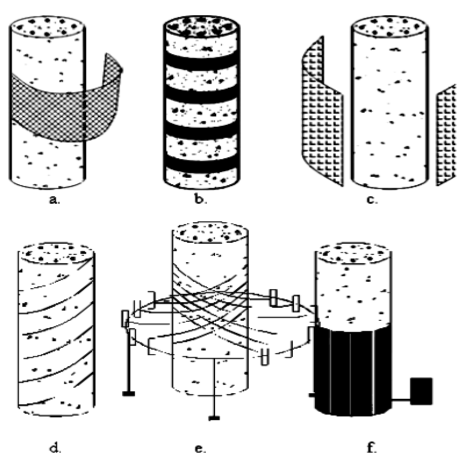


a **– slab soffit;** ***b*** **– cross section**
Figure 7.21. FRP strengthening of a two-way slab

Strengthening of RC Columns

Conventional strengthening measures for RC columns range from the external confinement of the core by heavily reinforced external concrete sections to the use of steel cables wound helically around the existing column at close spacing that are then covered by concrete and the use of steel jackets welded together in the field confining the existing columns. Some of these methods are effective but they have some disadvantages: they are time consuming and labour intensive; can cause significant interruption of the structure functioning due to access and space requirements for heavy equipment; rely on field welding, the quality of which is often questionable; susceptible to degradation due to corrosion; introduce changes in column stiffness, influencing the seismic force levels. The strengthening of existing RC columns using steel or FRP jacketing is based on a well established fact that lateral confinement of concrete can substantially enhance its axial compressive strength and ductility. The most common form of FRP column strengthening involves the external wrapping of FRP straps. The use of FRP composites provides a means for confinement without the increase in stiffness (when only hoop reinforcing fibers are utilized), enables rapid fabrication of cost effective and durable jackets, with little or no traffic disruption in most cases. A suitable classification of FRP composite jackets is given in Figure 7.22. In FRP-confined concrete subjected to axial compression, the FRP jackets are loaded mainly in hoop tension while the concrete is subjected to triaxial compression, so that both materials are used to their best advantages.

As a result of the confinement, both the strength and the ultimate strain of concrete can be enhanced, while the tensile strength of FRP can be effectively utilized. Instead of the brittle behaviour exhibited by both materials, FRP-confined concrete possesses an enhanced ductility. For FRP wrapped, axially loaded columns the design philosophy relies on the wrap to carry tensile forces around the perimeter of the column as a result of lateral expansion of the underlying column when loaded axially in compression. Constraining the lateral expansion of the column confines the concrete and, consequently increases its axial compressive capacity.



a **– wrapping of fabric;** ***b*** **– partially wrapping with strips;**
c **– prefabricated jackets;** ***d*** **– spiral rings;** ***e*** **– automated winding;** ***f*** **– resin infusion.**
Figure 7.22. Methods of FRP strengthening for RC columns

It should be underlined that passive confinement of this type requires significant lateral expansion of the concrete before the FRP wrap is loaded and confinement is initiated. In case of columns rectangular or square in cross section the confinement is effective at the column corners only with negligible resistance to lateral expansion being provided along the flat column sides. A number of different methods (based on form of jacketing material or fabrication process) have been tested at large or full-scale many of which are now used commercially all over the world.

7.11. Summary

Strengthening of RC members using jacketing, plate bonding are discussed in detail.

7.12. Keyword

Strengthening – jacketing – plate bonding – guniting shotcrete – wrapping.

7.13. Intext Questions

1. How will you repair cracks in RC elements using resins?
2. Explain about jacketing technique.
3. Explain flexural and shear strengthening using plate bonding technique.
4. Write short notes on ‘stitching’.
5. How will you repair corrosion demaged RC elements?

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MAINTENANCE OF STRUCTURES

Objective

❖ To study the maintenance of buildings in detail.

Contents

8.1. Introduction
8.2. Objective of maintenance
8.3. Maintenance Services
8.4. Classification of Maintenance
    8.4.1. Day-to-day Repairs
    8.4.2. Annual Repairs
    8.4.3. Special Repairs
    8.4.4. Additions and Alterations
    8.4.5. Preventive Maintenance
    8.4.6. Safety of Buildings
8.5. Summary
8.6. Key words
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8.1. Introduction

Building maintenance is work undertaken to keep, restore or improve every facility i.e. every part of a building, its services including horticulture operations to a currently acceptable standard and to sustain the utility and value of the facility.

8.2. Objective of maintenance

The objective of maintenance is: -

(i) To preserve machinery, building and services, in good operating condition.

(ii) To restore it back to its original standards, and

(iii) To improve the facilities depending upon the development that is taking place in the building engineering.

Inspite of recent improvements in building technology all the buildings deteriorate from the time they are completed. The rate of deterioration depends upon a number of factors. Not all the factors are under the control of the occupants.

During the design and construction stages, the following become essential:-

(i) Right choice of material.

(ii) Suitable construction techniques.

(iii) Adequate specifications for construction and installation work.

(iv) Effective supervision throughout construction and rectification of defects prior to final certification.

(v) Provision of adequate space for landscaping with proper design.

In fact the Government department contracts provide for obligatory maintenance by the original contractor in the initial stages for a period of six or three months, depending upon the nature of the work, immediately following the date of completion as there are bound to be teething troubles in any new construction. If these are attended to, the maintenance pressure will be reduced. Where there are inherent defects both in design and construction the maintenance cost raises disproportionately to a higher level and the anticipated life of building is reduced. Maintenance aims at effective and economic means of keeping the building and services fully utilizable. It involves numerous skills as influenced by occupancy and the performance level expected of a building. Programming of works to be carried out to keep the building in a good condition calls for high skills. Feedback from maintenance should also be a continuous process to improve upon the design and construction stages.

8.3. Maintenance Services:

These include primarily operations undertaken for maintaining proper condition of buildings, its services and works in ordinary use. The use for which buildings are designed is a prime factor in determining the requisite standard of care. Excessive maintenance should be avoided. At the same time, maintenance should ensure safety to the occupant or the public at large and should comply with the statutory requirements. The need also depends upon intensity of usage.

8.4. CLASSIFICATION OF MAINTENANCE

The repair works are classified in under mentioned categories:

- Day to day repairs / service facilities
- Annual repairs
- Special repairs

In addition to above the following works are also executed:

(a) Additions and Alterations Works in the buildings

(b) Supply & maintenance of furniture & furnishing articles,

8.4.1. Day to Day Repairs

Day to day repairs are carried out by the departments in all the buildings under its maintenance on the basis of day to day complaints received at the Service Centers. The works which are to be attended on day to day basis such as removing chokage of drainage pipes, man holes, restoration of water supply, replacement of blown fuses, repairs to faulty switches, watering of plants, lawn mowing, hedge cutting, sweeping of leaf falls etc. are attended under day to day service facilities. The purpose of this facility is to ensure satisfactory continuous functioning of various services in the buildings. These services are provided after receipt of complaint from the users at the respective Service Centers. Complaints of periodical nature like white washing, painting etc., which are usually got attended through contractors and cannot be attended on daily basis is transferred to register of periodical repairs.

8.4.2. Annual Repairs

To maintain the aesthetics of buildings and services as well as to preserve their life, some works like white washing, distempering, painting, cleaning of lines, tanks etc. are carried out periodically. These works are planned on year to year basis.

In addition, works such as patch repair to plaster, minor repairs to various items of work, replacement of glass panes, replacement of wiring damaged due to accident, replacement of switches, sockets tiles, Gap filling of hedges / perennial beds, Replacement / Replanting of trees, shrubs, painting of tree guards, planting of annual beds and trimming / pruning of plants etc., which are not emergent works and are considered to be of routine type, can be collected and attended to for a group of houses at a time and particular period of financial year, depending upon the exigency.

8.4.3. Special Repairs

Such works are undertaken to replace the existing parts of buildings and services which get deteriorated on ageing of buildings. It is necessary to prevent the structure & services from deterioration and restore it back to its original conditions to the extent possible. As the building ages, there is deterioration to the various parts of the building and services. Major repairs and replacement of elements become inevitable. It becomes necessary to prevent the structure from deterioration and undue wear and tear as u.ell as to restore it back to its original conditions to the extent possible.

The following types of works in general are undertaken under special repairs: -

- White Washing, Colour washing, distempering etc., after completely scrapping the existing finish and preparing the surface afresh.
- Painting after removing the existing old paint from various members.
- Provision of water proofing treatment to the roof.
- All the existing treatments known are supposed to last satisfactorily only for a period of about ten years.
- Repairs of internal roads and pavements.
- Repairslreplacement of flooring, skirting, dado and plaster.
- Replacement of doors, window frames and shutters. Replacement of door and window fittings .
- Replacement of water supply and sanitary installation like water tanks, WC cistern, Wash basins, kitchen sinks. pipes etc..
- Re-grassing of lawns/grass plots within 5-1 0 years.
- Renovation of lawn in 5-6 years.
- Replanting of hedges in 8- 10 years.
- Completely uprooting and removing hedges1 shrubbery.
- Replanting of
    - Rose beds in 5-6 years.

- Perennial beds in 5-6 years.
- Canna beds in 1-2 years.

❖ Shifting of any garden feature from one site to another within building.

The building services fixtures including internal wiring, water supply distribution system etc, are expected to last for 20-25 years. There afterwards it may be necessary to replace them after detailed inspection.

The expected economic life of the building under normal occupancy and maintenance conditions is considered to be as below:

(i) Monumental buildings 100 years.

(ii) RCC Framed construction *75* years

(iii) Load bearing construction *55* years.

(iv) Semi permanent structures 30 years

(v) Purely temporary structures *5* years

The life of the building mentioned above is only indicative and it depends on ~everalf actors like location, utilization, specifications, maintenance and upkeep / caretaking. The replacement, renovation and major repairs become inevitable as the life of all the components are not identical. All the three categories i.e. day to day, annual and special repairs/services are interrelated. Neglect of routine maintenance and preventive measures lead to more extensive periodical maintenance and in the long run major repair or restoration which could have been avoided or postponed.

8.4.4. Additions and Alterations

The works of additions/alterations are carried out in buildings to suit the special requirements of occupants for functional efficiency. Norms for facilities in govt. residential and non-residential buildings are revised from time to time. The facilities are updated by carrying out such works.

8.4.5. Preventive Maintenance

Preventive maintenance is carried out to avoid breakdown of machinery and occurrence of maintenance problems in buildings and services. Works of preventive maintenance are carried out on the basis of regular inspection / survey.

8.4.6. Safety of Buildings

All Buildings/structures are required to be inspected once a year by the Engineer in-charge to ensure that the building/structure is not unsafe for use. In case of electrical and other installations, the Engineer (Electrical) should inspect the same and record a certificate to that effect. The Junior Engineers are also required to inspect such structures/installations twice a year and record certificates to that effect. In case of any deficiency found in the structure/ installation necessary report should be made to higher authorities and immediate steps taken to get the same inspected by the Engineer and further action taken to remedy the defects. In case it is decided to demolish such unsafe building, it should be disposed off without land by auction under the powers vested in competent authorities.

8.4.5. Summary

The objective and classification of maintenance are studied in detail.

8.4.6. Keywords

Maintenance – Repair – Addition – Alteration – preventive measures.

8.4.7. Intext Questions

1. What are the objectives of maintenance of buildings?
2. Classify the maintenance of structures.

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