MODULE 1

INTRODUCTION

Time takes its toll on concrete structures which creates a real problem for the use of concrete in a country's infrastructure. Doing the right principles and procedures for the repair, rehabilitation and retrofitting of concrete structures is a critical element to financial success. Tearing down existing structures and rebuilding them up from the ground can be cost prohibitive and also against green building concept where we want to save energy for the future generation and even some time for our own generation also. Learning and perfecting the ways to make the most of existing structure are key elements when it comes to sustainable living and safe living conditions.

Many people look at concrete and see nothing but. But the knowledgeable mind sees much more. Are there any stress cracks in the surface? Were expansion joints provided properly? Does the colour of the concrete indicate a proper curing time? Is the surface as luck class like finish or brushed finish? Is the material flaking away? Can existing flaws be repaired in such a way to guarantee structural integrity?

Many people take concrete for granted. Yet, it is one of the strongest building materials of many bridges, highways and other significant infrastructure. Working with new structures of concrete is very different from repairing, rehabilitating and retrofitting of existing concrete structures. Both types of work have their rules of thumb and their engineering elements. It often requires more experience to repair concrete than it does to install it as a new construction. This is what will be learned in this subject.

Basic definitions

- **Repair:** The process of reconstruction and renewal of the existing buildings, either in whole or in part. Or in general worlds, restoration of a broken, damaged, or failed structure, equipment or part to an acceptable operating or usable condition or state.

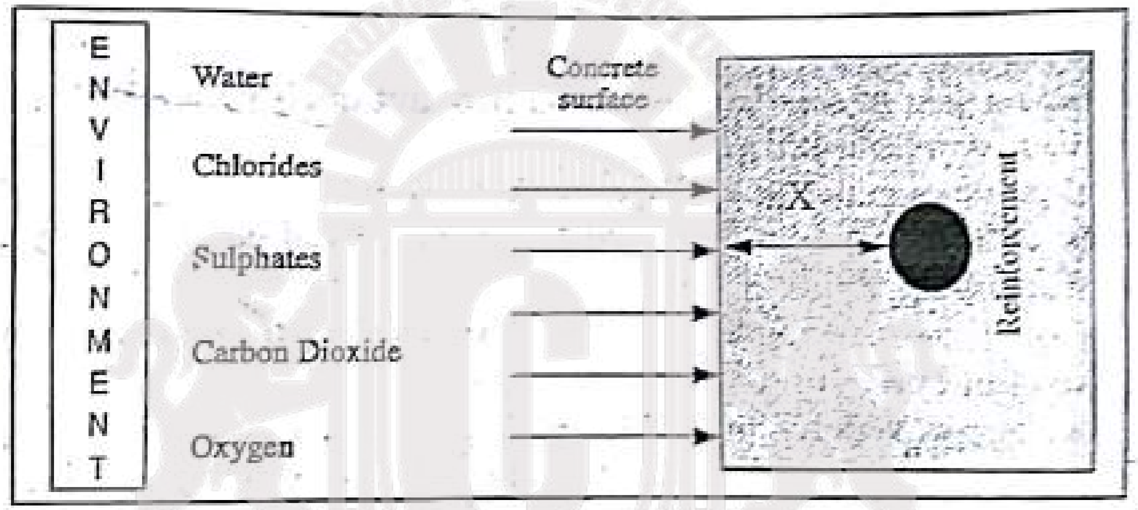
- **Retrofitting:** The process of strengthening of structure along with the structural system, if required so as to comply all relevant codal provisions in force during that period. **Or** in general words, provide something with a component or accessory not fitted during manufacturing.
- **Strengthening:** The process of increasing the load-resistance capacity of a structure or portion. **Or** in general worlds, to make something stronger or more effective.
- **Rehabilitation:** An upgrade required to meet the present needs – being sensitive to building features and a sympathetic matching of the original construction or the process of repairing or modifying a structure to a desired useful condition. **Or** in general worlds, the action of restoring something that has been damaged to its former condition.
- **Defects:** These are the flaws that are introduced through poor design, poor workmanship before a structure begins its design life or through inadequate operation and maintenance during its service life.
- **Renovation:** Process of substantial repair or alteration that extends a building's useful life.
- **Remodeling:** Essentially same as renovation – applied to residential structures.
- **Restoration:** The process of re-establishing the materials, form and appearance of a structure.
- **Demolition:** The process of pulling down of the structure not deemed to be fit for service.

Durability of Concrete

Concrete is said to be durable if it withstands the conditions for which it has been designed, without deterioration, over a period of years. **Or** Durability of concrete may be defined as the ability of concrete to resist weathering action, chemical attack, and abrasion while maintaining its desired engineering properties. The term durability of concrete is used to characterize the resistance of a concrete to a variety of physical or chemical attacks due to external or by internal causes.

The external causes may be due to weathering, occurrence of extreme temperature, abrasion, electrolytic action and attack by natural or industrial liquids and gases. The internal causes include the alkali aggregate reaction, volume change due to differences in thermal properties of aggregate and cement paste and above all the permeability of concrete.

Environmental penetrations which affect the durability of concrete are shown in figure.



Concrete will remain durable if:

- The cement paste structure is dense and of low permeability
- Under extreme condition, it has entrained air to resist freeze-thaw cycle.
- It is made with graded aggregate that are strong and inert
- The ingredients in the mix contain minimum impurities such as alkalis, Chlorides, sulphates and silt.

Factors influencing the durability of concrete

- **Water cement ratio and water content:** The added water can help easy workability and finish ability but concrete with added water shows segregation of aggregates and degradation of performance both in strength and durability. In the concrete with the same unit cement content, hydration can be more activated with larger unit water content.

- **Curing period:** It is very important to permit proper strength development aid moisture retention and to ensure hydration process occur completely.
- **Cement content:** Mix must be designed to ensure cohesion and prevent segregation and bleeding. If cement is reduced, then at fixed w/c ratio the workability will be reduced leading to inadequate compaction. However, if water is added to improve workability, water / cement ratio increases and resulting in highly permeable material.
- **Cover to reinforcement:** Concrete cover, in reinforced concrete, is the least distance between the surface of embedded reinforcement and the outer surface of the concrete. The concrete cover must have a minimum thickness for three main reasons.
    1. To protect the steel reinforcement bars (rebars) from environmental effects to prevent their corrosion.
    2. To provide thermal insulation, which protects the reinforcement bars from fire, and;
    3. To give reinforcing bars sufficient embedding to enable them to be stressed without slipping.
- **Aggregates:** Physical and mineralogical properties of aggregate must be known before mixing concrete to obtain a desirable mixture. These properties include shape and texture, size gradation, moisture content, specific gravity, reactivity, soundness and bulk unit weight. These properties along with the water/cementitious material ratio determine the strength, workability, and durability of concrete.
- **Mix design:** Concrete mix design is process of preparation of concrete with suitable proportion of ingredients to meet the required strength and durability of concrete structure. Every ingredient of concrete consists of different properties

so; it is not an easy task to get economical and good concrete mix. Wrong mix design affects the early strength as well as the durability of concrete.

- **Workability:** Workability of concrete is the property of freshly mixed concrete which determines the ease and homogeneity with which it can be mixed, placed, consolidated and finished.

  The strength of concrete decreases with increase in water cement ratio. The increase in water cement ratio indicates increase in workability of concrete. Thus, the strength of concrete inversely proportional to the workability of concrete so as the durability of concrete.

- **Admixtures:** admixtures are chemicals used to reduce the cost of concrete construction, to achieve certain properties in concrete more effectively than by other means, to maintain the quality of concrete during the stages of mixing, transporting, placing, and curing in adverse weather conditions and to overcome certain emergencies during concreting operations.

- **Compaction:** The concrete as a whole contain voids can be caused by inadequate compaction. Usually it is being governed by the compaction equipments used, type of formworks, and density of the steelwork.

- **Permeability:** It is considered the most important factor for durability. It can be noticed that higher permeability is usually caused by higher porosity. Therefore, a proper curing, sufficient cement, proper compaction and suitable concrete cover could provide a low permeability concrete.

Protective measures for durable concrete

- Proper concrete composition including special additions of admixtures.
- Proper reinforcement detailing including minimum concrete cover.
- Limiting or avoiding crack development.
- Provision of coatings as additional protective measures
- Inspection and proper monitoring procedure.

Distress in Concrete Structures

Distress in concrete members occurs with age due to corrosion in reinforcement, loading, settlement of foundations etc. This distress in building can be found by development of cracks in concrete members such as slabs, beams, columns etc.
The cracking of concrete in building is developed in three stages:

1. Stage – I: Volume of rust formed due to corrosion of reinforcement is increased about 2.5 times the volume of steel. With the result of corroded reinforcement bar presses the concrete outwards. Since concrete is poor in tension, longitudinal cracks are developed along the reinforcement bar.

2. Stage – II: Longitudinal cracks in RCC provide wide access to oxygen, carbon dioxide, and moisture with result excessive carbonation starts and structural damages starts. Fear in the mind of users starts.

3. Stage – III: In this stage cover comes out and causes danger to the life of structure and structure becomes unserviceable.

Causes of distress in concrete structures

- Structural deficiency arising out of faulty designing and detailing as well as wrong assumption in the loading criteria.
- Structural deficiency due to defects in construction, use of inferior and substandard materials.
- Damages caused due to fire, floods, tsunami and earthquakes, etc.
- Physical deterioration and creep (continuous deformation of concrete with time under sustained load).
- Chemical deterioration and marine environments.
- Damages caused due to abrasion, wear and tear.
- Damages due to impact, vibration, fatigue.
- Settlement of foundation, thermal expansion.

Remedial Measures for Distressed Concrete

1. **At design stage**

- Adequate cover to concrete should be planned.
- Adequate thickness of structural members particularly non-structural members e.g. chajjas, parapets, pergolas and fins etc. should be provided.
- Proper detailing of reinforcement especially at junctions should be designed to avoid congestion and to ensure smooth placement of concrete.

2. **At the construction stage**

- Concrete should be workable with minimum water cement ratio (< 0.45). It should be well compacted by vibrator. Thus we should try to achieve highest density with minimum void.
- Ensure proper grading and quality of aggregate free from deleterious material.
- Use potable water only.
- Leak proof and properly designed from work should be used. Ensure proper mixing, placement, compaction and curing of concrete. No segregation, honeycombing is allowed.
- Use of plasticizers and super-plasticizers to achieve workability for controlled water-cement ratio in placement of concrete in congested conditions.
- Binding wires to be turned inside (should not touch formwork). G.I. wires to be used in aggressive environment.
- Proper cover with dense concrete / mortar is must.
- Provision of drip course for projections should be made.
- Proper cement to be used to prevent sulphate and chloride attack.
- Cement not more than 3 months old should be used.

3. Protection of reinforcement bars

Protection to reinforcement bars against corrosion can also be provided by:

- **Using corrosion resistant steel:** Constituents of steel alloy are adjusted to resist corrosion.
- **Fusion bonded epoxy coating:** This process has good results in protecting reinforcement bars from corrosion. Powder epoxy is fusion bonded to bar at about 250^{0}C temperature. This is being used in coastal areas.
- **Passive coating with polymer based cement slurry:** In this process, reinforcement bars are cleaned for rust and freshly prepared polymer cement slurry is applied by brushes.
- **Protective epoxy coating:** Certain epoxy coating is also available in the market. Firstly rust is removed by wire brush or sand blasting. Thereafter, epoxy as per manufacturer's specifications are applied by spray / brush. Loss of bond may be up to 30%.

Deterioration of Concrete Structures

Deterioration: The process of becoming progressively worse.

Degrading or damaging of concrete due to external media is known a deterioration of concrete.

Reinforced concrete is a very versatile construction material. Properly designed concrete structures are both strong and durable. However, concrete structures are vulnerable to a number of factors that can cause deterioration. Deterioration can result in loss of strength and unsafe conditions. Therefore it is important to have an understanding of the vulnerabilities of concrete structures in order to help minimize long-term repair and maintenance costs.

A structure can be considered to have failed or damaged, not merely when it collapses, but also in cases when it fails to perform the functions for which it was designed.

Damaged to structures can be broadly classified as under:

- A total or partial collapse.
- Discolouration of materials.
- Cracking.
- Spalling of materials resulting in reduction in size of members.
- Deformation such as deflection, buckling, twisting and distortion.

Common causes for deterioration of concrete

- Accidental loading
- Chemical reactions
  - Acid attack
  - Aggressive water attack
  - Miscellaneous chemical attack
- Construction errors
- Design errors
  - Inadequate structural design
  - Poor design details
- Freezing and thawing
- Settlement and movement
- Shrinkage
  - Plastic
  - Drying
- Temperature changes
  - Internally generated
  - Externally generated
  - Fire
- weathering

Physical causes of deterioration of concrete

Shrinkage in Concrete

It may be defined as the volume changes in concrete due to loss of water or moisture caused by evaporation or hydration of cement. It may be divided into two general categories, that which occurs before setting (**plastic shrinkage**) and that which occur after setting (**drying shrinkage**). Shrinkage is caused by the loss of moisture from concrete.

Factors Affecting Shrinkage:

The shrinkage of concrete depends on several factors which are listed below.

1. Water-Cement Ratio: Shrinkage is mostly influenced by the water cement ratio of concrete. It increases with the increases in the water-cement ratio.

2. Environmental Condition: It is one of the major factors that affect the total volume of shrinkage. Shrinkage is mostly occurred due to the drying condition of the atmosphere. It increases with the decrease in the humidity.

3. Time: The rate of shrinkage rapidly decreases with time. It is found that 14-34% of the 20 years shrinkage occurs in two weeks, 40-80% shrinkage occurs in three months and the rest 66-85% shrinkage occurs in one year.

4. Type of Aggregate: Aggregates with moisture movement and low elastic modulus cause large shrinkage. The rate of shrinkage generally decreases with the increase of the size of aggregates. It is found that concrete made from sandstone shrinks twice than the concrete of limestone.

5. Admixtures: The shrinkage increases with the addition of accelerating admixtures due to the presence of calcium chloride (CaCl2) in it and it can be reduced by lime replacement.

6. Other Factors:

- The type and quantity of cement.
- Granular and microbiological composition of aggregates.
- The strength of concrete.
- The method of curing.
- The dimension of elements etc.

Types of Shrinkage in Concrete

- Plastic Shrinkage in concrete
- Drying Shrinkage in concrete
- Autogenous Shrinkage in concrete
- Carbonation Shrinkage in concrete

Plastic Shrinkage in concrete

Mechanism: 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 reasons of plastic shrinkage.

During the period between placing and setting, most concrete will exhibit bleeding to some degree. Bleeding is the appearance of moisture on the surface of concrete; it is caused by the setting of the heavier components of the mixture. Usually the bleed water evaporates slowly from the concrete surface. If environmental conditions are such that evaporation is occurring faster than water is being supplied to the surface by bleeding, high tensile stresses can develop. The stresses can lead to the development of cracks on the concrete surface. Typically the cracks are isolated rather than partner, these cracks are generally wide and shallow.

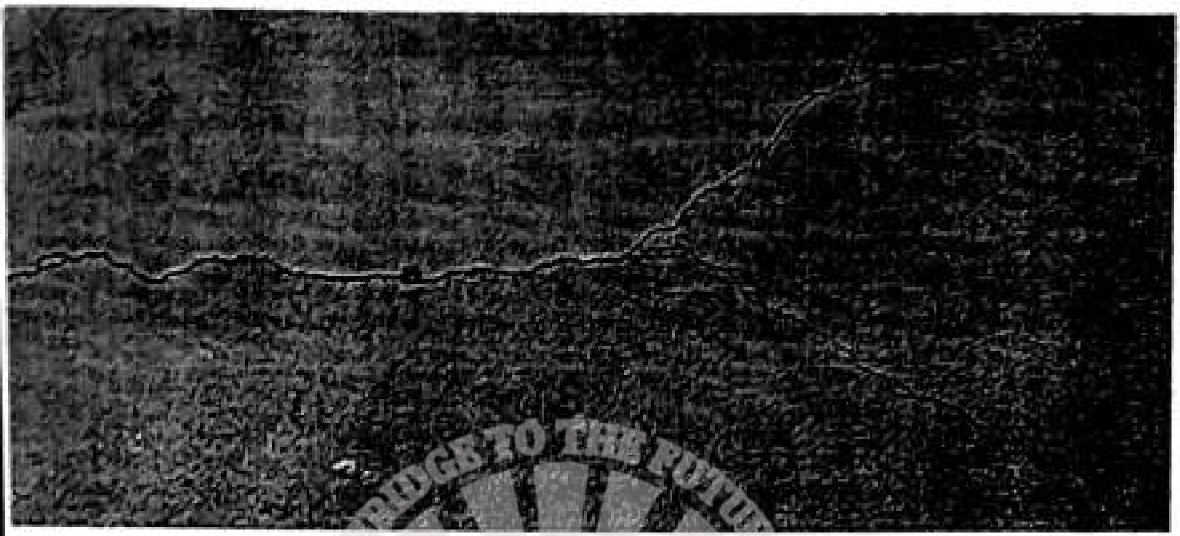


Figure: **Plastic shrinkage cracks**

The primary cause of plastic shrinkage cracks is the rapid evaporation of water from the surface. Plastic shrinkage cracks occur within few hours after placing the concrete while it is still in plastic state. Cracks occur almost entirely on horizontal surface exposed to the atmosphere. They can be deep and width varying from 0.1 mm to 3 mm. The magnitude of cracks depends upon ambient temperature, relative humidity and wind velocity.

Measures to reduce plastic shrinkage cracks

- Moisten the subgrade and formworks.
- Erect temporary wind breakers to reduce wind velocity over fresh concrete.
- Erect temporary roof to protect green concrete from hot sun.
- Reduce the time between placing and finishing.
- Minimize the evaporation by using fog spray and curing compound.

Drying Shrinkage in concrete

Mechanism: The shrinkage that appears after the setting and hardening of the concrete mixture due to loss of capillary water is known as drying shrinkage. Drying shrinkage generally occurs in the first few months and decreases with time. If this shrinkage could take place without any restraint, there would be no damage to the concrete.

This shrinkage causes an increase in tensile stress, which may lead to cracking, internal warping, and external deflection, before the concrete is subjected to any kind of loading. All Portland cement concrete undergoes drying shrinkage or hydral volume change as the concrete ages. Drying shrinkage can occur in slabs, beams, columns, bearing walls, pre-stressed members, tanks, and foundations.

Drying shrinkage is caused by the physical loss (evaporation) and chemical loss (hydration) of water during the hardening process and exposure to unsaturated air. The drying shrinkage cracks appear at about 7 to 10 days after concreting and around 80% of drying shrinkage occurs in about a year. Drying shrinkage is influenced by a number of factors such as cement content, water content, aggregates, curing, humidity and temperature.

Figure: Drying shrinkage cracks

Measures to reduce drying shrinkage cracks

- Use of minimum water content.
- Use of highest possible water content.
- Provide adequate and early curing.
- Eliminating the external restraints as much as possible.
- Providing sufficient close spaced reinforcement.
- Placing the concrete at as low a temperature as practical.
- Damping the subgrade and the forms.
- Damping the aggregates if they are dry and absorptive.
- Providing adequate contraction joints.

Autogenous Shrinkage in concrete

Autogenous shrinkage occurs due to no moisture movement from concrete paste under constant temperature. This shrinkage is caused by loss of water consumed up in the hydration of cement. The magnitude of this shrinkage is very small and is not much of significance and is a minor problem of concrete and can be ignored.

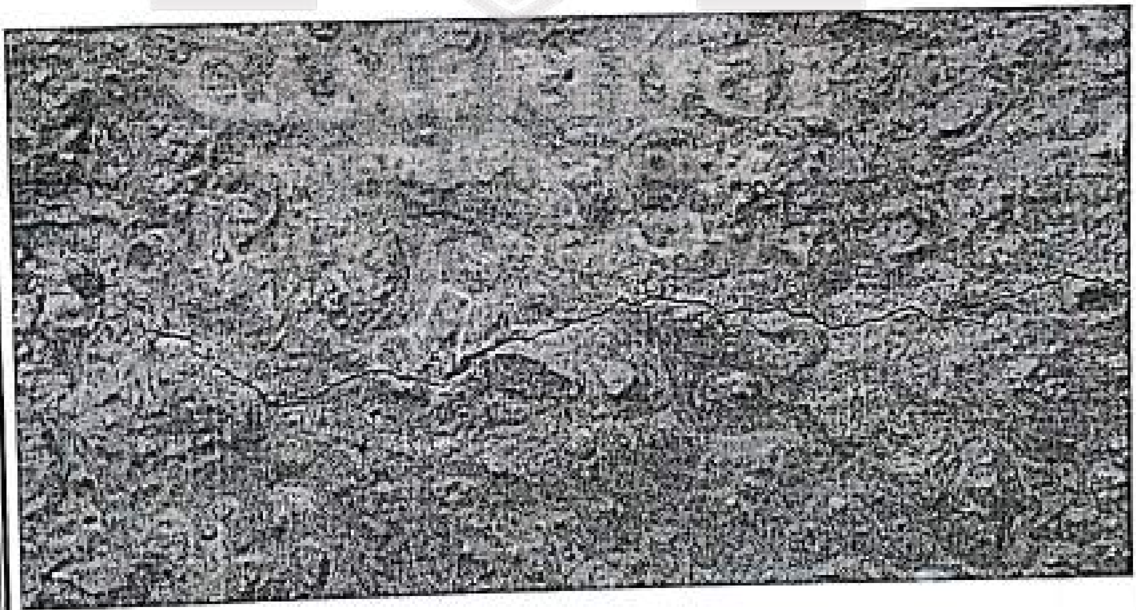


Figure: Autogenous shrinkage cracks

Carbonation Shrinkage in concrete

Carbonation is a process by which carbon dioxide from air penetrates into the concrete and reacts with calcium hydroxide to form calcium carbonates.

Carbonation shrinkage occurs due to the reaction of carbon dioxide (Co_2) with the hydrated cement minerals, carbonating $Ca(Oh)_2$ to $CaCo_3$ in the presence of moisture. The carbonation slowly penetrates the outer surface of the concrete. This type of shrinkage mainly occurs at medium humidity's and results increased strength and reduced permeability. The simultaneous reaction of Co_2 with hydrated cement minerals in concrete induces contraction of concrete.

Figure: Carbonation shrinkage cracks

Freeze and Thaw in Concrete

Freezing: Freezing is a phase transition in which a liquid turns into a solid when its temperature is lowered below its freezing point.

Thawing: Make (something) warm enough to become liquid or soft.

Freeze & Thaw cycle: When water freezes, it expands about 9 percent. As the water in moist concrete freezes it produces pressure in the pores of the concrete. The accumulative effect of successive freeze-thaw cycles and disruption of paste and aggregate can eventually cause expansion and cracking, scaling, and crumbling of the concrete.

Concrete is porous, so if water gets in and freezes it breaks off small flakes from the surface. This is typically called scaling and it can occur during the first winter and get worse over time. When severe, it can lead to complete destruction of the concrete.

Freeze-thaw disintegration or deterioration takes place when the following conditions are present.

- Freezing and thawing temperature cycles within the concrete.
- Porous concrete that absorbs water.

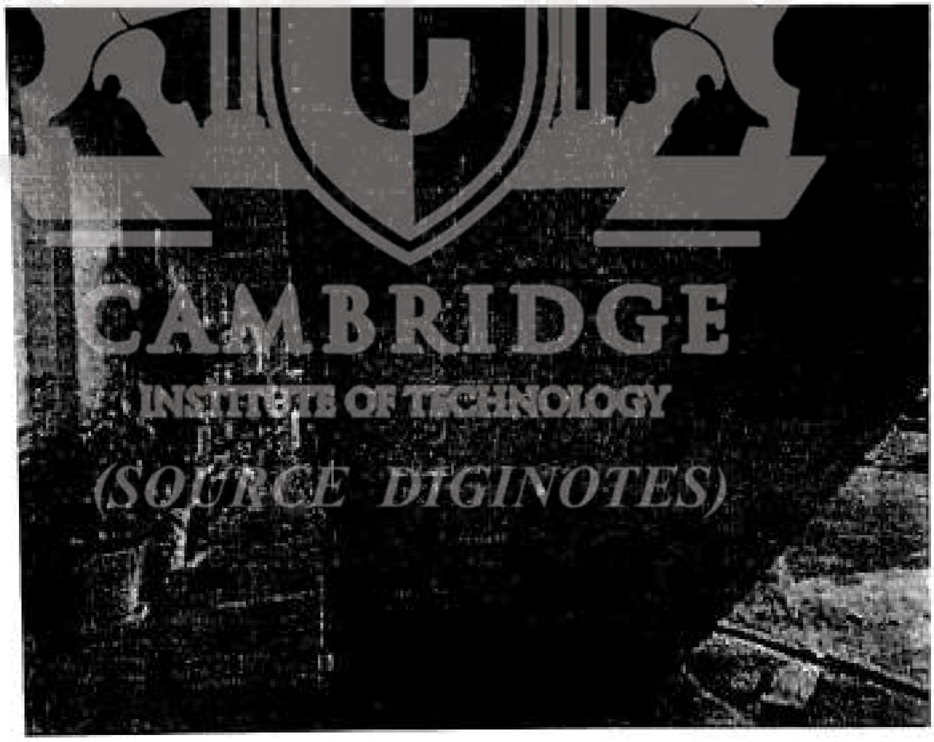


Figure: Cracked Stairs due to Freeze Thaw Cycles

Figure: Damage of Bridge due to Freeze Thaw Cycles

Figure: Pavement damage due to Freeze Thaw Cycles

Mechanism: The freezing water contained in the pore structure expands as it converts to ice. The expansion causes local tension forces that fracture the surrounding concrete matrix. The fracturing occurs in small pieces, working from outer surfaces inward.

Deterioration of concrete from freeze thaw actions may occur when the concrete is critically saturated, which is when approximately 91% of its pores are filled with water. When water freezes to ice it occupies 9% more volume than that of water. If there is no space for this volume expansion in a porous, water containing material like concrete, freezing may cause distress in the concrete. Distress to critically saturated concrete from freezing and thawing will commence with the first freeze-thaw cycle and will continue throughout successive winter seasons resulting in repeated loss of concrete surface.

Preventive measures

- Use of lowest practical water cement ratio.
- Usage of adequate of air entraining agent.
- Usage of durable aggregate.
- Adequate curing of concrete prior to exposure to freezing and thawing conditions.
- Providing positive drainage rather than flat surface.

Rate of freeze-thaw deterioration

- Porosity (increases rate)
- Moisture saturation (increases rate)
- Number of freeze-thaw cycles (increases rate)
- Air entrainment (reduces rate)
- Horizontal surfaces that trap standing water (increases rate)
- Aggregate with small capillary structure and high absorption (increases rate)

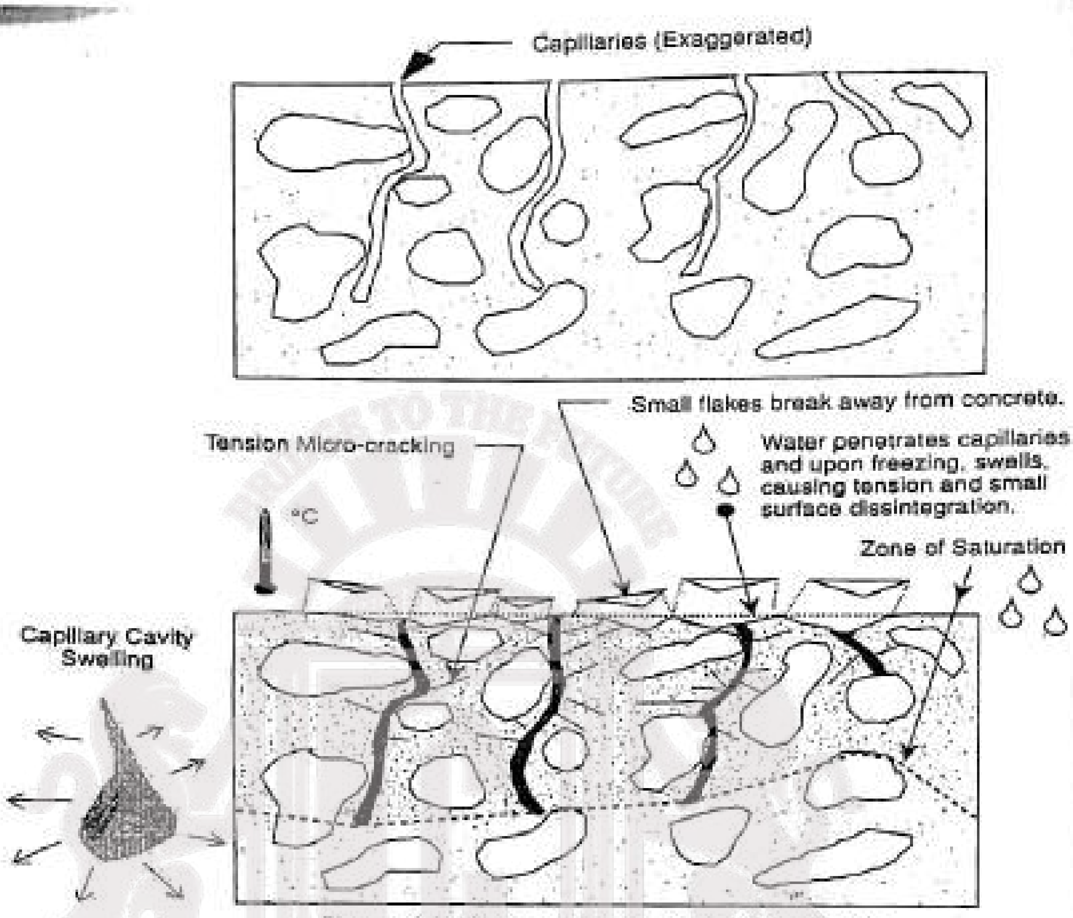


Figure: Freeze & Thaw mechanism

Symptoms

- Spalling and scaling of the surface
- Large chunks (cm size) are coming off
- Exposing of aggregate
- Usually exposed aggregate are un-cracked
- Surface parallel cracking
- Gaps around aggregate - in the ideal case

Weathering on concrete

Weathering: Wear away or change the appearance or texture of (something) by long exposure to the atmosphere.

Weathering is frequently referred as a cause of concrete deterioration. It defines as change in colour, texture, strength, chemical composition, and other properties of a natural or artificial material due to the action of weather.

The environmental factors that can cause cracking include: **freezing & thawing, wetting & drying, cooling & heating.**

- The damage from freezing and thawing is the most common weather related physical deterioration.
- If the volume changes due to these processes are excessive, cracks may develop and give impression that the concrete is on verge of deterioration.
- The damage due to these factors may appear in the form of general flacking and spalling of concrete from the surface.
- Concrete generally loses its strength with increase in temperature about 300^{0}c, damage being greater with aggregate having high coefficient of thermal expansion.

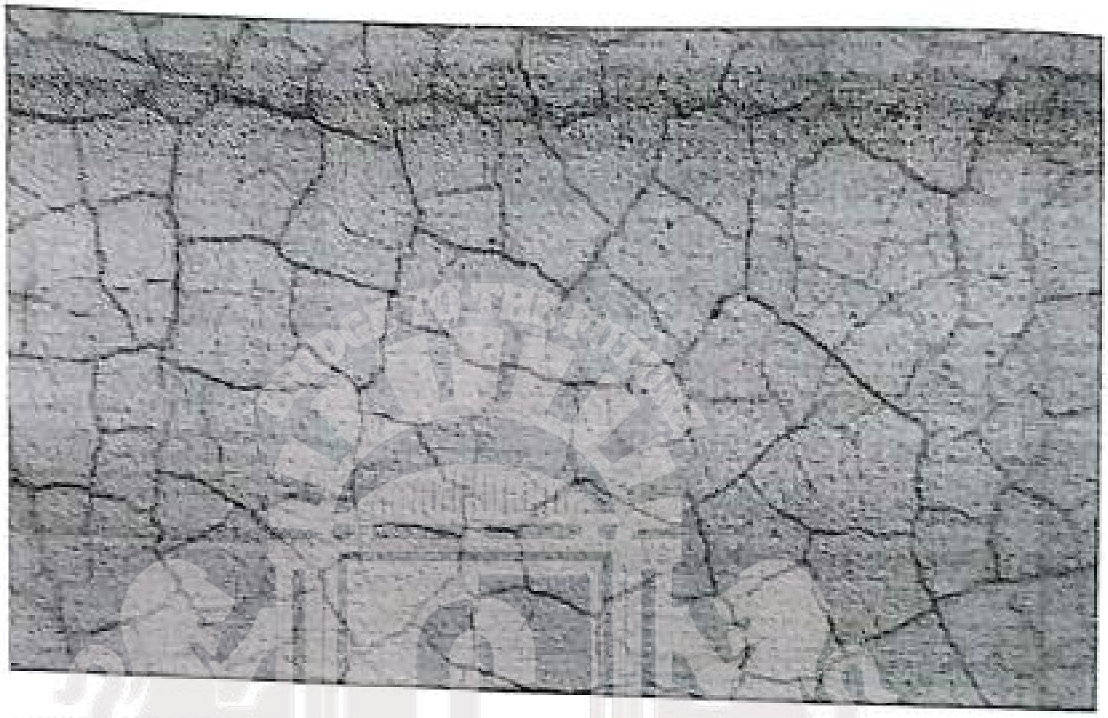
Crazing of concrete

Crazing is the development of a network of fine random cracks or fissures on the surface of concrete or mortar caused by shrinkage of the surface layer.

These cracks are rarely more than 3mm deep, and are more noticeable on over floated or steel- troweled surfaces. The irregular hexagonal areas enclosed by the cracks are typically no more than 40mm wide and may be as small as 10mm.

Generally, craze cracks develop at an early age and are apparent the day after placement or at least by the end of the first weak. Often they are not readily visible until the surface has been wetted and it is beginning to dry.

They do not affect the structural integrity of concrete and rarely do they affect durability. However crazed surfaces can be unsightly.



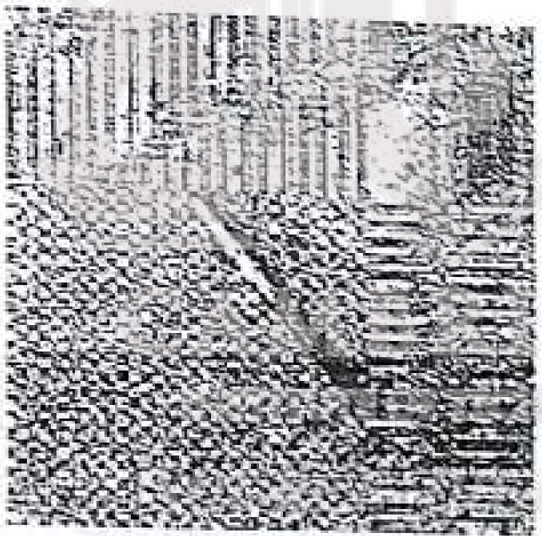


Fig. 1(a): Crazing

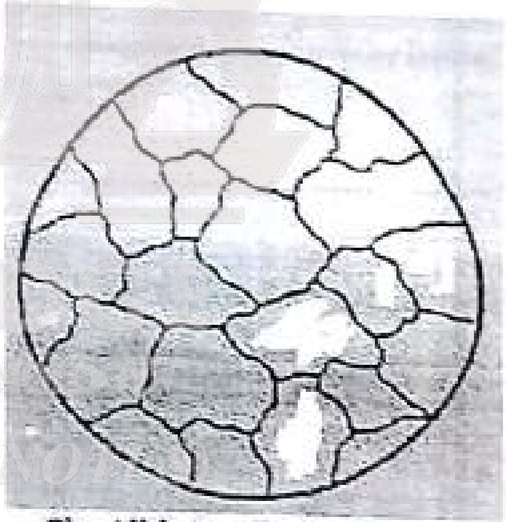


Fig. 1(b): Crazing Pattern

Figure: Cracks due to crazing

Causes of crazing in concrete

- Poor or inadequate curing
- Intermittent wet curing and drying
- Finishing with float when bleed water is on the surface.
- Spreading dry cement on a surface that is too wet to trowel and sprinkling water on concrete that is too dry to trowel, both produce skin likely to suffer from crazing.
- Use of highly absorptive aggregates.

Preventive measures

- Avoid over finishing the surface.
- Delaying toweling until the surface moisture has disappeared.
- Avoid sprinkling of dry cement or water on the surface during finishing operations.
- Starting curing as soon as possible.
- Avoiding higher temperature differentials between the concrete and curing water.
- Use of curing compound on the surface prevents the rapid evaporation of moisture from concrete surface and crazing is prevented.

Honeycombing on concrete

Honeycombing: When concrete is poured into the structural element forms it does not just flow in like water and fill up the forms to the top. If it is not vibrated properly it may leave voids called "honeycombing." The exposed aggregate leaves a honeycomb look and hence the name honeycombing of concrete.

Honeycombing is the term used to describe areas of the surface that are coarse and stony. It may be caused by insufficient fine material in the mix, perhaps due to incorrect aggregate grading or poor mixing.

This can be corrected by increasing the sand and cement content of the mix and by proper mixing, placing and compaction. Alternatively, honeycombing may be caused by leakage of grout or mortar fraction from the concrete at construction or formwork joints. The obvious solution here is to ensure that joints are well sealed and leak-free.

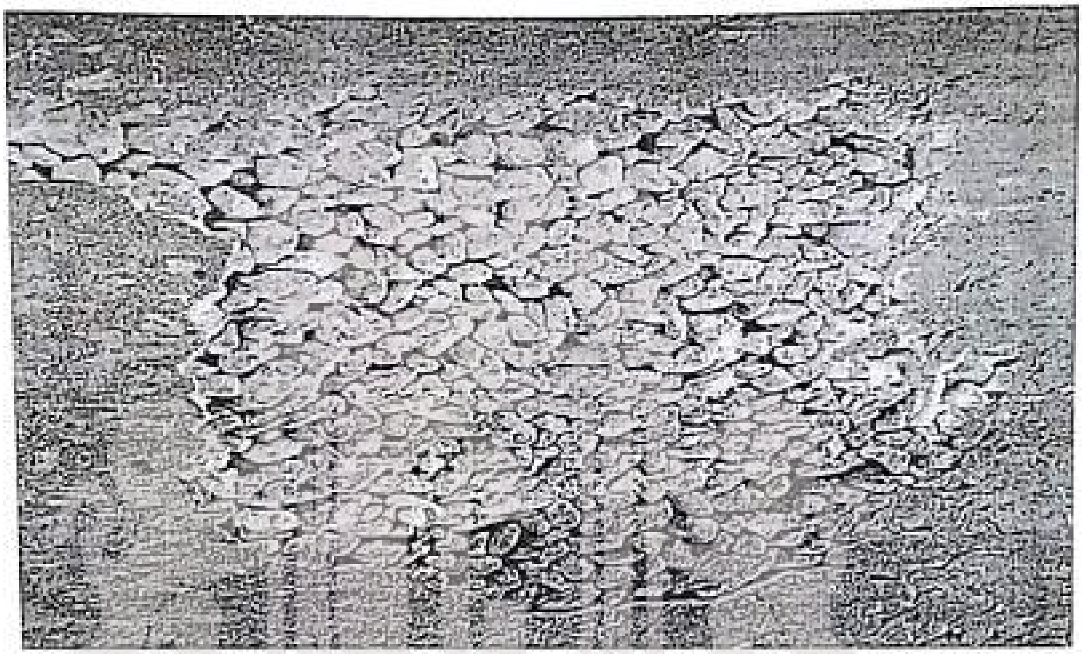


Figure: Exposed pockets of coarse aggregates

Preventive measures

- To follow good concrete construction practice.
- To use workable concrete.
- To provide good and tight forms.
- To give proper vibration.
- Maintaining proper water cement ratio.

Pop-outs on concrete

Pop-outs are usually caused by the expansion of porous aggregate particles having a high rate of absorption. As the offending aggregate absorbs moisture or freezes under moist conditions, its swelling creates internal pressures sufficient to scale the concrete surface.

A pop-out is a conical shaped hole in the surface with a portion of coarse aggregate particle exposed. These occur outdoors on the horizontal and vertical surfaces. They are caused by freezing of water in aggregate particles that have an internal pore structure which is causes them to expand unduly upon freezing.

Pop-outs don't appear during construction but they start to appear during the first winter and may continue to form for several years. These do not harm the concrete but they are unsightly. Rocks that produce pop-outs include: Chert, Shale, Clay stone, Mudstone, Argillaceous limestone, and Siltstone.

Preventive measures

- Pop-outs can be prevented only by avoiding aggregates which cause them.
- The use of air entraining admixtures, extra care in placement, consolidation, finishing, curing and sealing can all contribute to preventing moisture from reaching these aggregates.

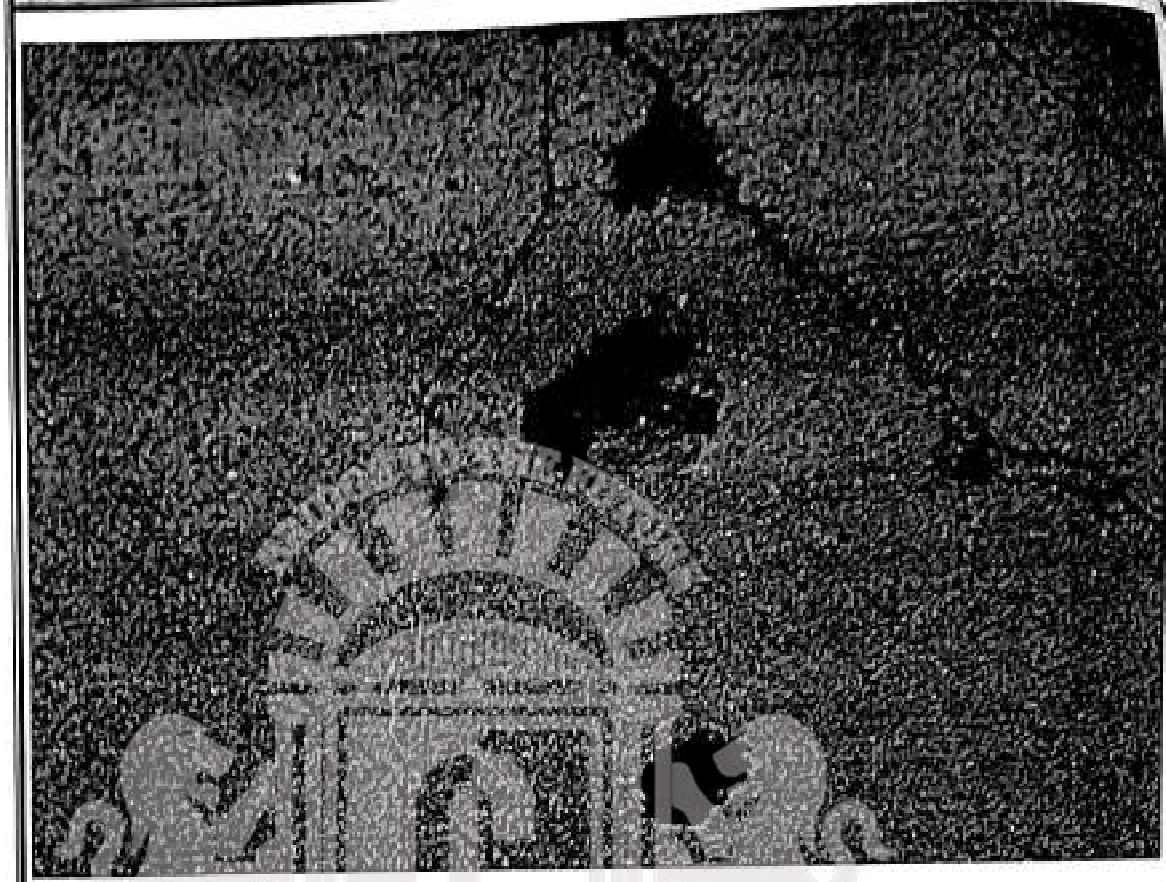


Figure: Pop-outs in concrete

Creep on concrete

Concrete when subjected to sustained loads not only undergoes instantaneous elastic deformation but also exhibit a time-dependent deformation known as creep.

Concrete creep is defined as deformation of structure under sustained load. Basically, long term pressure or stress on concrete can make it to change in shape. This deformation usually occurs in the direction the force is being applied. Like a concrete column getting more compressed, or a beam bending. Creep does not necessarily cause concrete to fail or break apart.

In concrete, creep results in a sustained increase in elastic deformation, which sometimes leads to formation of cracks in brick masonry of framed and load bearing structures. When the sustained load is removed, the strain decreases immediately by an amount equal to the elastic strain at the given age and is termed as instantaneous recovery.

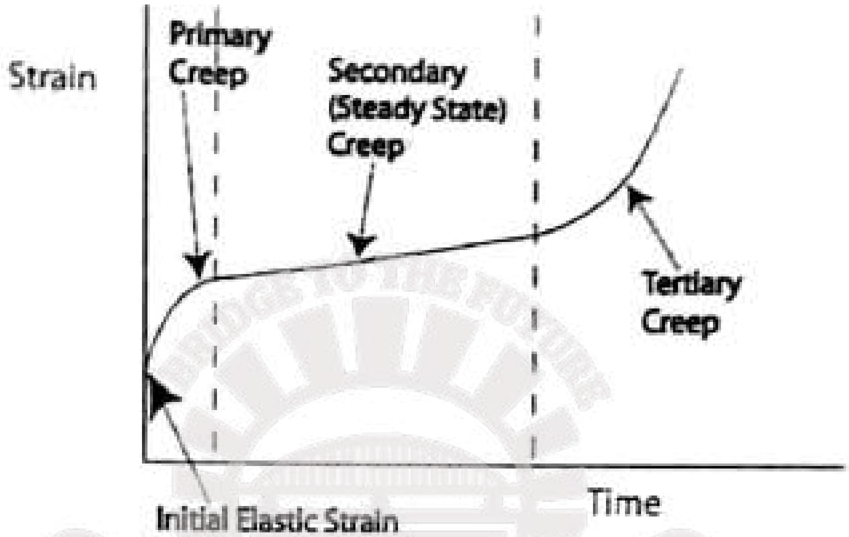


Figure: Creep in concrete

Factors Affecting Creep

Major factors

- **Influence of Aggregate:** Aggregate undergoes very little creep. It is really the paste which is responsible for the creep. However, the aggregate influences the creep of concrete through a restraining effect on the magnitude of creep. The paste which is creeping under load is restrained by aggregate which do not creep. The stronger the aggregate the more is the restraining effect and hence the less is the magnitude of creep. The modulus of elasticity of aggregate is one of the important factors influencing creep. It can be easily imagined that the higher the modulus of elasticity the less is the creep. Light weight aggregate shows substantially higher creep than normal weight aggregate.

- **Influence of Mix Proportions:** The amount of paste content and its quality is one of the most important factors influencing creep. A poorer paste structure undergoes higher creep. Therefore, it can be said that creep increases with increase in water/cement ratio. In other words, it can also be said that creep is inversely proportional to the strength of concrete. Broadly speaking, all other factors which are affecting the water/cement ratio are also affecting the creep.
- **Influence of Age:** Age at which a concrete member is loaded will have a predominant effect on the magnitude of creep. This can be easily understood from the fact that the quality of gel improves with time. Such gel creeps less, whereas a young gel under load being not so stronger creeps more. What is said above is not a very accurate statement because of the fact that the moisture content of the concrete being different at different age also influences the magnitude of creep.

Major factors

- Type of cement and cement content
- Entrained air
- Mixing time
- Level of sustained stress
- Ambient humidity
- Temperature
- Size of the specimen

Effects of Creep on Concrete and Reinforced Concrete

- Unwanted deflections in reinforced concrete beams
- In columns, creep in concrete will transfer greater load on to the reinforcing steel bars.
- In eccentrically loaded columns, creep increases the deflection and can lead to buckling.
- In mass concrete structures, creep accompanied by differential interior temperature conditions can cause cracking of the concrete.
- In pre-stressed concrete, creep reduces the pre-stressing magnitude.

Abrasion, Erosion & Cavitations on concrete

Progressive loss of mass from a concrete surface can occur due to abrasion, erosion and cavitations.

Abrasion refers to wearing away of the surface by friction.

Erosion refers to wearing away of surface by fruits

Cavitations refers to the damage due to non linear flow of water at velocity is more than 12 m/s.

The concrete used on roads, floors, pavements and for hydraulic structures should exhibit resistance against abrasion, corrosion and cavitations. More the compressive strength of concrete, higher is the resistance against abrasion, erosion and cavitations.

Abrasion: It is wearing away of the surface by dry attrition, repeated rubbing, rolling, sliding or frictional process. Surface abrasion is mainly caused by dry attrition as in pavements and industrial floors due to heavy trucking and vehicles. In hydraulic structures, the abrasion occurs due to the cutting action of water borne debris, the suspended solids in water i.e., rolling, sliding and grinding of debris suspended in water against the concrete structures. The factors that affect the abrasion resistance of concrete include compressive strength, aggregate properties, finishing methods, use of toppings and curing.

Erosion: It refers to manifestation of wear on the concrete surface by the abrasive actions of the suspended solid particles in fluids. The impinging, sliding, rolling action of suspended solid particles in water that come in contact with the concrete causes the surface wear. The rate of erosion depends on porosity and strength of concrete, duration of exposure, flow velocity of the water and its direction, and the amount, size, shape, density, hardness and velocity of the water borne debris.

Cavitations: It is the damage that is caused to concrete by action of high velocity water. Concrete generally shows excellent resistance to the latter, however, cavitations damage occurs when high velocity water-flows encounters discontinuities on the surface. Discontinuities, in the form of surface misalignment or abrupt change in slope, in the flow path cause the water to lift off the flow surface, creating negative pressure zones and resulting in the formation of bubbles of water vapour. These bubbles flow downstream with the water. On entering the region of high pressure, they collapse with great impact. Such high impacts can remove the particles of concrete, forming another discontinuity that creates more extensive damage. Cavitations damage results in erosion of the cement matrix, leaving harder aggregate in place.

Temperature Changes

Temperature Variation leads to Volume Changes in concrete. Resulting stresses lead to cracking, spalling and excessive deflections.

Basically, there are three temperature change phenomena that may cause damage to concrete.

- Temperature changes that are generated internally by the heat of hydration of cement in large placements.
- Temperature changes generated by variations in climatic conditions.
- Special case of externally generated temperature change-fire damage.

Internally generated temperature differences

The hydration of Portland cement is an exothermic chemical reaction. In large volume placements, significant amounts of heat may be generated and the temperature of the concrete may be raised by more than 38^0 Celsius over the concrete temperature at placement. Usually, this temperature rise is not uniform throughout the mass of the concrete and steep temperature gradients may develop.

These temperature gradients give rise to a situation known as internal restraint; the outer portions of the concrete may be losing heat while the inner portions are gaining. If the differential is great, cracking may occur. Simultaneously with the development of this internal restraint condition, as the concrete mass begins to cool, a reduction in volume takes place. If the reduction in volume is prevented by external conditions (such as by chemical bonding, by mechanical interlock) the concrete is extremely restrained. If the strains induced by the external restraints are great enough, cracking may occur.

Internally generated temperature differences

The basic failure mechanism in this case is same as for internally generated temperature differences; the tensile strength concrete is exceeded. In this case, the temperature change leading to the concrete volume change and is caused by external factors usually change in climatic conditions.

This cause of deterioration is best described by the following examples.

1. A pavement slab cast in summer. As the air and ground temperatures drops in winter, the slab may undergo a temperature drop. The slab would experience a shortening (shrinkage), if the slab were restrained, such movement would certainly lead to cracking.
2. A foundation or retaining wall that is cast in summer, as weather cools, the concrete may cool at different rates- exposed concrete will cool faster than that

insulated by soil or other backfill. The restrain provided by this differential cooling may lead to cracking if adequate construction joints are not provided.

3. Concrete that experience significant expansion during the warmer portions of the year, spalling may occur if there are no adequate expansion joints provided. In severe cases, pavement slabs may be lifted out of alignment, resulting in blowups.

4. Concrete that has been repaired or overlayed with materials that do not have the same coefficient of thermal expansion as the underlying material, annual heating and cooling may lead to cracking or de-bonding of the two materials.

Fire on concrete

A fire in a concrete structure causes damage. The extent of damage depends upon the intensity and duration of the concrete of the fire.

The principal types of damages are

- Reduction in strength of concrete.
- Cracking and spalling of concrete.
- Deflection and deformation of members.
- Discolouration.
- Other miscellaneous functional failures.

Concrete has good fire resistant properties. Fire resistant properties of concrete is determined by three main factors

- The capacity of concrete itself to withstand heat.
- The conductivity of the concrete to heat.
- The coefficient of thermal expansion of concrete.

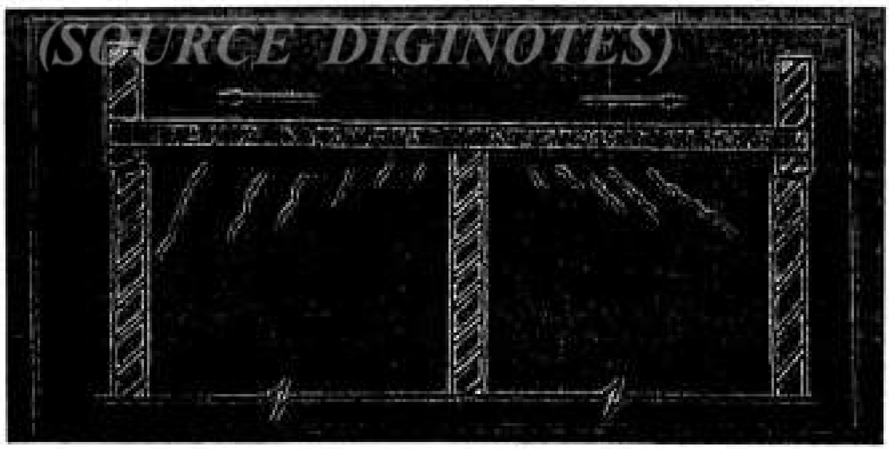
In the case of RC, the fire resistance is not only dependent upon the type of concrete but also the thickness of cover to reinforcement. The fire introduces gradients and as a result of it, the surface layers tend to separate and spall off from the cooler interior. The effect of increase in temperature on the strength of concrete is not much up to a temperature of about 250^0 c but above 300^0 c, definite loss of strength takes place.

Thermal movement in concrete

Thermal movement takes place due to the considerable amount heat liberated during hydration, and heat due atmosphere temperature and may also due to external fire. Because of thermal movement, the concrete undergoes changes in shape and volume and the same results in cracking of concrete structure. The extent of temperature rise depends on the properties of concrete and shape and size of the component. The heat of hydration may not be significant in mass concrete works it is an important factor to be contented with the control of heat and avoidance of cracks to maintain the integrity.

Preventive Measures

- Use of pozzolana.
- Use of low heat cement.
- Pre cooling of aggregates and mixing water.
- Post cooling of concrete by circulating refrigerated water through pipes embedded in the body of concrete.
- Provide joints to relieve the restraints in the structure.
- Providing adequate reinforcement to distribute the stresses.
- Providing suitable insulation cover.



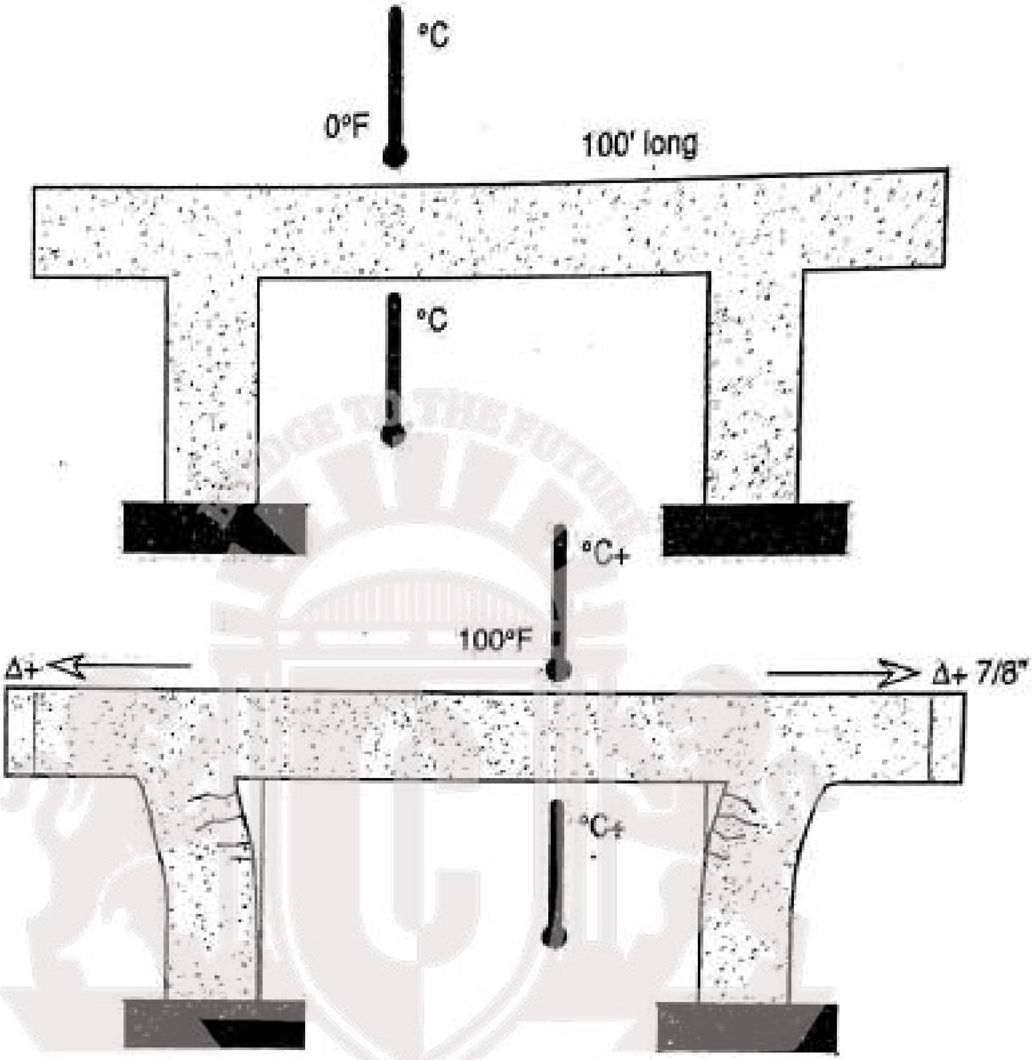


Figure: Thermal movement in concrete structure

Construction Errors

Construction errors during concreting at site may occur due to failure to follow specified procedures and good practice or outright carelessness.

Most of these errors may not lead to failure or deterioration of concrete, but they may have adverse impact on the structure with time.

The construction errors which are likely to occur at site with preventive measures of them are discussed in detail below. These errors not only occur during new construction, but may also happen during repair or rehabilitation works.

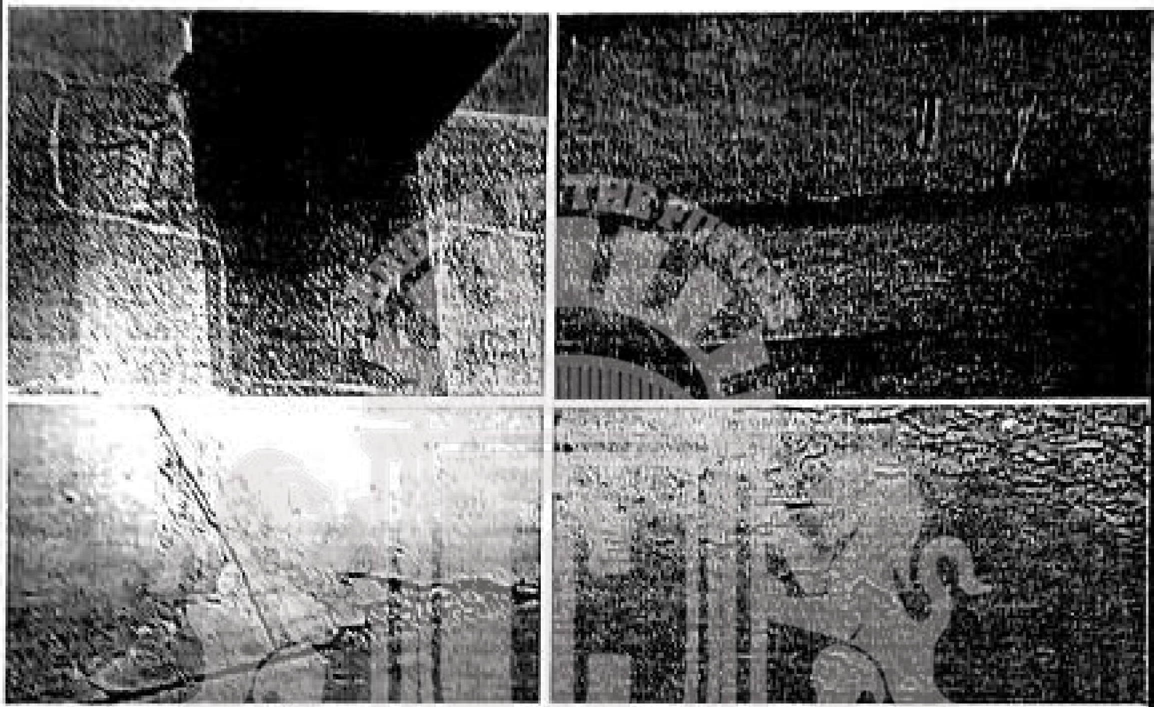


Figure: Construction errors

Adding water to concrete

Water is usually added to concrete in one or both of the following circumstances:

First, water is added to the concrete in a delivery truck to increase slump and decrease pouring or placement effort. This will lead to concrete with lowered strength and reduced durability. As the water/cement ratio of the concrete increases, the strength and durability will decrease.

In the second case, water is commonly added during finishing of structural member. This leads to scaling, crazing, and dusting of the concrete.

Improper alignment of formwork

Improper alignment of the formwork will lead to discontinuities on the surface of the concrete. While these discontinuities are unsightly in all circumstances, their occurrence may be more critical in areas that are subjected to high velocity flow of water, where cavitation-erosion may be induced, or in lock chambers where the "rubbing" surfaces must be straight.

Improper consolidation or compaction of concrete

Improper compaction of concrete may result in a variety of defects, the most common being bug holes, honeycombing, and cold joints.

Bugholes are formed when small pockets of air or water are trapped against the forms. A change in the mixture to make it less "sticky" or the use of small vibrators worked in the form has been used to help eliminate bug holes.

Honeycombing can be reduced by inserting the vibrator more frequently, inserting the vibrator as close as possible to the form face without touching the form, and slower withdrawal of the vibrator. Obviously, any or all of these defects make it much easier for any damage-causing mechanism to initiate deterioration of the concrete.

Frequently, a fear of over consolidation is used to justify a lack of effort in consolidating concrete.

Over consolidation is usually defined as a situation in which the consolidation effort causes all of the coarse aggregate to settle to the bottom while the paste rises to the surface. If this situation occurs, it is reasonable to conclude that there is a problem of a poorly proportioned concrete rather than too much consolidation.

Improper curing

Curing is probably the most abused aspect of the concrete construction process. Unless concrete is given adequate time to cure at a proper humidity and temperature, it will not develop the characteristics that are expected and that are necessary to provide durability. Symptoms of improperly cured concrete can include various types of cracking and surface disintegration.

In extreme cases where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

Improper location of reinforcing steel

This section refers to reinforcing steel that is improperly located or is not adequately secured in the proper location.

Either of these faults may lead to two general types of problems. First, the steel may not function structurally as intended, resulting in structural cracking or failure.

A particularly prevalent example is the placement of welded wire mesh in floor slabs. In many cases, the mesh ends up on the bottom of the slab which will subsequently crack because the steel is not in the proper location.

The second type of problem stemming from improperly located or tied reinforcing steel is one of durability. The tendency seems to be for the steel to end up near the surface of the concrete. As the concrete cover over the steel is reduced, it is much easier for corrosion to begin.

Movement of formwork

Movement of formwork during the period while the concrete is going from a fluid to a rigid material may induce cracking and separation within the concrete. A crack open to the surface will allow access of water to the interior of the concrete. An internal void may give rise to freezing or corrosion problems if the void becomes saturated.

Premature removal of shores or re-shores

If shores or re-shores are removed too soon, the concrete affected may become overstressed and cracked. In extreme cases there may be major failures.

Settling of the concrete

During the period between placing and initial setting of the concrete, the heavier components of the concrete will settle under the influence of gravity. This situation may be aggravated by the use of highly fluid concretes.
If any restraint tends to prevent this settling, cracking or separations may result. These cracks or separations may also develop problems of corrosion or freezing if saturated.

Settling of the subgrade

If there is any settling of the subgrade during the period after the concrete begins to become rigid but before it gains enough strength to support its own weight, cracking may also occur.

Vibration of freshly placed concrete

Most construction sites are subjected to vibration from various sources, such as blasting, pile driving, and from the operation of construction equipment.
Freshly placed concrete is vulnerable to weakening of its properties if subjected to forces which disrupt the concrete matrix during setting.

Improper finishing of flat concrete surface

The most common improper finishing procedures which are detrimental to the durability of flat concrete surface are discussed below:

- **Adding water to the surface:** Evidence that water is being added to the surface is the presence of a large paint brush, along with other finishing tools. The brush is dipped in water and water is "slung" onto the surface being finished.

- **Timing of finishing:** Final finishing operations must be done after the concrete has taken its initial set and bleeding has stopped. The waiting period depends on the amounts of water, cement, and admixtures in the mixture but primarily on the temperature of the concrete surface. On a partially shaded slab, the part in the sun will usually be ready to finish before the part in the shade.
- **Adding cement to the surface:** This practice is often done to dry up bleed water to allow finishing proceeding and will result in a thin cement-rich coating which will craze or flake off easily.
- **Use of tamper:** A tamper or "jitterbug" is unnecessarily used on many jobs. This tool forces the coarse aggregate away from the surface and can make finishing easier. This practice, however, creates a cement-rich mortar surface layer which can scale or craze. A jitterbug should not be allowed with a well designed mixture. If a harsh mixture must be finished, the judicious use of a jitterbug could be useful.
- **Jointing:** The most frequent cause of cracking in flatwork is the incorrect spacing and location of joints.

Cracking due to construction overloads

The loads induced during construction can be far more severe that those experienced in service. Unfortunately, these conditions in the early ages when the concrete is most susceptible to damage and often results in permanent cracks. A common error occurs when the precast members are not properly supported during transportation and erection. The use of arbitrary or convenient lifting points may cause severe damage. A big element lowered too fast, and stopped suddenly carries significant momentum, which is translated into an impact load that may be several times the dead weight of the element.

Storage of materials and equipment can easily result in loading conditions during construction for more severe that any load for which the structure is designed. Damage from unintentional construction overloads can be prevented only if the designers provide information on load limitations for the structure.

Causes

- Inadequate provision of main steel reinforcement, or inadequate provision of temperature reinforcement or wrong spacing of bars, or absence of corner reinforcement may cause unacceptable cracks in concrete.
- One of the most common occurrences is the displacement of top bars in cantilever chajjas, by the movement of concreting gang, causes cracks at the junction point of the cantilever chajjas.

Design errors

Design errors may be divided in to two general types

- Those resulting from inadequate structural design
- Those resulting from lack of attention to relatively minor design details.

Inadequate Structural Design

Failure mechanism: when the concrete is exposed to greater stress than it is capable of carrying or it sustains greater strain than its capacity.

Symptoms:

- First, errors in design resulting in excessively high compressive stress will result in spalling. Similarly high torsion or shear stress may also result in spalling or cracking.
- Second, high tensile stresses will result in cracking.

Prevention: Through careful review of all design calculations.

Poor design details

Poor design detailing may allow water to pond on a structure, resulting in saturated concrete.

Poor detailing does not lead directly to concrete failure; rather it contributes to the action of one or the other causes of concrete deterioration.

Several specific type of poor detailing and their possible effect on concrete are:

Abrupt changes in section

Abrupt changes in section may cause stress concentrations that may result in cracking.

Typical examples would include the use of relatively thin sections such as bridge decks rigidly tied into massive abutments and re-placement concrete that are not uniform in plan dimensions.

Insufficient reinforcement at re-entrant corners and openings

Re-entrant corners and openings also tend to cause stress concentrations that may cause cracking. In this case, the best prevention is to provide additional reinforcement in areas where stress concentrations are expected to occur.

Inadequate provision for deflection

Deflections in excess of those anticipated may result in loading of members or sections beyond the capacities for which they were designed.

Inadequate provision for drainage

Poor attention to the details of a structure may result in pounding of water. This ponding may result in leakage or saturation of concrete. Leakage may result in damage to the interior of the structure or in staining on the structure. Saturation may result in severely damaged concrete if the structure is in an area that is subjected to freezing and thawing.

Insufficient expansion joints

Inadequately designed Expansion joints may result in spalling of concrete adjacent to the joints. The full range of possible temperature differential that a concrete may be expected to experience should be taken into account in the specification for expansion joints.

Incompatibility of materials

The use of materials with different properties (modulus of elasticity or coefficient of thermal expansion) adjacent to one another may result in cracking or spalling as the structure is loaded or as it is subjected to daily or annual temperature variations.

Rigid joints between precast units

Designs utilizing precast elements must provide for movement between adjacent precast elements or between the precast elements on the supporting frame. Failure to provide for this moment can result in liking our spalling.

The design & detailing errors that may cause an acceptable cracking as follows

- Improper selection and / or detailing of reinforcement..
- Use of poor detailed re-entrant corners in walls, precast members and slabs.
- Restraint of members subjected to volume changes due to variation in temperature and moisture
- Lack of adequate contraction joints.
- Improper design of foundations results in differential settlement within the structure.
- Re-entrant corners provide a location for stress concentration and these are main locations for initial cracks, as in case of windows and door openings in concrete walls and beams.

Chemical causes of deterioration of concrete

Chemical attack on the concrete

Chemical attack is the reaction of chemical elements from exposure and moisture present in the concrete which results into deterioration of RCC structure.

Ingress of dissolved substances from the external environment may cause various forms of chemically induced deterioration by reaction with the cement paste or aggregate constituents.

Chemical attack on concrete can be classified as

- Carbonation
- Acid attack
- Alkali attack
- Chloride attack
- Leaching
- Salt attack
- Sulphate attack

Ingress of dissolved substances from the external environment may cause various forms of chemically induced deterioration by reaction with cement paste or aggregate constituents.

Resistance of concrete to chemical attack is directly influenced by:

- Its porosity.
- The cement composition used in the concrete.
- Conditions under which the cement paste hardened.
- Properties of concrete.
- Ability to resist various effects of fluids in the environment.

The penetration may be influenced by the effects of

- Temperature gradient.
- Freezing.
- Loading.
- Electric current & other factors.

Most of the problems due to chemical attack arise from the aggressive fluid penetrating into the interior pore space in the concrete. Therefore, damage is in many cases dependent on the permeability of the surface layers and not on the body of the concrete.

Types of chemical reactions

Extensively five main types of chemical reactions can be recognized

- **Type 1:** Simple leaching of free calcium hydroxide (Hydrated lime).
- **Type 2:** Reaction between attacking solutions and cement compounds resulting in the formation of secondary compounds, which are either leached from the concrete, or remain in a non bound form, resulting in gradual strength loss.
- **Type 3:** Reaction similar to type 2, but resulting in the crystallization of the reaction products giving rises to expansive forces, which disrupt the concrete.
- **Type 4:** Crystallization of salts directly from the attacking solution causing disruption of the concrete.
- **Type 5:** Corrosion of the embedded steel reinforcement.

Carbonation on the concrete

Carbonation: Carbonation is the formation of calcium carbonate ($CaCO_3$) by a chemical reaction in the concrete.

The creation of calcium carbonate requires three equally important substances: carbon dioxide (CO_2), calcium phases (Ca), and water (H_2O). Carbon dioxide (CO_2) is present in the surrounding air, calcium phases (mainly Ca $(OH)_2$ and CSH) are present in the concrete, and water (H_2O) is present in the pores of the concrete.

The first reaction is in the pores where carbon dioxide (CO2) and water (H2O) react to form carbonic acid (H_2CO_3):

$$CO_2 + H_2O \rightarrow H_2CO_3$$

The carbonic acid then reacts with the calcium phases:

$$H_2CO_3 + Ca(OH)_2 \rightarrow CaCO3 + 2 \cdot H_2O$$

Once the Ca (OH)2 has converted and is missing from the cement paste, hydrated CSH (Calcium Silicate Hydrate - CaO•SiO2•H2O) will liberate CaO which will then also carbonate:

$$H_2CO_3 + CaO\ CaCO_3 + H_2O$$

Carbonation of concrete is one of the reasons for corrosion of reinforcement. It is a process by which carbon dioxide from air penetrates into the concrete and reacts with calcium hydroxide to form calcium carbonates.

Mechanism

- The percentage of CO_2 present is air vary from place to place. In case of rural areas the concentration of CO_2 in air may be about 0.03% by volume, where as in urban areas it may vary from 0.3% to 1.0%. This CO_2, in presence of moisture changes into dilute carbonic acid and attacks the concrete and reduces alkalinity of concrete.

- Due to reduction of alkalinity of concrete, the pH value of pore water in the hardened cement paste reduces from 13 to 9.0. When all the Ca $(OH)_2$ has become carbonated, the pH value again reduces to about 8.3. And at this low pH value, the protective layer gets destroyed and the steel is exposed to corrosion.

In good quality concrete, the carbonation process is slow. Lesser is the porosity lesser will be the penetration of CO_2. The carbonation process requires constant change in the moisture levels (dry to damp to dry). The process does not occur when concrete pores are filled with water – or when concrete is constantly underwater.

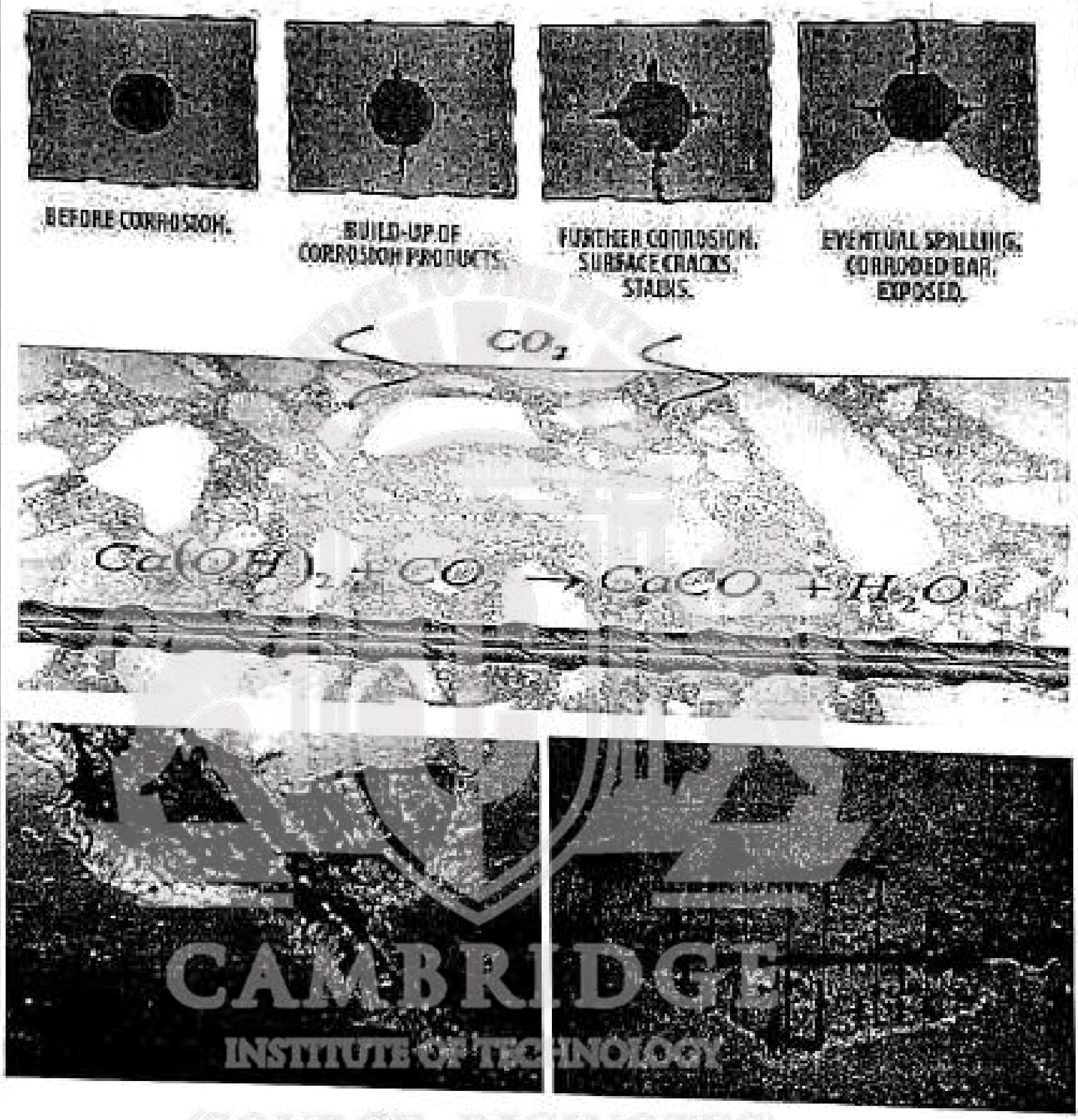


Figure: Corrosion of steel

The corroded steel occupies a much greater volume than the original metal and thus sets up bursting forces in the surrounding concrete.

Influencing factors

The rate at which carbonation reaches the reinforcement depends on the following factors.

- **Time:** Rate decreases with increasing time of exposure to air.
- **Cover:** Greater the cover better is the protection afforded to the steel.
- **Concentration of CO2 in the atmosphere:** Rate increases with increasing carbon dioxide in the air.
- **Permeability:** This depends on concrete quality, carbonization depth increases by four times when the water cement ration is increased from 0.4 to 0.8.
- **Alkali content in the concrete:** this depends upon the cement content and type of cement.
- **Condition of concrete cover:** any imperfection in the cover such as segregation, poor compaction or cracking enables carbonation to progress more rapidly.
- Concrete of good quality carbonates very slowly, even after a period of 50 years, carbonation is to penetrate to a depth of 5-10 mm. a permeable concrete may carbonate to a depth of 25 mm in less than 10 years.

Sulphate attack on the concrete

Sulphate attack on concrete is a chemical breakdown mechanism where sulphate ions attack components of the cement paste.

The compounds responsible for sulphate attack on concrete are water-soluble sulphate-containing salts, such as alkali-earth (calcium, magnesium) and alkali (sodium, potassium) sulphates that are capable of chemically reacting with components of concrete.

Sulphate sources:

- Internal Sources

- External sources

Internal Sources

This is rarer but, originates from such concrete-making materials as hydraulic cements, fly ash, aggregate, and admixtures.

- Portland cement might be over-sulphated.
- Presence of natural gypsum in the aggregate.
- Admixtures also can contain small amounts of sulphates.

External Sources

External sources of sulphate are more common and usually are a result of high-sulphate soils and ground waters, or can be the result of atmospheric or industrial water pollution.

- Soil may contain excessive amounts of gypsum or other sulphate.
- Ground water is transported to the concrete foundations, retaining walls, and other underground structures.
- Industrial waste waters.
- Atmosphere near the oceans may carry sulphate contents.

Solid sulphate salts do not attack concrete, but when present in solution they react with hardened cement paste (HCP).

Sulphate attack occurs when pore system in concrete is occupied by solution of sulphate.

Water sulphates enter into the porous concrete and react with the HCP products forming a whitish appearance, this indicates sulphate attack.

Sulphate attack results in form of expansion and cracking of concrete.

Sulphate reaction is dependent on the following parameters

- Concentration of sulphate ions
- Cations present in the sulphate solution
- The exposure conditions i.e., the amount of aggressive substance.

- Density & Permeability of concrete
- Amount of water available
- Type of cement in concrete
- Alternate wetting and drying cycles

Figure: Sulphate attack on concrete

Chemical Mechanism

- The effect of sulphate on concrete can be mainly chemical and physical and they are closely related.
- The sulphate attack or reaction is indicated by the characteristic whitish appearance on the surface.
- The chemical reaction between sulphate and hydration products results in changes in the microstructure and pore size distribution of the cement paste.
- Sulphate converts calcium hydroxide into large volume of calcium sulphate (Gypsum).

Reactions on hardened cement paste

- Sodium sulphate attacking $Ca(OH)_2$

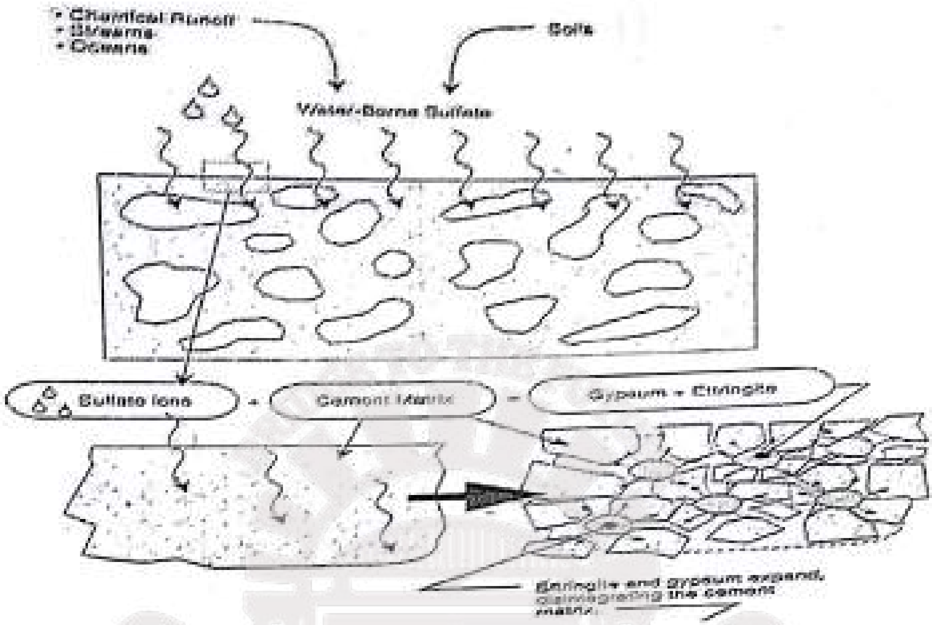
$$Ca(OH)_2 + Na_2SO_4.10\,H_2O \longrightarrow CaSO_4.2\,H_2O + 2NaOH + 8H_2O$$

- Reaction with Calcium aluminate hydrate

$$2(3CaO\,.\,Al_2O_3\,.\,12\,H_2O) + 3(Na_2SO_4.\,10H_2O) \longrightarrow 3CaO.\,Al_2O_3.\,3CaSO_4.\,31H_2O + 2Al(OH)_3 + 6NaOH + 17H_2O$$

(ettringite)

- The second hydration product, tricalcium aluminates hydrate reacts with sulphate solution to form sulpho-aluminates hydrate (Ettrignate), which has a greater volume than that of the original compound.
- Due to this, the resultant internal expansive stress may be great enough to cause deformation, cracking and eventually loss of cohesion.
- When concrete cracks, its permeability increases and the aggressive water penetrates more easily in to the interior, thus accelerating the process of deterioration.



Physical Mechanism

- In addition to the two chemical reactions, there is a purely physical phenomenon in which the growth of crystals of sulphate salt disrupts the concrete.
- The damage is usually starts at ends and corners and is followed by progressive cracking and spalling.
- The rate of sulphate attack increases with an increase in the strength of the solution concentration of sulphide.
- The rate of sulphate attack can be reduced by using the cement having low tricalcium aluminate content and by the addition of pozzolanic materials.

Preventive Measures

The quality of concrete, specifically a low permeability, is the best protection against sulphate attack.

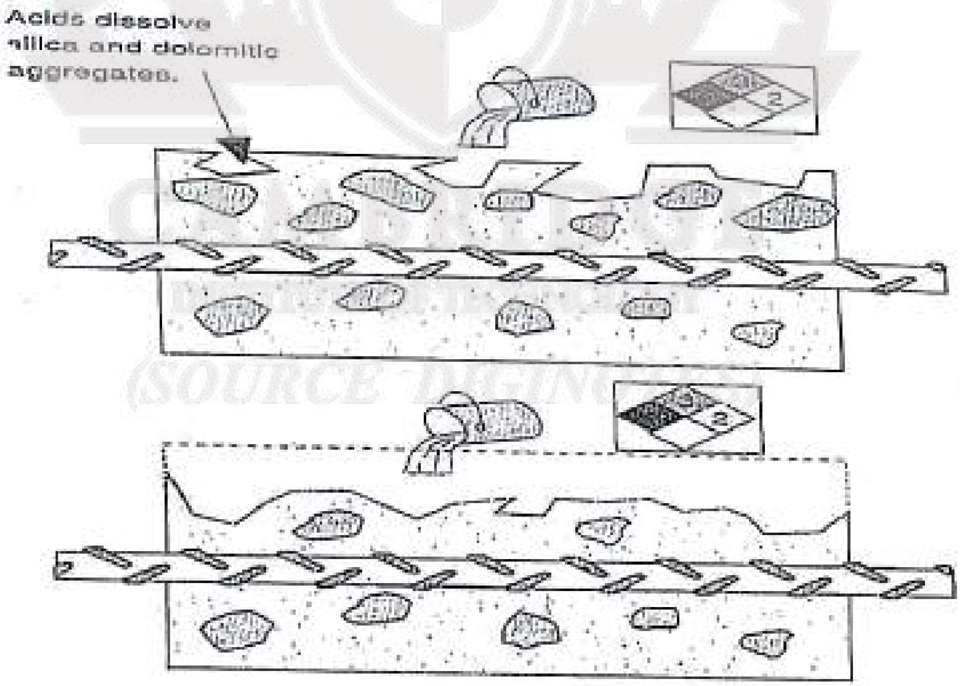
The concrete must have the following other characteristics:

- Adequate concrete thickness
- High cement content – with low tricalcium aluminate
- Low w/c ratio

- Proper compaction and curing
- Proper proportions of admixtures such as silica fume, fly ash and ground slag improve resistance against sulphate attack.
- Reducing the amount of reactive elements such as calcium that is needed for expansive sulphate reactions.

Acid attack on concrete

- Portland cement is a highly alkaline material and is not very resistant to attack by acids.
- Reaction between the acid and the calcium hydroxide of the hydrated Portland cement results in water soluble calcium compounds, which are leached away.
- When limestone or dolomite aggregates are used then the acid dissolves them.
- Dolomite is a carbonate mineral composed of calcium magnesium carbonate.
- If the acid is able to reach the reinforcing steel through cracks or pores of concrete, corrosion of the reinforcing steel will result and will cause further deterioration of the concrete.



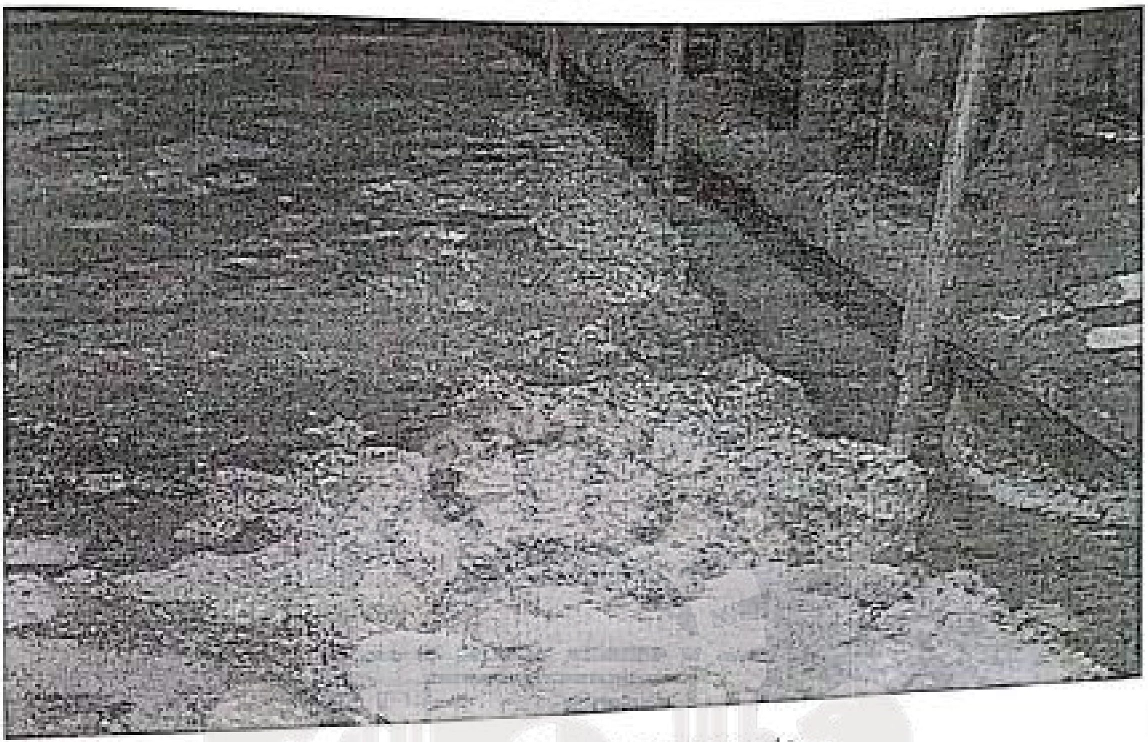
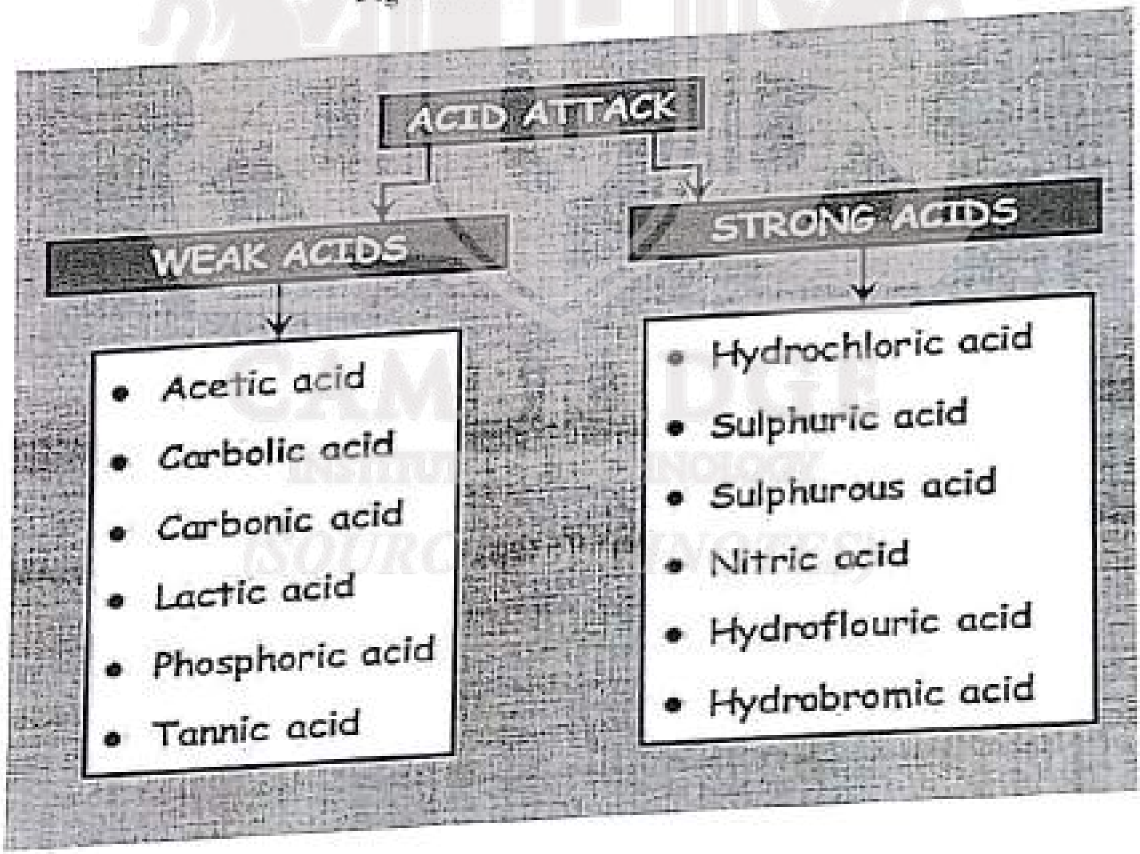


Figure: Acid attack on concrete



Mechanism of acid attack

- Concrete is susceptible to acid attack because of its alkaline nature. The components of the cement paste break down during contact with acids.
- The severity of the attack depends on the type of acid and the porosity of the cement paste.
- The action of acid on the concrete is to dissolve away the cement, and in case of limestone aggregate, the aggregate also gets dissolved.
- This results in the formation of water soluble calcium compounds which are then leached away by the aqueous solutions, the same leads to increase in porosity and the permeability of the system.
- Acids such as nitric acid, hydrochloric acid and acetic acid are very aggressive as their calcium salts are readily soluble and removed from the attack front.
- Other acids such as phosphoric acid and oxalic acid are less harmful as their calcium salt, due to their low solubility, inhibits the attack by blocking the pathways within the concrete such as interconnected cracks, voids and porosity.
- Sulphuric acid is very damaging to concrete as it combines an acid attack and a sulphate attack.

Symptoms

- Visual examination will show disintegration of the concrete evidenced by loss of cement paste and aggregate from the matrix.
- If reinforcing steel is reached by the acid, rust, staining, cracking and spalling may be present.
- All cement except high Alumina cement are equally susceptible to acid attack.

Preventive Measures

- By increasing cement content and reducing W/C ratio.
- By improving quality of cover concrete.
- By treating the surface with sodium silicate known as water glass.
- By surface treatment with coal, tar, bituminous paint, epoxy resins etc.
- By using pozzolanic materials like micro silica, slag to reduce the calcium hydroxide content.

Alkali aggregate reaction

Aggregates in most of the Concrete are chemically inert. However, certain types of sand and aggregate such as opal, chert or volcanic aggregate with high silica content are reactive with the alkalies like calcium, sodium and potassium hydroxide present in Portland cement concrete. This phenomenon of chemical reaction is referred as alkali-aggregate reaction. This reaction can cause expansion of the altered aggregate, leading to spalling and loss of strength of the concrete.

The alkali–aggregate reaction is a general, but relatively vague expression.

- Alkali–silica reaction (ASR)
- Alkali–silicate reaction and
- Alkali–carbonate reaction

Alkali Silica Reaction

It is the reaction between the alkalies in cement and silica-containing aggregates. The ASR reaction is the same as the pozzolanic reaction, which is a simple acid-base reaction between calcium hydroxide, ($Ca(OH)_2$), and silicic acid (H_4SiO_4, or $Si(OH)_4$).

This reaction can be schematically represented as following:

$Ca(OH)_2 + H_4SiO_4 \rightarrow Ca^{2+} + H_2SiO_4^{2-} + 2\ H_2O \rightarrow CaH_2SiO_4 \cdot 2\ H_2O$

This reaction causes the expansion of the altered aggregate by the formation of a swelling gel of calcium silicate hydrate (C-S-H). 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.

Mechanism

The mechanism of ASR causing the deterioration of concrete can be described in four steps as follows:

- The alkaline solution attacks the siliceous aggregate, converting it to viscous alkali silicate gel.

- 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 C-S-H.
- The penetrated alkaline solution converts the remaining siliceous minerals into bulky alkali silicate gel. The resultant expansive pressure is stored in the aggregate.
- The accumulated pressure cracks the aggregate and the surrounding cement paste when the pressure exceeds the tolerance of the aggregate.

The alkali-aggregate reaction may go unrecognized for some period of time, possibly years, before associate severe distress develops.

Figure: Typical effect of Alkali-Aggregate Reaction

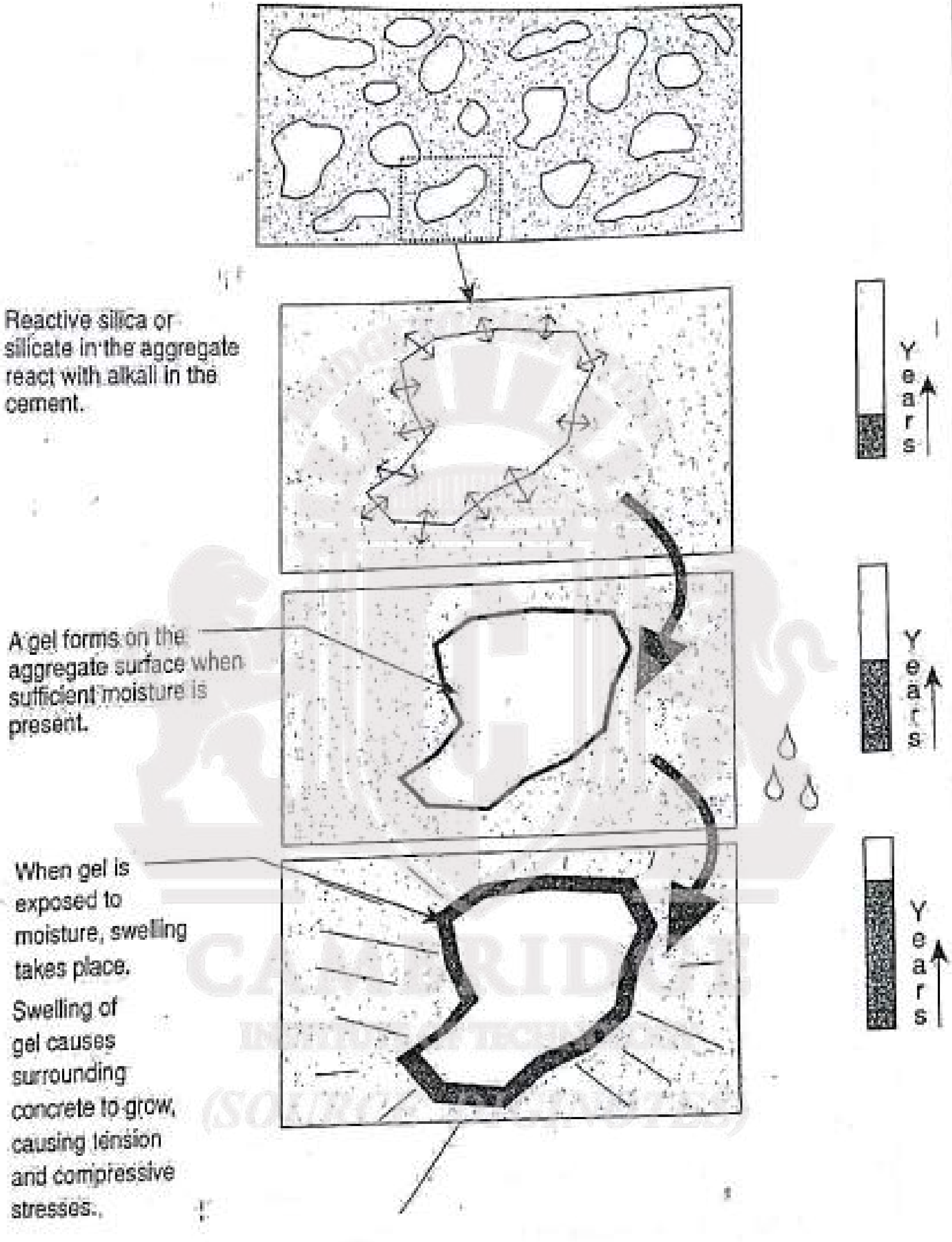


Figure: Alkali-Silica reaction mechanism

Influencing Factors

- Porosity of the aggregate.
- Alkali content of the aggregate.
- Fineness of the cement particles.
- Availability of non-evaporable water in the paste.
- Alternative wetting and drying.
- Temperature in the range 10-40 degree C accelerates the reaction.

Preventive measures

- Avoiding the use of reactive aggregates.
- Use of low alkali Portland cement, slag cement or pozzolanic admixtures.
- The rate of expansion can be reduced by taking steps to maintain concrete in as dry state as possible by the use of surface coatings or impregnation material.
- The repair of the concrete undergoing AAR should be carried out only after the expansion ceases because the continuing expansion will disrupt and destroy the repair material.

Alkali Silicate Reaction

In the alkali–silicate reaction, the layer of silicate minerals (clay minerals), sometimes present as impurities, are attacked.

Preventive measures

ASR can be controlled using certain supplementary cementitious materials. In proper proportions, silica fume, fly ash, and ground granulated blast-furnace slag have significantly reduced expansion due to alkali-silica reactivity. In addition, lithium compounds have been used to reduce ASR.

It is also important to note that not all ASR gel reactions produce destructive swelling.

Alkali Carbonate Reaction

The **alkali–carbonate reaction** is a process suspected for the degradation of concrete containing dolomite aggregate.

Alkali from the cement might react with the dolomite crystals present in the aggregate inducing the production of brucite, (MgOH)2, and calcite (CaCO3).

This mechanism is as follows:

$CaMg(CO_3)_2 + 2\ NaOH \longrightarrow CaCO_3 + Na_2CO_3 + Mg(OH)_2$

Brucite (Mg(OH)2), could be responsible for the volumetric expansion after de-dolomotisation of the aggregate, due to absorption of water.

ACR is relatively rare because aggregates susceptible to this phenomenon are less common and are usually unsuitable for use in concrete for other reasons. Aggregates susceptible to ACR tend to have a characteristic texture that can be identified by petrographers.

Salt attack on the concrete / salt weathering

Solid salts do not attack concrete, but when present in solution they can react with hardened concrete. It is a more general problem in masonry structures. Efflorescence is a whitish crystalline deposit on the surface. Efflorescence is the formation of calcium carbonate precipitation on the concrete surface owing to carbonation.

Salt-attack occurs when a surface made from stones or concrete is regularly exposed to water that contains salt. While the water may evaporate, the salt crystallizes and is left behind inside the pores of the concrete. As time passes, more salt is left behind each time the concrete is exposed to the salt water.

As more salt crystallizes, the pores in the concrete are forced to expand. Eventually this can leave to serious damage. This damage is what's referred to as salt-attack.

Degrees of damage caused by salt attack

The level of damage caused by the salt-attack will depend on a number of factors including the type of masonry, the type of salt, and the period of time in which the surface has been exposed.

Some of the signs of salt-attack include:

- Cracking, crumbling, and chipping.
- The surface may appear as if it has been exfoliated.
- Cavities may appear.

Figure: Salt attack on concrete

Preventive measures

- Using a material that's of the highest quality to ensure a very thick and dense cement paste that will greatly reduce the chances of getting salt-attack.
- Using salt-resistant chemicals.
- By applying sealers to a porous surface.
- By proper concrete proportioning. .
- Proper consolidation and curing.
- Preventing the access of moisture to the structure.

MODULE 2

DAMAGE ASSESSMENT

Purpose of assessment

Damage takes place due to natural or man-made causes. Even structures built in recent past have shown extensive damage and distress due to various causes, giving rise to problems both investigation and repair.

Diagnosis of the damage is of greatest importance. Before remedies can be correctly prescribed, the illness must be properly diagnosed.

Damage Assessment is a preliminary onsite evaluation of damage or loss caused by an accident or natural event. Damage assessments records the extent of damage, what can be replaced, restored or salvaged. It may also estimate the time required for repair, replacement and recovery.

Damage assessment is an integral part of facilitating effective and efficient response by government agencies and other organizations. Good damage assessment would start the ball rolling for effective response and relief operations such as evacuation, sheltering, search and rescue, mass casualty management, etc.

- Local damage assessment plays a critical role in your community's response and recovery following a hazard event.
- The information gathered by the damage assessment response team provides a snapshot of the situation detailing the extent and location of damages.
- This information is evaluated to determine the needs of the survivors and the community as a whole. Thus, the damage assessment sets the tone for the entire response operation and drives the recovery process.
- Damage assessment helps your community set priorities for response activities such as search and rescue, as well as for recovery operations such as removal of storm debris and rebuilding or repair of infrastructure.

- It also helps identify needs for additional resources from local, State, and Federal agencies and provides some of the documentation necessary for applying for these avenues of assistance.
- Damage assessment can also help you identify mitigation opportunities and create a mitigation plan that will make your community more disaster-resistant for the next hazard event.

Rapid structural safety assessment :-

The objective for the rapid structural safety assessment is to quickly inspect and evaluate the concrete structure and determine if the damaged structure is unsafe for personnel within the building and rescue personnel accessing the building. Two primary concerns need to be considered when performing this assessment of the structure that has sustained structural damage. This includes a quick evaluation of the building "structural" components (e.g., beams, columns, decking, etc.) and of the building "nonstructural" components "(e.g., structural debris, partitions, ceilings, glass, pipe anchoring, electrical/mechanical equipment anchoring, etc.). If there are any visual signs of structural and/or nonstructural damage, then the specific building area needs to be isolated, secured, and marked as UNSAFE. The on-scene commander should be informed and the area remained in this UNSAFE condition, until a structural engineer proves otherwise.

The rapid structural damage_assessment would note the major failures within the structure including the major structural elements of beams, columns, roof and floor decks. Typical failures would be found at the connections of the major structural elements, or at elements that no longer have adequate vertical support (e.g., unsupported roof and floor decks that are now cantilever elements.) Indications would include cracking, spalling (i.e., loss of concrete from an exterior surface), and/or complete loss of all or part of a structural element. The on-scene commander should be notified immediately of the risk, and the area secured and marked UNSAFE.

The rapid nonstructural damage_assessment would note the major failures within the building structure envelope including such items as structural debris, partitions, ceilings, glass, piping, and electrical/mechanical equipment. Concrete is a brittle material and, therefore, has a tendency to fragment into small, dense, hard pieces with rough edges. Many of these fragments may be precariously lying near or hanging from the exposed building edges and some fragments may be barely attached to exposed reinforcement steel. Settlement or shifting of the damaged structure may cause fragments to fall resulting in serious harm to personnel and/or additional damage to the remaining structure.

The issue of potential harm to personnel from concrete fragments and other building materials (such as glass, and other items that may have been in and around the structure) is exacerbated by activities that may be occurring in and around the remaining structure by rescue personnel operating throughout the building, along with rescue and/or press rotary-wing aircraft operating around the building. These activities may tend to cause accidental (e.g., physical interaction with something causing debris to fall) or inadvertent (e.g., vibration or rotor wash from rotary-wing aircraft causing debris to fall) mishaps. Typical failures in other nonstructural elements would be found where they originally were attached and/or secured. Failures would include anchoring tensile or shear failure, nonstructural element damage, and potential of future damage due to gravity loads and/or inadequate support bracing. The on-site commander should be notified immediately of the risk, and the area secured and marked UNSAFE.

The rapid structural assessment should be performed in the following order:

- Review the entire outside of the structure.
- Enter the building only if necessary to determine extent of damage.
- Determine what degree of damage found in the structural and non-structural elements.
- Secure all areas that need to be isolated and post UNSAFE signage.

Investigation of damage

The investigation of reasons for damaged to structure is largely a matter of gathering information by observation, studying records and asking questions, supplemented if necessary by a certain amount of testing and then interpreting the information thus obtained.

A systematic investigation of concrete structures is highly important an essential in order to:

- Find out the cause of damage.
- Access the condition of the structure and its damaged state.
- Formulate recommendations for repair and restoration.

T

- Documentation of damage.
- Visual observation.
- Measurement on geometrical parameters such as deflections and deformations, vertically and horizontally.
- Experiments for evaluating material properties and behaviour.
- Interpretation and analysis of test results.
- Analysis of the building in its damaged state taking into account the test results obtained during investigation.
- Formulation of repair measures.
- Post-repair evaluation.

O

The texture of a concrete surface may suggests the possibility of chemical attack by a general softening, leaching of the matrix or in the case of sulphate attack, and whitening of the concrete.

Rust stains often indicate corrosion of reinforcement but they may be caused by contamination of aggregates with iron pyrites. If cracked concrete is broken out, the state of the crack surface gives useful information. General flaking of an exposed concrete surface suggest frost damage.

In fire damaged structures the colour of the concrete gives an indication of the maximum temperature reached. When concrete is cracked the crack pattern can be informative. A mesh pattern suggest drying shrinkage, surface grazing, and frost attack or in a rare cases alkali-aggregate reaction.

Cracks caused by unidirectional bending will be widest in the zone of maximum tensile stress and will taper along their length, while cracks caused by direct tension will be of roughly uniform width. Continuing moment at a crack often produces crumbling at the edges. Pop-outs in concrete are usually associated with particles of coarse aggregate just below the surface. Highly absorptive particles may expand if severe frost occurs while they are saturated. Occasionally, coarse aggregates are contaminated with particles of lime or clay that expand with weathering.

Records of mix proportions, sources of materials, cube test results, weather conditions etc, may be available particularly for recent built structures but even these reliable information may be difficult to obtain. It is always useful to ask questions as many as possible to the people who are concerned with the design or construction. Their recollections may not be completely accurate and their accounts of events may conflict but, by questioning one can often find out what actually happened as distinct from what should have happened. This applies not only to work inside during construction but also to subsequent use of our misuse of the structure.

Evaluation of surface and structural cracks

S C This type of cracking often occurs in freshly placed concrete due to rapid evaporation of moisture from the concrete surface and is commonly referred to as plastic-shrinkage cracking. This can occur at any time due to an unfavorable combination of temperature, low humidity and wind.

Two types of surface cracks occur in concrete.

Cracks that form prior to the stiffening of hardening of concrete or while it is still workable.

H Cracks that form after the concrete sets and hardens.

S C Structural cracks are caused by a variety of issues, like poor soil bearing, overloading, swollen soil, and poor construction sites.

The structural cracks can be classified in to active or static crack.

A a crack where a movement is observed to continue is termed as active crack.

S a crack where no movement occurs is termed as static crack.

T

- Whether the crack is new or old.
- Type of crack, i.e., whether it is active or static.
- Pattern of the cracks.
- Soil Condition, type of foundation used, sign of movement of ground if any.
- Observations on similar structures in the same locality.
- Study of specifications, method of construction used and the test results at the site if any.
- Views of the designer, builder, and occupants of the building if any.
- Weather during which the structure has been constructed.

As in the case of a medical practitioner prescribing medicine without thoroughly examining the patient, it is difficult for a repair engineer to advocate any repair technology without making a thorough investigation. Before proceeding with repair, the investigations should be made to determine the location and extent of cracking, the causes of damage, and the objectives of repair. Calculation can be made to determine stresses due to applied loads. For detailed information, the history of the structure, structural drawings and specifications, and construction and maintenance records should be reviewed. The objectives of repair include restoration and enhancement of durability, structural strength, functional requirements and aesthetics.

T

- To identify the cause of cracking.
- To assess the structure for its safety and serviceability.
- To establish the extent of cracking.
- To establish the likely extent of the further deterioration.
- To study the suitability of various remedial measures.
- To make a final assessment for serviceability after repair.

Apart from visual inspection, tapping the structure surface and listening to the sound for hollow areas may be one of the simplest methods of identifying the weak spots. The suspected areas are then opened up by chipping the weak concrete for further assessment. The comparative strength of concrete in the structure may be assessed to a reasonable accuracy by non destructive testing and by tests on the cores extracted from the concrete. The commonly used non destructive tests are the rebound hammer and ultrasonic pulse velocity test.

Damage assessment procedure

The first step of Investigation is to make a diagnosis of the defect or the failure. It is basically an inductive procedure and it requires abundant caution.

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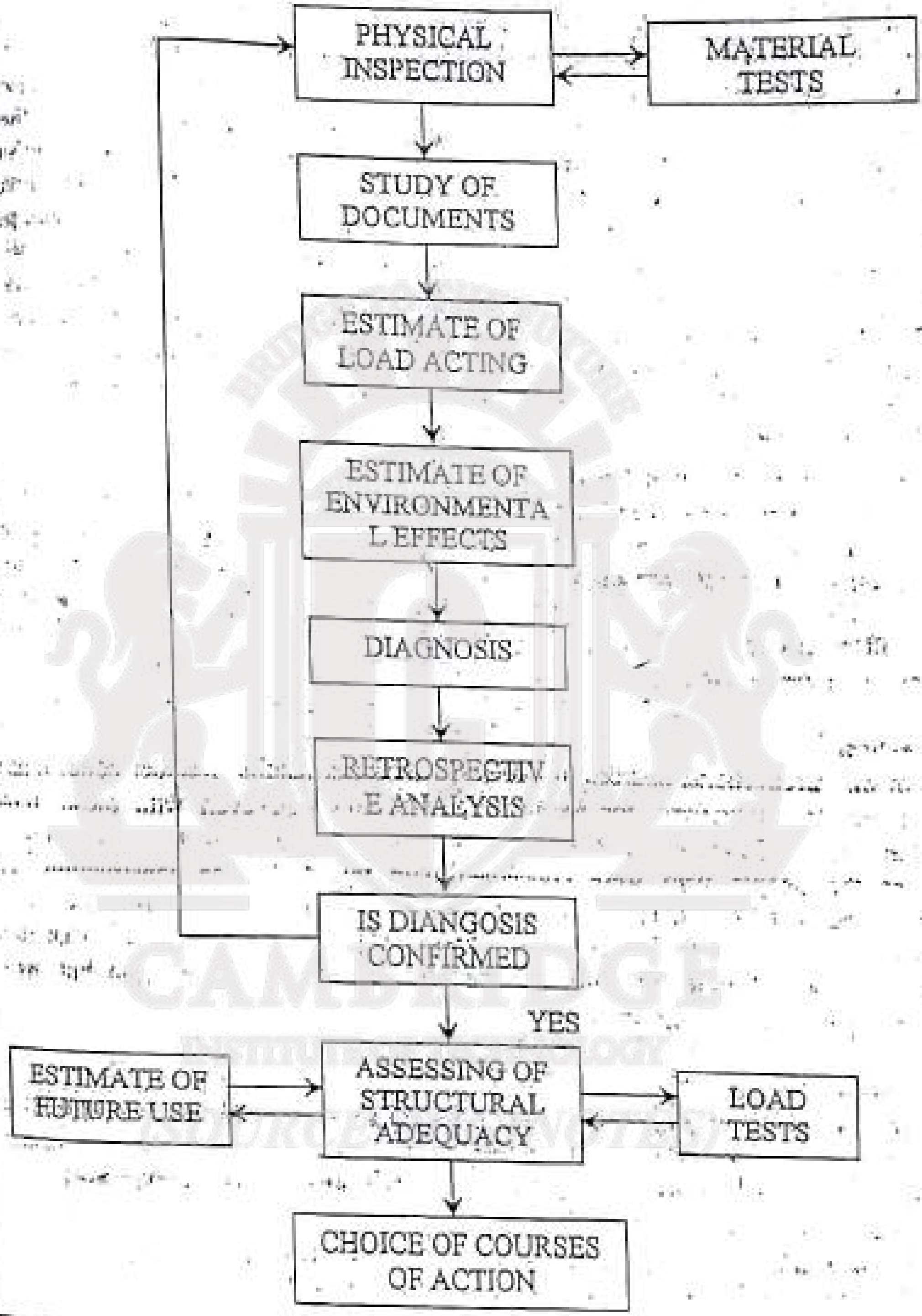
- Visual inspection
- Study of available documentation
- Estimation of actual loads and Environmental effects
- Diagnosis

The flowchart indicates the damage assessment procedure.



V

Any damaged structure, as a first step requires an extensive visual inspection followed by documentation of the details. A perusal of the documentation supported by photographs will reveal all possible evidence of structural defects and damages.

S

Study of available documentation will give some idea on the history of construction, original quality, analysis and design methods with assumptions made and the type of materials used. It is unfortunate that the relevant details are not readily available in many cases; a comparison of adjacent buildings also helps in a proper diagnosis.

E E

It is generally found that there is, in majority of cases the load acting on a structure will be much different from the loads assumed in design calculations. Therefore cracking or any other damage may sometimes be attributed to the fact that these loads or a certain load combinations were not considered in the analysis and design. Environment affects are likely to be different from those assumed or not considered at the design stage. Effects of temperature changes, or a hostile atmosphere would impose serviceability or durability problems. Environmental changes will result in undesirable effects in foundations.

In any investigation, diagnosis of the cause or causes of damage is of prime importance and is difficult too. A proper and reliable diagnosis can be made only by conducting a systematic investigation using proven test methods and experienced personnel.

Testing system of hardened concrete

Concrete as a material has high adaptability to satisfy many aspects in structure, such as functional, economy, maintenance, aesthetic acceptability, protection against corrosive environment, protection against fire, resistance to cyclic loading, explosion resistance and control over deflection etc. The concrete structure that are to be constructed as per specifications during its execution needs testing of constituent materials of concrete, tests on mixed concrete when it is fresh and finally when it has hardened. These tests on concrete are very important as their properties reflect durability of concrete. The structures which have already been built do exhibit distress after lapse of some years during their service life. There are number of testing system available and they could be used on structures to access the quality of concrete and steel.

T

- Non destructive testing system (NDTS)
- Partially destructive testing system (PDTS)
- Destructive testing system (DTS)

N To assess the quality of concrete in its damage state without any disturbance of surrounding concrete. Example: Rebound hammer, Ultrasonic pulse velocity method, Resonance frequency method, Dynamic or Vibration method, Pulse attention method, Pulse echo method, Radioactive method, Nuclear method, Magnetic method, Electromagnetic method, Electric method, Acoustic emission method, Radar Technique, Radiography method.

S To access the quality of concrete in its damaged state with partial disturbance of surrounding concrete. Example: Penetration techniques, Pull-out and Pull-off tests, Core sampling and testing, Break off test, Permeability test, Resistivity survey, Carbonation and pH value test, Chloride content test, Abrasion resistance test.

To access the quality of concrete with complete disturbance of concrete (loaded up to failure). Example: Compression strength of concrete, Flexural strength of beam, Split tensile strength of cylinder.

Testing system of hardened concrete

Concrete as a material has high adaptability to satisfy many aspects in structure, such as functional, economy, maintenance, aesthetic acceptability, protection against corrosive environment, protection against fire, resistance to cyclic loading, explosion resistance and control over deflection etc. The concrete structure that are to be constructed as per specifications during its execution needs testing of constituent materials of concrete, tests on mixed concrete when it is fresh and finally when it has hardened. These tests on concrete are very important as their properties reflect durability of concrete. The structures which have already been built do exhibit distress after lapse of some years during their service life. There are number of testing system available and they could be used on structures to access the quality of concrete and steel.

Testing system of hardened concrete can be divided into three categories

- Non destructive testing system (NDTS)
- Partially destructive testing system (PDTS)
- Destructive testing system (DTS)

Non destructive testing systems: To assess the quality of concrete in its damage state without any disturbance of surrounding concrete.

Non destructive evaluation is widely employed for inspecting the condition of structures, as its less time consuming and relatively inexpensive, can be used for the following purpose.

- Test on actual concrete.
- Test at several locations.
- Test at various stages.
- Assess the quality control of actual structures.
- Assess the uniformity of concrete.
- Assess the materials used and workmanship with specification.
- Assess the poor construction practices.
- Assessment of extent of cracks, voids and honeycombs.

- Conformation of suspected distress due to poor design.
- Monitoring of progressive changes in structures.

The various non destructive tests conducted to evaluate the structure are:

- Surface hardness method (Rebound hammer)
- Ultrasonic pulse velocity method
- Resonance frequency method
- Dynamic or Vibration method
- Pulse attention method
- Pulse echo method
- Radioactive method
- Nuclear method
- Magnetic method
- Electromagnetic method
- Electric method
- Acoustic emission method
- Radar Technique
- Radiography method.

Non destructive testing system (NDTS)

1. Surface hardness method (Rebound Hammer test):

The rebound hammer also called Schmidt hammer – a swiss hammer is one of the oldest & best known methods for comparing the concrete in different parts of structure. A simple hand held device measures the hardness of concrete surface through a rebound of a spring loaded mass rebound measured on a graduated scale giving a "Rebound Number" Larger the rebound number - harder the surface concrete. Most hammers come with a calibration chart relating the compressive strength of concrete and rebound number.

Objective of Rebound Hammer Test

As per the Indian code IS: 13311(2)-1992, the rebound hammer test have the following objectives:

- To determine the compressive strength of the concrete by relating the rebound index and the compressive strength
- To assess the uniformity of the concrete
- To assess the quality of the concrete based on the standard specifications
- To relate one concrete element with other in terms of quality

Principle of Rebound Hammer Test

Rebound hammer test method is based on the principle that the rebound of an elastic mass depends on the hardness of the concrete surface against which the mass strikes. The operation of the rebound hammer is shown in figure. When the plunger of rebound hammer is pressed against the concrete surface, the spring controlled mass in the hammer rebounds. The amount of rebound of the mass depends on the hardness of concrete surface. Thus, the hardness of concrete and rebound hammer reading can be correlated with compressive strength of concrete. The rebound value is read off along a graduated scale and is designated as the rebound number or rebound index. The compressive strength can be read directly from the graph provided on the body of the hammer.

Procedure for Rebound Hammer Test

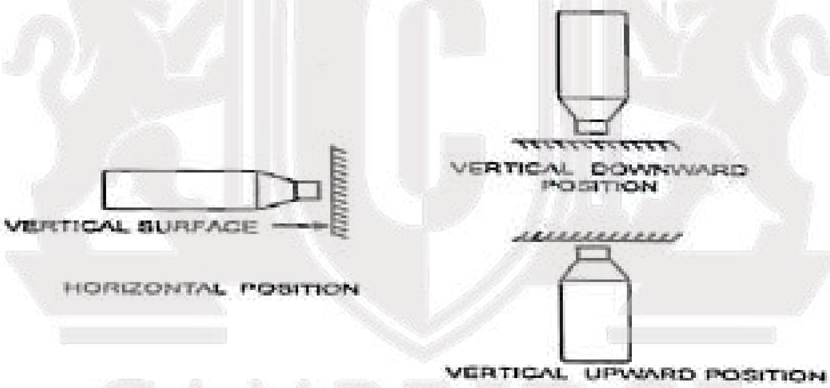
Procedure for rebound hammer test on concrete structure starts with calibration of the rebound hammer. For this, the rebound hammer is tested against the test anvil made of steel having Brinell hardness number of about 5000 N/mm2.

After the rebound hammer is tested for accuracy on the test anvil, the rebound hammer is held at right angles to the surface of the concrete structure for taking the readings. The test thus can be conducted horizontally on vertical surface and vertically upwards or downwards on horizontal surfaces as shown in figures below.

If the rebound hammer is held at intermediate angle, the rebound number will be different for the same concrete.

Steps to be followed

- All members to be marked with well defined grid points - spacing of 200 - 300 mm preferred.
- Each grid point to be cleaned and surface smoothened.
- A minimum of 6 readings to be obtained at each point and average considered omitting too low and too high values.
- Do not repeat impacts on same point.
- A hammer of 0.225 kg m impact energy used for normal concrete and structural members of medium size.



2.Rebound Hammer Positions for Testing Concrete Structure

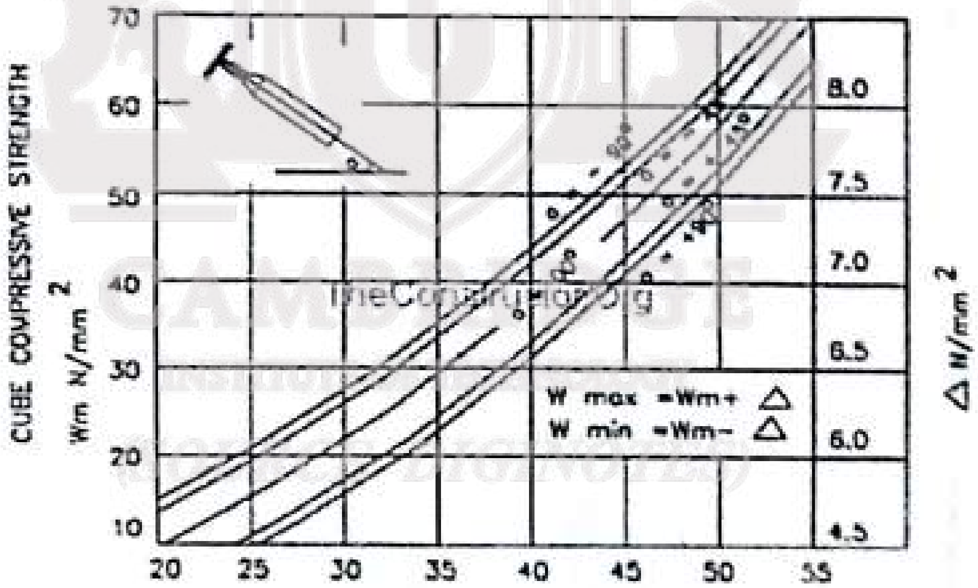
The impact energy required for the rebound hammer is different for different applications. Approximate Impact energy levels are mentioned in the table below for different applications.

Table-1: Impact Energy for Rebound Hammers for Different Applications As per IS: 13311(2)-1992

Sl.No	Applications	Approximate Impact Energy for Rebound Hammer in Nm
1	For Normal Weight Concrete	2.25
2	For light weight concrete / For small and impact resistive concrete parts	0.75
3	For mass concrete testing Eg. In roads, hydraulic structures and pavements	30.00

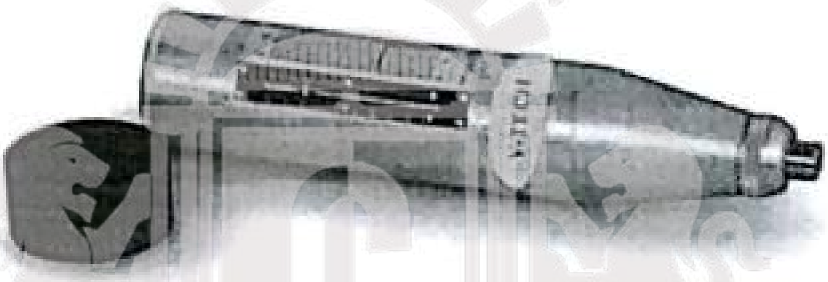
Interpretation of Rebound Hammer Test Results

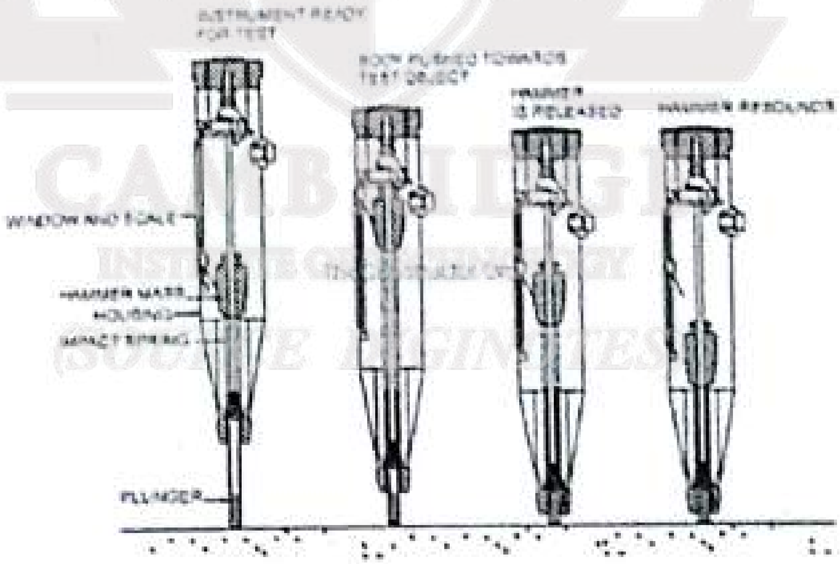
After obtaining the correlation between compressive strength and rebound number, the strength of structure can be assessed. In general, the rebound number increases as the strength increases and is also affected by a number of parameters i.e. type of cement, type of aggregate, surface condition and moisture content of the concrete, curing and age of concrete, carbonation of concrete surface etc.

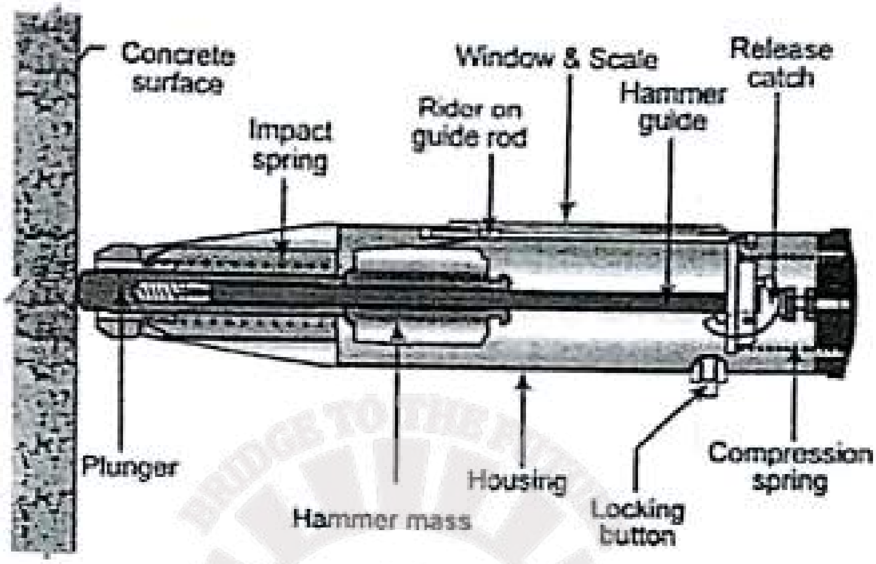


Quality of Concrete for different values of rebound number

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







Factors affecting rebound number readings

- Mix Characteristics- Cement type and content and coarse aggregate content.
- Angle of inclination of hammer
- Member Characteristics- Mass, compaction and surface type
- Age of concrete and Rate of hardening, curing type
- Surface texture
- Concrete mix characteristics
- Carbonated concrete, and
- Moisture content

Application: This test is used for estimating the uniformity & quality of concrete, monitoring the strength development, in-situ strength estimation and assessing relative quality of structural members.

Advantage: It is simple & quick method. A large number of measurements can be rapidly taken so as to map large exposed areas of concrete.

Limitation: Results are affected by the angle of test, surface smoothness and mix proportions of concrete. It does not provide a reliable prediction of the strength of concrete. The possible error may be up to ± 25%.

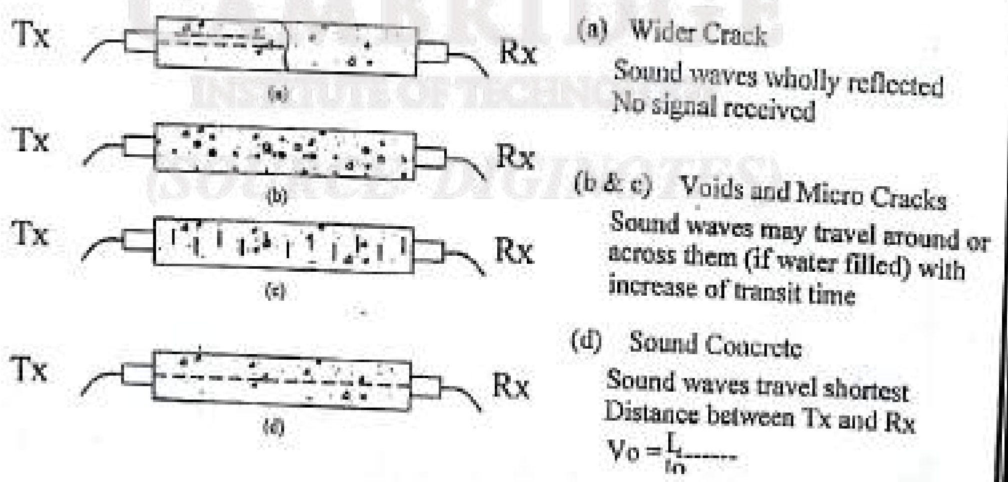
2. Ultrasonic pulse Velocity (USPV) test

It is most widely used test in evaluation of in-situ concrete. The method is based on the principle that the velocity of an ultrasonic pulse through any material depends upon the density, modulus of elasticity, the presence of the reinforcing steel & poison's ratio of the material.

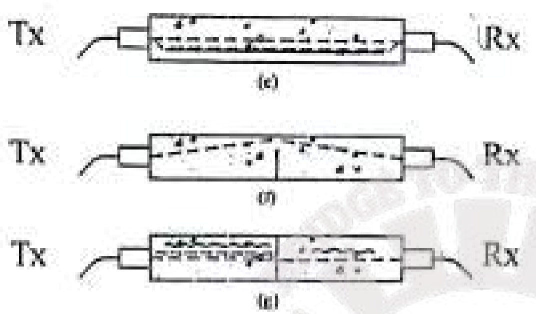
Procedure for UPV test

- Divide the members into well defined grid points - spacing of 200 - 300 mm preferred identical to rebound hammer survey.
- Each grid point is prepared to obtain smooth surface - a thorough cleaning.
- Application of acoustical coupling - grease, thick oil, petroleum jelly.
- Transmitting the pulses by placing the transmitter and receiving at other end (50-54 kHz).
- Recording the transit time displayed by the instrument - a reliable steady reading to be recorded.
- Measurement of length between transmitter and receiver.
- Calculation of velocity, V = L / T (L – Path length, T-time).

Behaviour of ultrasonic pulses in a concrete medium under different conditions



The principle of assessing the quality of concrete is that values of ultrasonic pulse velocities are found to be higher when it encounters a dense, homogeneous and uniform concrete along its path. Lower velocities are obtained in case of poorer quality.



(e) Reinforcing Steel
Sound waves travelling through steel
Arrive before those through concrete

(f) Shallow Crack
Sound waves travel around the carck
T1 > To = V1 < Vo

(g) Narrow Crack
Sound waves partially reflected and partially transmitted with large loss of amplitude but only slight increase in To

Interpretation of UPV values

UPV value km/sec. (V)	Concrete quality
Greater than 4.00	Very good
Between 3.50 and 4.00	Good, but porous
Between 3.00 and 3.50	Poor
Between 2.50 and 3.00	Very poor
Between 2.00 and 2.50	Very poor and low integrity
Less than 2.00	No integrity, large voids suspected

Applications: The tests help to determine the homogeneity of concrete, changes in structure of concrete with time, to assess the extent and severity of cracks in concrete. Precisely describes the areas of deteriorated and poor quality concrete.

Advantage: The test equipment is portable, can be performed quickly and has sufficient power to penetrate about 11m in good continuous concrete.

Limitations: The test does not give the precise strength of concrete. A large number of factors affect the values of pulse velocity that include surface condition and moisture content, temperature of concrete, micro-cracks in concrete, age of concrete, presence of steel reinforcement, aggregate type, content & size. When the concrete is subjected to abnormally high stress, pulse velocity value is reduced due to development of micro-cracks.

Semi / Partial destructive systems

This system of testing is employed to access the quality of concrete in its damaged state with partial disturbance of surrounding concrete.

Partially destructive tests are used for assessing the in-situ concrete strength. These tests cause localised damage, which do not cause any loss in the performance of the structure. All Tests in this group are surface zone tests which require assess to only one exposed concrete face compared to core test. The accuracy may not be as good but estimation of strength is immediately available and the testing is a less destructive and damaging. Some of the testing methods are available to access the quality of concrete under partial disturbance of the surrounding concrete; the common methods in construction industry are given below

- Penetration techniques (Windsor probe test)
- Pull-out and Pull-off tests
- Core sampling and testing
- Break off test
- Permeability test
- Resistivity survey
- Carbonation and pH value test
- Chloride content test
- Abrasion resistance test.

1. Windsor Probe test

It is one of the most well-known penetration resistance methods. It is based on the determination of the depth of the penetration of probes (steel rods or pins) into the concrete. The apparatus used in this method is Windsor probe which is a special gun that uses a 0.32 Caliber blank with a precise quantity of powder to fire a high strength steel probe into concrete. The principle underlying the technique is that penetration depth is inversely proportional to the compressive strength of concrete, but the relation depends on hardness of aggregate. The minimum distance between the edges of concrete member should be of the order of 150 mm and that between the test positions is 200 mm.

The Windsor probe is fired into a concrete surface using a special gun and a standardized explosive charge. The surface is cleared of loose material and a steel plate is positioned adjacent to the probe. A steel cap is screwed onto the probe and the length of the probe protruding from the surface is measured using a depth gauge and the depth of penetration can be determined. As the tip of the probe enters the concrete, it causes local crushing of the surface and creates a shock wave, which results in local spalling.

The penetration will be effected by the presence of reinforcing bars within the zone of influence of the penetrating probe. Thus the location of the reinforcing steel should be determined prior to selecting test locations.

Manufacturers provide calibration charts of strength versus penetration for the normal probe for aggregates with hardness between 3 and 7 on Mohr's scale. However, the penetration resistance should be correlated with the compressive strength of a standard test specimen or core of the actual concrete used.

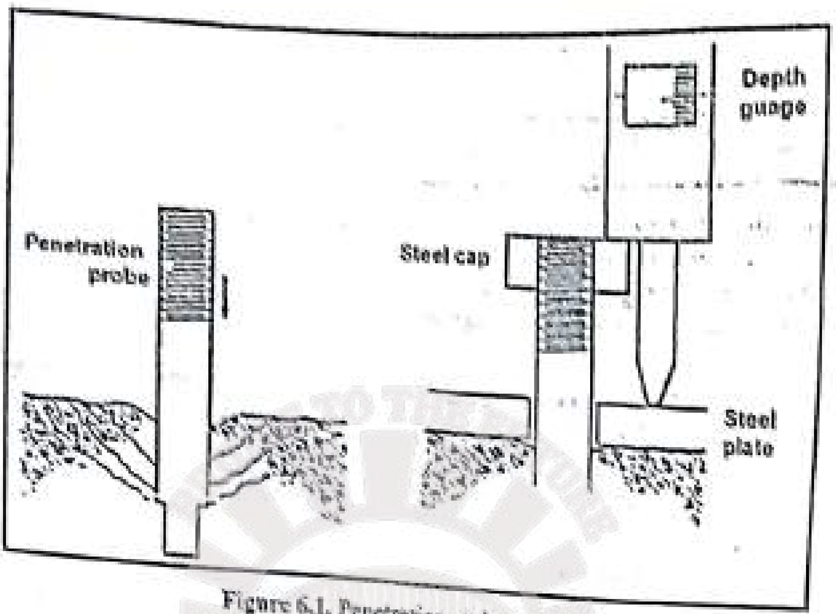


Figure 6.1. Penetration resistance test

Application: This test is used for estimating the uniformity & quality of concrete. An area of poor concrete can be easily described by making a series of penetration tests at regular spaced locations. This method provides excellent means for determination of relative concrete strength in the same structure or in different structures without extensive calibration.

Advantage: The test equipment is simple, durable and requires less maintenance and can be easily used with least training given to inspectors. Can also be used in places where access is limited.

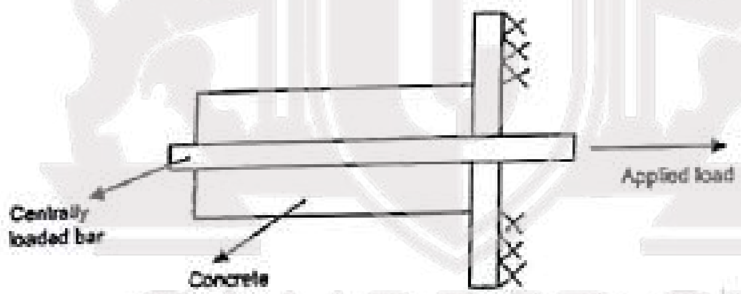
Limitation: It does not give reliable results on strength values. Type of aggregates affects the penetration depth; hence a separate calibration chart needs to be prepared for each type. This test damage the concrete leaving a hole of about 8 mm in diameter for the depth of the probe, hence minor cracks of exposed surface becomes necessary. Damage in the form of cracking may be caused in case of slender members.

2. Pullout and pull off test

Several tests are available which gives estimates of compressive strength by measuring the force required to pull out embedded anchors or to pull off disc glued to the surface. Many of the pull out tests use anchors, which are placed into the concrete at the time of construction.

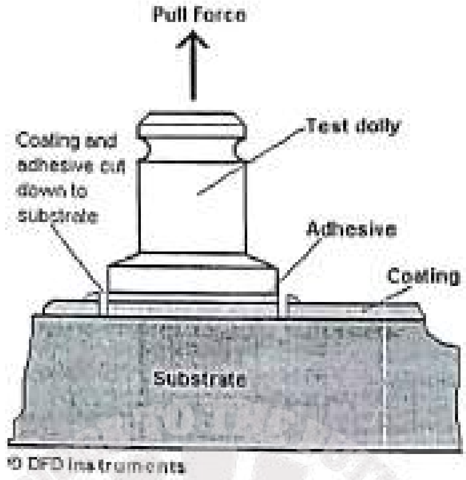
Pull out test: In this test, either an insert is cast in the concrete or fixed into a hole, which is drilled into the concrete. Force required to pull out the insert is measured which is correlated with the compressive strength. Although the results relates to the surface zone only, the advantage is that a more direct measure of strength and a greater depth compared to the surface hardness test and is available.

Pullout test measures the force required to pull out the special shaped rod inserted from the concrete. The stronger the concrete more is the force required to pull out. The ideal way to use pullout test in the field would be to incorporate assemblies is in the structure. These standard specimens could then be pulled out at any point of time. The force required denotes the strength of concrete. Another way to use pull out test in the field would be to cast one or two large blocks of concrete incorporating pull out assemblies. Pull out test could then be performed to assess the strength of concrete.



Pullout Test

Pull off test: is based on the measurement of the in-situ tensile strength of the concrete by applying a direct tensile force. The method is especially useful in measuring the bond between the overlays, in this test a metallic disk is glued to the concrete surface. The force required to full of the disk, causing tensile failure of concrete is measured and correlated to the strength of concrete. The test requires the care in preparing surface and causes difficulty with damp surfaces.



Destructive testing systems: To access the quality of concrete with complete disturbance of concrete (loaded up to failure).

In **destructive testing** (or **destructive physical analysis, DPA**) tests are carried out to the specimen's failure, in order to understand a specimen's performance or material behaviour under different loads. These tests are generally much easier to carry out, yield more information, and are easier to interpret than nondestructive testing. Destructive testing is most suitable, and economic, for objects which will be mass-produced, as the cost of destroying a small number of specimens is negligible. It is usually not economical to do destructive testing where only one or very few items are to be produced (for example, in the case of a building). Analyzing and documenting the destructive failure mode is often accomplished using a high-speed camera recording continuously (movie-loop) until the failure is detected. Detecting the failure can be accomplished using a sound detector or stress gauge which produces a signal to trigger the high-speed camera. These high-speed cameras have advanced recording modes to capture almost any type of destructive failure. After the failure the high-speed camera will stop recording. The capture images can be played back in slow motion showing precisely what happen before, during and after the destructive event, image by image.

The various destructive tests are:

Compression strength of concrete, Flexural strength of beam, Split tensile strength of cylinder.

MODULE 3

INFLUENCE ON SERVICEABILITY & DURABILITY

Serviceability: In civil engineering, serviceability refers to the conditions under which a building is still considered useful. Should these limit states be exceeded, a structure that may still be structurally sound would nevertheless be considered unfit. It refers to conditions other than the building strength that render the buildings unusable.

The serviceability limit state identifies a civil engineering structure which fails to meet technical requirements for use even though it may be strong enough to remain standing. A structure that fails serviceability has exceeded a defined limit for one of the following properties:

- Excessive deflection
- Vibration
- Local deformation (engineering)

Durability: Concrete is said to be durable if it withstands the conditions for which it has been designed, without deterioration, over a period of years. Or Durability of concrete may be defined as the ability of concrete to resist weathering action, chemical attack, and abrasion while maintaining its desired engineering properties. The term durability of concrete is used to characterize the resistance of a concrete to a variety of physical or chemical attacks due to external or by internal causes.

Effects due to climate

The lack of durability of concrete on account of freezing and thawing action of frost is not of great importance to Indian conditions. But it is of greatest considerations in most part of the world.

The most severe climatic attack on concrete occurs, when concrete containing water is subjected to cycle of freezing and thawing. The capillary pores in the cement paste are of such a size that water in them will freeze, when the ambient temperature is below 0° C.

The gel pores are so small that water in them does freeze at normal winter temperatures. As water, when freezing expands by 9% of its volume, excess water in the capillaries has to move. Since the cement paste is relatively impermeable high pressures are necessary to move the excess water even over quite small distances. For normal strength concrete, it has been found that movement of the order of 0.2mm is sufficient to require pressures which approach the tensile strength of the paste.

Concrete can be protected from freeze-thaw damage by the entrainment of the appropriate quantities of air distributed through the cement paste, with spacing between bubbles of not more than about 0.4mm. The air bubbles must remain partially empty, so that they can accommodate the excess water moved to them. This will generally be the case, since the bubbles constitute the coarsest pore system, and are therefore the first to, most moisture as the concrete dries. Fully saturated concrete, if permanently submerged, will not need protection against freezing, but concrete which has been saturated and is exposed to freezing as for example in the tidal range, may not be effectively protected by air entrainment.

For effective protection, an air entraining agent must be added to the mix, to entrain the appropriate amount of air, and to induce a bubble system, with an appropriate spacing. When AEA is used, it is only the amount of air entrained which can be measured in the wet concrete. The amount of air required is between 4-8%, depending on the maximum size of aggregate. Air is entrained during the mixing action, even when no AEA is added. The effect of AEA is to stabilize the air bubbles in the form desired.

More air is entrained with a larger dose of AEA but the effect is not linear and with most agents levels off larger doses. For mixes with higher slump, more air is entrained. It is difficult to entrain air is very stiff mixes; the grading and nature of the particles in the fine aggregates have a very marked effect, on the amount of air entrained. It has been shown that the sand is the most important single factor in air entrainment.

It has been suggested that if concrete can be so dense, that there are no inter-connected capillary pores, and then resistances to freeze- thaw deterioration will exist without the need for air entrainment.

The use of high cement content and low w/c ratio will lead in this direction as will the introduction of silica flume, but there is yet firm evidence to show that, it would be wise to dispense with air entrainment, if freeze-thaw resistance is wanted.

Effects due to temperature

Temperature variation will cause changes in the concrete volume. When the temperature increases the volume of the concrete increases and when the temperature falls the concrete contracts.

If the concrete is unrestricted then the volume changes will not create too many consequences but generally the concrete always restrained by foundations, reinforcement or connecting members, due to this the change in volume will produce significant stress in concrete and that may cause crack.

At high temperature, the cement paste will shrink due to dehydration of calcium silicate hydrate(C-S-H), while the aggregate will expand for normal aggregate concrete.

- Exposure to high temperature mainly to fire will result in crack in concrete but the time of exposure should sufficiently high.
- When the tensile strain of concrete exceeds its tensile strength capacity due to differential volume, it will crack.
- The liberation of the heat of hydration of cement causes the internal temperature of concrete to raise during the initial curing period, so that is usually slightly warmer than its surroundings.
- Any restraint on the free contraction during cooling will result in tensile stresses which are proportional to the temperature change, coefficient of thermal expansion, effective modulus of elasticity and degree of restraint.

- Temperature gradients cause deflection and rotation in structural members; if these are restrained serious stresses can result.
- The more massive the structure, the greater is the potential for temperature differential and degree of restraint.
- Thermally induced cracking can be reduced by controlling the maximum internal temperature, delaying the onset of cooling by insulating the formwork and exposed surfaces, controlling the rate of cooling, and increasing the tensile strain capacity of the concrete.
- A drop in temperature may result in the cracking of the exposed element while increase in temperature may cause cracking in the protected portion of the structure.

The common effects of temperature on concrete are

- The removal of evaporable water
- The removable of combined water
- Alteration of cement paste
- Disruption from disparity of expansion and resulting thermal stress
- Alteration of aggregate
- Change of the bond between aggregate and paste

Effects due to Chemical

The most important constituent of concrete namely cement is alkaline; so it will react with acids or acidic compounds in presence of moisture, and in consequence the matrix becomes weakened and its constituents may be leached out. The concrete may crack, as a result of expansive reactions between aggregate containing active silica and alkalis derived from cement hydration, admixture or external sources (e.g. curing water, ground water, alkaline solutions stored). The alkali – silica reaction results in the formation of a swelling gel, which tends to draw water from other portions of concrete. This causes local expansion and accompanying tensile stresses which if large may eventually result in the complete deterioration of the structure. Control measures include proper selection of aggregate, use of low-alkali cement and use of pozzolana. Typical symptoms in unreinforced and highly reinforced concrete are *map cracking*, usually in a rough hexagonal mesh pattern and gel excluding from cracks.

- The alkali-carbonate reactions occurs with certain limestone aggregate and usually results in the formation of alkali-silica product between aggregate particles and the surrounding cement paste. The problem may be minimized by avoiding reactive aggregate, use of smaller size aggregate and use of low-alkali cement.

- When the sulphate bearing waters come in contact with the concrete, the sulphate penetrates the hydrated paste and reacts with hydrated calcium aluminate to form calcium sulpho-aluminate with a subsequent large increase in volume, resulting in high tensile stresses causing the deterioration of concrete. The blended or pozzolana cements impart additional resistance to sulphate attacks.

- The calcium hydroxide in hydrated cement paste will combine with carbon dioxide in the air to form calcium carbonate which occupies smaller volume tan the calcium hydroxide resulting called *carbonation shrinkage*. This situation may result in significant surface grazing and may be especially serious on freshly placed concrete surface kept warm during winter by improperly vented combustion heaters.

Some of the factors, which increases the vulnerability of concrete to external chemical attack:

- High porosity and hence high permeability.
- Improper choice of cement type for the conditions of exposure.
- Inadequate curing prior to exposure.

- Exposure to alternate cycles of wetting and drying and to a lesser extent heating and cooling, with allowance for the fact that higher temperature increase reactivity.
- Increased fluid velocities.
- Expansive reactions of any sort which may cause cracking and any other physical phenomena, which lead to greater exposure of reactant surfaces.
- Suction forces.
- Unsatisfactory choice of shape and surface to volume ratios of concrete section

Effects due to wear and erosion

Wear is caused by two solids rubbing against each other, the resulting friction of which causes damage to the surface. So erosion and wear is the same thing, but wear is caused by a fluid and in erosion is caused by a solid.

- The concrete which has been damaged by erosion, it is almost certain that any repaired section will again be damaged unless the cause of the erosion is removed.
- Any concrete made will not withstand the forces of cavitations or severe abrasion for a prolonged period.
- It may be more economical to replace the concrete periodically rather than to reshape the structure to produce streamlined flow or to eliminate the solids which are causing abrasion.
- Abrasion-erosion damage is caused by the action of debris rolling and grinding against the concrete surface.
- In hydraulic structures, the areas most likely to be damaged are spillway aprons and basin slabs.

- The sources of the debris include construction trash left in the hydraulic structure, riprap brought back into the basin by eddy currents and debris thrown into a basin by the public.
- The rate of erosion depends on strength of concrete, duration of exposure, flow velocity of the water and its direction, and the amount, size, shape, density, hardness and velocity of the water borne debris.

Symptoms

- Concrete surfaces abraded by waterborne debris are generally smooth and may contain localized depressions.
- Mechanical abrasion is usually characterized by long shallow grooves in the concrete surface and spelling along joints.

Design Errors and Construction Errors

Design Errors

Design errors may be divided into two general types:

1. Those resulting from inadequate structural design
2. Those resulting from lack of attention to relatively minor design details.

Each of the two types of design errors is discussed below.

(1) Inadequate structural design.

(a) Mechanism. The failure mechanism is simple – the concrete is exposed to greater stress than it is capable of carrying or it sustains greater strain than its strain capacity.

(b) Symptoms. Visual examinations of failures resulting from inadequate structural design will usually show one of two symptoms.

1. First, errors in design resulting in excessively high compressive stresses will result in spalling. Similarly, high torsion or shear stresses may also result in spalling or cracking.

2. Second, high tensile stresses will result in cracking.

To identify inadequate design as a cause of damage, the locations of the damage should be compared to the types of stresses that should be present in the concrete. For example, if spalls are present on the underside of a simple-supported beam, high compressive stresses are not present and inadequate design may be eliminated as a cause. However, if the type and location of the damage and the probable stress are in agreement, a detailed stress analysis will be required to determine whether inadequate design is the cause.

(c) Prevention.

Inadequate design in prevented by thorough and careful review of all design calculations. Any rehabilitation method that makes use of existing concrete structural members must be carefully reviewed.

(2) Poor design details

A structure may be adequately designed to meet loadings and other overall requirements, poor detailing may result in localized concentrations of high stresses in otherwise satisfactory concrete. These high stresses may result in cracking that allows water or chemicals access to the concrete. In other cases, poor design detailing may simply allow water to pond on a structure, resulting in saturated concrete. In general, poor detailing does not lead directly to concrete failure; rather, it contributes to the action of one of the other causes of concrete deterioration described in this chapter. Several specific types of poor detailing and their possible effects on a structure are described in the following paragraphs. In general, all of these problems can be prevented by a thorough and careful review of plans and specifications for the project. In the case of existing structures, problems resulting from poor detailing should be handled by correcting the detailing and not by simply responding to the symptoms.

(a) Abrupt changes in section.

Abrupt changes n section may cause stress concentrations that may result in cracking. Typical examples would include the use of relatively thin sections such as bridge decks rigidly tied into massive abutments or patches and replacement concrete that are not uniform in plan dimensions.

(b) Insufficient reinforcement at reentrant corners and openings.

Reentrant corners and openings also tend to cause stress concentrations that may cause cracking. In this case, the best prevention is to provide additional reinforcement in areas where stress concentrations are expected to occur.

(c) Inadequate provision for deflection.

Deflection in excess of those anticipated may result in loading of members or sections beyond the capacities for which they were designed. Typically, these loadings will be induced in walls or partitions, resulting in cracking.

(d) Inadequate provision for drainage.

Poor attention to the details of draining a structure may result in the ponding of water. This ponding may result in leakage or saturation of concrete. Leakage may result in damage to the interior of the structure or in staining and encrustations on the structure. Saturation may result in severely damaged concrete if the structure is in an area that is subjected to freezing and thawing.

(e) Insufficient travel in expansion joints.

Inadequately designed expansion joints may result in spalling of concrete adjacent to the joints. The full range of possible temperature differentials that a concrete may be expected to experience should be taken into account in the specification for expansion joints. There is no single expansion joint that will work for all cases of temperature differential.

(f) Incompatibility of materials.

The use of materials with different properties (modulus of elasticity or coefficient of thermal expansion) adjacent to one another may result in cracking or spalling as the structure is loaded or as it is subjected to daily or annual temperature variations.

(g) Neglect of creep effect.

Neglect of creep may have similar effects as noted earlier for inadequate provision for deflections. Additionally, neglect of creep in prestressed concrete members may lead to excessive prestress loss that in turn results in cracking as loads are applied.

(h) Rigid joints between precast units.

Designs utilizing precast elements must provide for movement between adjacent precast elements or between the precast elements and the supporting frame. Failure to provide for this movement can result in cracking or spalling.

(i) Unanticipated shear stresses in piers, columns, or abutments.

Through lack of maintenance, expansion bearing assembles are allowed to become frozen, horizontal loading may be transferred to the concrete elements supporting the bearings. The result will be cracking in the concrete, usually compounded by other problems which will be caused by the entry of water into the concrete.

Construction Errors:

Failure to follow specified procedures and good practice or outright carelessness may lead to a number of conditions that may be grouped together as construction errors. Most of these errors do not lead directly to failure or deterioration of concrete. Instead, they enhance the adverse impacts of other mechanisms. Each error will be briefly described along with preventative methods. In general, the best preventive measure is a thorough knowledge of what these construction errors are, plus an aggressive inspection program. It should be noted that errors of the type described in this section are equally as likely to occur during repair or rehabilitation projects as they are likely to occur during new construction.

(a) Adding water to concrete. Water is usually added to concrete in one or both of the following circumstances:

1. First, water is added to the concrete in a delivery truck to increase slump and decrease emplacement effort. This practice will generally lead to concrete with

lowered strength and reduced durability. As the w/c of the concrete increases, the strength and durability will decrease.

2. In the second case, water is commonly added during finishing of flatwork. This practice leads to scaling, crazing, and dusting of the concrete in service.

(b) Improper alignment of formwork.

Improper alignment of the formwork will lead to discontinuities on the surface of the concrete. While these discontinuities are unsightly in all circumstances, their occurrence may be more critical in areas that are subjected to high-velocity flow of water, where cavitations erosion may be induced, or in lock chambers where the "rubbing" surfaces must be straight.

(c) Improper consolidation.

Improper consolidation of concrete may result in a variety of defects, the most common being bugholes, honeycombing, and cold joints.

"Bugholes" are formed when small pockets of air or water are trapped against the forms. A change in the mixture to make it less "sticky" or the use of small vibrators worked near the form has been used to help eliminate bugholes.

Honeycombing can be reduced by inserting the vibrator more frequently, inserting the vibrator as close as possible to the form face without touching the form, and slower withdrawal of the vibrator. Obviously, all of these defects make it much easier for any damage-causing mechanism to initiate deterioration of the concrete.

Frequently, a fear of "over consolidation" is used to justify a lack of effort in consolidating concrete. Over consolidation is usually defined as a situation in which the consolidation effort causes all of the coarse aggregate to settle to the bottom while the paste rises to the surface. If this situation occurs, it is reasonable to conclude that there is a problem of a poorly proportioned concrete rather than too much consolidation.

(d) Improper curing.

Curing is probably the most abused aspect of the concrete construction process. Unless concrete is given adequate time to cure at a proper humidity and temperature, it will not develop the characteristics that are expected and that are necessary to pro-vide durability. Symptoms of improperly cured concrete can include various types of cracking and surface disintegration. In extreme cases where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

(e) Improper location of reinforcing steel.

This section refers to reinforcing steel that is improperly located or is not adequately secured in the proper location. Either of these faults may lead to two general types of problems.

1. First, the steel may not function structurally as intended, resulting in structural cracking or failure. A particularly prevalent example is the placement of welded wire mesh in floor slabs. In many case, the mesh ends up on the bottom of the slab which will subsequently crack because the steel is not in the proper location.
2. The second type of problem stemming from improperly located or tied reinforcing steel is one of durability. The tendency seems to be for the steel to end up near the surface of the concrete. As the concrete cover over the steel is reduced, it is much easier for corrosion to begin.

(f) Movement of formwork

Movement of formwork during the period while the concrete is going from fluid to a rigid material may induce cracking and separation within the concrete. A crack open to the surface will allow access of water to the interior of the concrete. An internal void may give rise to freezing or corrosion problems if the void becomes saturated.

(g) Premature removal of shores or reshores.

If shores or reshores are removed too soon, the concrete affected may become overstressed and cracked. In extreme cases there may be major failures.

(h) Settling of the concrete.

During the period between placing and initial setting of the concrete, the heavier components of the concrete will settle under the influence of gravity. This situation may be aggravated by the use of highly fluid concretes. If any restraint tends to prevent this settling, cracking or separations may result. These cracks or separations may also develop problems of corrosion or freezing if saturated.

(i) Settling of subgrade.

If there is any settling of the subgrade during the period after the concrete begins to become rigid but before it gains enough strength to support its own weight, cracking may also occur.

(j) Vibration of freshly placed concrete.

Most construction sites are subjected to vibration from various sources, such as blasting, pile driving, and form the operation of construction equipment.

Freshly placed concrete is vulnerable to weakening of its properties if subjected to forces which disrupt the concrete matrix during setting.

(k) Improper finishing of flat work.

The most common improper finishing procedures which are detrimental to the durability of flat work are discussed below.

(1) Adding water to the surface. Evidence that water is being added to the surface is the presence of a large paint brush, along with other finishing tools. The brush is dipped in water and water is "slung" onto the surface being finished.

(2) Timing and finishing. Final finishing operations must be done after the concrete has taken its initial set and bleeding has stopped. The waiting period depends on the amounts of water, cement, and admixtures in the mixture but primarily on the temperatures of the concrete surface. On a partially shaded slab, the part in the sun will usually be ready to finish before the part in the shade.

(3) Adding cement to the surface. This practice is often done to dry up bleed water to allow finishing to proceed and will result in a thin cement-rich coating which will craze or flake off easily.

Effect of Cover Thickness

Concrete cover, in reinforced concrete, is the least distance between the surface of embedded reinforcement and the outer surface of the concrete. The concrete cover depth can be measured with a cover meter.

The concrete cover must have a minimum thickness for three main reasons:

- To protect the steel reinforcement bars from environmental effects to prevent from corrosion
- To provide thermal insulation, which protects the reinforcement bars from fire to give reinforcing bars sufficient embedding to enable them to be stressed without slipping?

The following are the reinforcement thickness of covers for various levels of exposure.

- For mild exposure : At least 20mm thickness
- For moderate exposure : At least 30mm thickness
- For severe exposure : At least 45mm thickness
- For very severe exposure : At least 50mm thickness
- For very extreme exposure : At least 75 mm thickness

Significance of providing concrete cover

- A sufficient thickness of concrete cover is required in order to slow down the carbonation process towards the rebar. Carbonation of concrete is one of the reasons for corrosion of reinforcement. It is a process by which carbon dioxide from air penetrates into the concrete and reacts with calcium hydroxide to form calcium carbonates.
- The percentage of CO_2 present is air vary from place to place. In case of rural areas the concentration of CO_2 in air may be about 0.03% by volume, where as in urban areas it may vary from 0.3% to 1.0%. This CO_2, in presence of moisture changes into dilute carbonic acid and attacks the concrete and reduces alkalinity of concrete.
- Due to reduction of alkalinity of concrete, the pH value of pore water in the hardened cement paste reduces from 13 to 9.0. When all the $Ca(OH)_2$ has

become carbonated, the pH value again reduces to about 8.3. And at this low pH value, the protective layer gets destroyed and the steel is exposed to corrosion.

Effect of cracks

The formation of cracks is dangerous for protection against corrosion. Once concrete cracks, the external agents can penetrate deep into concrete and set off the process of corrosion. Cracks running transversely to the reinforcement are less harmful than the longitudinal cracks along the reinforcement.

Thus in the order to induce the process of corrosion and to keep it going, at least one of the following conditions must exist in any RC structure.

- Chloride ion concentration in excess of the threshold value at the interface of the reinforcement and concrete or sufficient advancement of the carbonation front to destroy the passivity of the ferric oxide surface layer of the reinforcement.
- Adequate moisture in the concrete to facilitate the movement of chloride ions and provide a conduction path to the steel.
- Sufficient oxygen supply to the corrosion areas.
- Difference in electrochemical potentials at the surface of the reinforcement.
- Low values of electrical resistivity of concrete.
- Relative humidity in the range 50-70%.
- Higher ambient temperature.

Corrosion

Corrosion is a natural process, which converts a refined metal to a more chemically-stable form, such as its oxide, hydroxide, or sulfide. It is the gradual destruction of materials (usually metals) by chemical and/or electrochemical reaction with their environment.

Basic principal of corrosion

Carbonation of concrete is one of the reasons for corrosion of reinforcement. It is a process by which carbon dioxide from air penetrates into the concrete and reacts with calcium hydroxide to form calcium carbonates.

Concrete is a high alkalinity material. The pH of newly produced concrete is usually between 12 and 13. In this range of alkalinity, embedded steel is protected from corrosion by a passivating film bonded to the reinforcing bar surface. However, when the passivating film is disrupted, corrosion may take place.

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The passivation of steel is a process performed to make a surface passive, i.e., a surface film is created that causes the surface to lose its chemical reactivity. Passivation can result in the very much-desired low corrosion rate of the metal.

Corrosion influencing factors

- The cover thickness
- Quality of concrete
- Environmental conditions
- pH value in concrete
- Presence of cracks

Causes of corrosion

Corrosion of reinforcement bars can be due to: Entry of moisture through cracks, availability of oxygen and moisture at rebar level, carbonation and entry of acidic gaseous pollutants that reduce the pH of concrete, ingress of chloride ions, relative humidity & electrochemical action.

Corrosion mechanism of embedded steel

The corrosion process that takes place in concrete is electrochemical in nature very similar to a battery. The mechanism of corrosion involves four basic elements

- Anode: Site where metal atoms lose electrons i.e., where corrosion is initiated.
- Cathode: Site where electrons flow to and combine with other metallic and non-metallic ion.
- Electrolyte: A medium capable of conducting electric current by ionic current flow.
- Metallic path: Connection between the anode and cathode that completes the circuit.

At, anode the oxidation process releases Fe^{++} ions to concrete pore solution which flows to cathode to combine with hydroxyl ions to form Ferrous hydroxide, $Fe(OH)_2$. In highly alkaline solution and in absence of chloride ions, the anodic dissolution reaction of iron is balanced by the cathodic reaction. Fe^{2+} ions combine with OH^- ions to produce the stable passive film.

Corrosion protection techniques

Epoxy coating to reinforcement

The objective of epoxy coating to steel rebar is to provide a sufficiently durable barrier to aggressive materials which caused corrosion.

- The epoxy coating is applied in a factory to the steel prior to shipping to ensure corrosion resistance. Epoxy-coated steel reinforcing bars (rebar) may be used in any concrete subjected to corrosive conditions.
- Initially the rebar body is shot blasted to remove mill-scale; this ensures an adequate bond between the epoxy and the steel.
- The bar is then heated to a control temperature before passing through a spray booth. Hence, electro-statically charged epoxy powder particles are deposited evenly on the surface of the bar. Shortly after spraying, the epoxy starts to harden sufficiently on the bars.
- The coating thickness typically where is from 130 microns to 300 Micron.

Galvanized reinforcement

Galvanized reinforcement consists of standard black bar hot dipped in molten zinc. This process forms a coating, which is metallurgically bonded to the surface of the metal.

The surface of zinc reacts with calcium hydroxide in the concrete to form a passive layer preventing corrosion. However, it is a more difficult process to control as uneven coating can form and it is more expensive.

Improving metallurgy by adding certain elements

The bars can be improved for its corrosion resistance by adding certain elements such as chromium and copper during the formation process itself.

Stainless steel reinforcement

Stainless steel is the name given to the family of corrosion resistant steels containing a minimum of 12% chromium. On contact with air, the chromium forms a thin oxide layer on the surface of steel. This resists corrosion.

Non ferrous reinforcement

In conventional reinforced concrete structures exposed to aggressive environments, where chloride ingress or concrete carbonation can occur,, and corrosion can take place. This can lead to cracking and spalling of the concrete and eventually the structure may become unserviceable or unsafe.

The application of non-ferrous bars in place of steel can eliminate the susceptibility of the reinforcement to corrosion and subsequently lead to significantly longer service lives and lower maintenance requirements. Nonferrous re-bars are manufactured from polymer composite materials containing glass or carbon fibres embedded in a highly durable resin binder with a textured surface to achieve bonding to concrete.

Features of the product include high tensile strength but lower modulus than conventional materials. The bars are 25% the weight of steel; highly corrosion resistant to acids, chlorides and alkalis; nonmagnetic; electrically and thermally insulating. Limitations include the inability to bend the product on site; longer transfer lengths; poor shear strength and low strain to failure.

Costs for the bars are typically 3 times that of conventional reinforcement or 50% that of stainless steel bars.

Corrosion Inhibitors

Inhibitor: A substance which slows down or prevents a particular chemical reaction or other process or which reduces the activity of a particular reactant.

Corrosion inhibitors are substances which, when added (in small quantities) to an aggressive environment, reduce the corrosion rate of embedded steel in concrete in that environment.

This can inhibit corrosion when passivation would otherwise have been lost as a result of chloride ingress or carbonation. They are added to concrete during production and are referred to as 'integral' corrosion-inhibitors. So-called 'migratory corrosion-inhibitors' applied to hardened concrete are not admixtures.

Corrosion inhibitors are used to protect metals from corrosion in concrete as well as for temporary protection during storage or transport.

Corrosion-inhibiting admixtures are effective after the concrete has hardened and give a long-term increase in the passivation state of steel reinforcement and other embedded steel in concrete structures.

The three most common generic types of corrosion inhibiting admixture are:

- Calcium nitrite (normally contains a residual amount of calcium nitrate)
- Amino alcohols
- Amino alcohols blended with inorganic inhibitors

The dosage of corrosion inhibitors is usually dependent upon the client's expected serviceable life of the structure and on a range of factors that affect the durability of concrete. These include cement type, water-to-cement ratio, cover concrete to the steel, ambient temperature and the expected level of exposure to chlorides.

The typical dosage range for a 30% solution of calcium nitrite is 10–30 litres/m3 but is more usually used between 10 and 20 litres/m3.

Calcium nitrite corrosion inhibitor comes in a liquid form, containing about 30% calcium nitrite solids by weight. The more corrosion inhibitor is added, the longer the onset of corrosion will be delayed.

The dosage of amino alcohol-based corrosion inhibitors is usually between 3 and 4 volume per cent by weight of cement.

Mechanisms of actions of inhibitors

The inhibitor is chemically adsorbed on the surface of the metal and forms a protective thin film on the steel which protects against corrosion.

Corrosion-inhibiting admixtures have little effect on strength at either early or later ages. The function of this type of admixture is to enhance the long term durability of reinforced concrete by enhancing passivation to the cathodic and/or anodic areas of embedded steel.

- The addition of calcium nitrate extends the time to corrosion initiation.
- Total corrosion of samples with calcium nitrate is substantially less.

Use of Corrosion Inhibitors

- Corrosion inhibitors can significantly reduce maintenance costs of reinforced concrete structures throughout a typical service life of 40–60 years.
- Structures especially at risk are those exposed to a marine environment or other situations where chloride penetration of the concrete is likely. Such structures include jetties, wharves, and sea walls.
- Highway structures can be affected by the application of de-icing salts during winter months, and also as at multi-storey car parks where salt-laden water drips off cars and evaporates on the floor slab.

Corrosion resistant steel

Corrosion resistant steel alternatively can be said as *Stainless Steel* or *Inox steel.* Any metal to be made corrosion less, it has to be mixed with certain metal and be made into an alloy.

Alloy: a metal made by combining two or more metallic elements, especially to give greater strength or resistance to corrosion.

In case of stainless steel, it is an alloy of Nickel, manganese and chromium, added according to the property we desire to derive out of steel.

Chromium is added minimum of 10.5% of the mass of steel, from where steel derives its property of being corrosion less, not-staining, no rusting. However steel is still susceptible to stain under low oxygen, high salinity and poor air-circulation.

Generally, unless coated steel will readily corrode. However, when steel is placed into concrete it develops a passive oxide film, due to the high pH of the concrete. This passive film prevents further corrosion of the steel. If steel corrodes in concrete it may cause cracking or spalling of the concrete.

Corrosion of reinforcing steel may occur if the pH of the concrete is decreased, either from chemical attack or from reaction of the concrete with CO2 in the atmosphere. It may also occur if sufficient chloride ions reach the bar. These are typically introduced by into the concrete, from either de-icing salts or sea water.

Stainless steels do not suffer uniform corrosion, like carbon steel, when exposed to wet environments.

Unprotected carbon steel rusts readily when exposed to the combination of air and moisture. The resulting iron oxide surface layer (the rust) is porous and fragile.

Since iron oxide occupies a larger volume than the original steel this layer expands and tends to flake and fall away exposing the underlying steel to further attack.

In comparison, stainless steels contain sufficient chromium to undergo passivation, spontaneously forming a microscopically thin inert surface film of chromium oxide by reaction with the oxygen in air and even the small amount of dissolved oxygen in water.

This passive film prevents further corrosion by blocking oxygen diffusion to the steel surface and thus prevents corrosion from spreading into the bulk of the metal.

This film is self-repairing if it is scratched or temporarily disturbed by an upset condition in the environment that exceeds the inherent corrosion resistance of that grade.

Reinforcing bars with improved corrosion resistance over traditional carbon steel reinforcing bars are readily available. When selecting a particular type of corrosion resistant bar, issues such as level of corrosion resistance, cost, and availability should be considered.

Types of Rebar with Improved Corrosion Resistance:

- Stainless Steel Bars
- Galvanized Steel Bars
- Epoxy-Coated Reinforced Bars

Corrosion resistant Coating of steel

Corrosion resistant coatings protect metal components against degradation due to moisture, salt, oxidation or exposure to a variety of environmental or industrial chemicals.

Anti-corrosion coating allows for added protection of metal surfaces and acts as a barrier to inhibit the contact between chemical compounds or corrosive materials.

The object of coating to steel bar is to provide a durable barrier to aggressive materials, such as chlorides. The coatings should be robust to withstand fabrication of ribcage, and pouring of concrete and compaction by vibrating needle.

Benefits of Coating of steel

Coatings with anti-corrosive properties ensure metal components have the longest possible lifespan and the metal coatings below provide corrosion protection against humidity, saltwater, and chemicals.

Process of Steel coating

Simple cement slurry coating is a cheap method for temporary protection, against rusting of reinforcement in storage.

De-rusting

The reinforcement is cleaned with a de-rusting solution. This is followed without delay by leaning the rods, with wet waste cloth and cleaning powder. The rods are then rinsed in running water and air dried.

Phosphating

- Phosphate jelly is applied to the bars, with fine brush.
- The jelly is left for 45-60 minutes, and then removed by wet cloth an inhibitor solution is then brushed over the phosphated surface.

Cement Coating

- Slurry is made by mixing the inhibitor solution, with Portland cement and applied on the bar. A sealing solution is brushed after the rods are air cured. The sealing solution has an insite curing effect. The second coat of slurry is then applied and the bars are air dried.

Sealing

- Two coats of sealing solution are applied to the bars, in order to seal the micro-pores of the cement coated and to make it impermeable to corrosive salts.

Other methods of steel coating are:

- Epoxy coating of reinforcement steel.
- Galvanization of steel.

DAMAGE ASSESMENT

Introduction

➢Primary objective of Construction is Safety, Durability of structure, Serviceability of Structure

➢Maintenance of building structure is required to ensure proper functioning of its elemental as required by the users.

➢Deterioration of materials is either by ageing / by action of other destructive forces, poor selection of materials, occupational and human factors etc.. hence timely investigation will facilitate effective remedial measure to prevent further deterioration.

➢Visual damages – crack, spalling of concrete.

➢Primary task in investigation– 1) Structural damage 2) Non– Structural damage.

Aim of the Investigation:

✓To find root cause

✓To determine extent of damage either to Structural damage /Non– Structural damage

✓To formulate recommendations for repair and restoration

Purpose of Investigation

1. Legal
2. Insurance surveys
3. Structural failures

Various steps involved In investigation

1. Preliminary investigation
2. Physical inspections
3. Material tests
4. Non destructive tests
5. Detailed diagnosis of defects
6. Estimation of loads
7. Check In error in design
8. Strengthening requirements

Evaluation of Cracks

Classification : 1)Superficial Cracks
2) Structural cracks
i) Active Crack ii) Dormant/static Crack

The following information may help in diagnosing the cracks

1. Crack is new/ old
2. Type of crack
3. Pattern of crack
4. Soil condition
5. Observations on the similar structures in the same locali
6. Weather condition which the structure has been construc

Purpose of crack evaluation

1. To identify the cause of cracking.
2. To assess the structure for its safety and serviceability.
3. To know extent of cracking.
4. To know extent of deterioration.
5. To study suitability of remedial measures.
6. To make a final assessment for serviceability after repairs.

Module - 2 — M. Mukaram. Eye

Damage Assessment : Purpose of assessment, ~~Repai~~ Rapid Assessment, Investigation of damage, Evaluation of surface and structural cracks, damage assessment procedure, Destructive, Non-destructive & semi Destructive testing systems.

Damage Assessment Procedure

(i) Physical inspection of damaged structure
(ii) Presentation & documenting the damage
(iii) Collection of sample and carrying out test both in situ and in laboratory
(iv) Studying the documents including structural aspects
(v) Estimation of loads acting on the structure
(vi) Estimate of environmental effects including soil structure interaction.
(vii) Diagnosis.
(viii) Taking preventive steps not to cause further damage
(ix) Retrospective analysis to get the diagnosis confirmed.
(x) Assessment of structural adequacy
(xi) Estimation of future uses.
(xii) Remedial ~~measures~~ measures necessary to strengthen & repairing the Str.
(xiii) Post repair evaluation through ~~tests~~
(xiv) Load test to study the behavior.
(xv) choice of course of action for the restoration of Str.

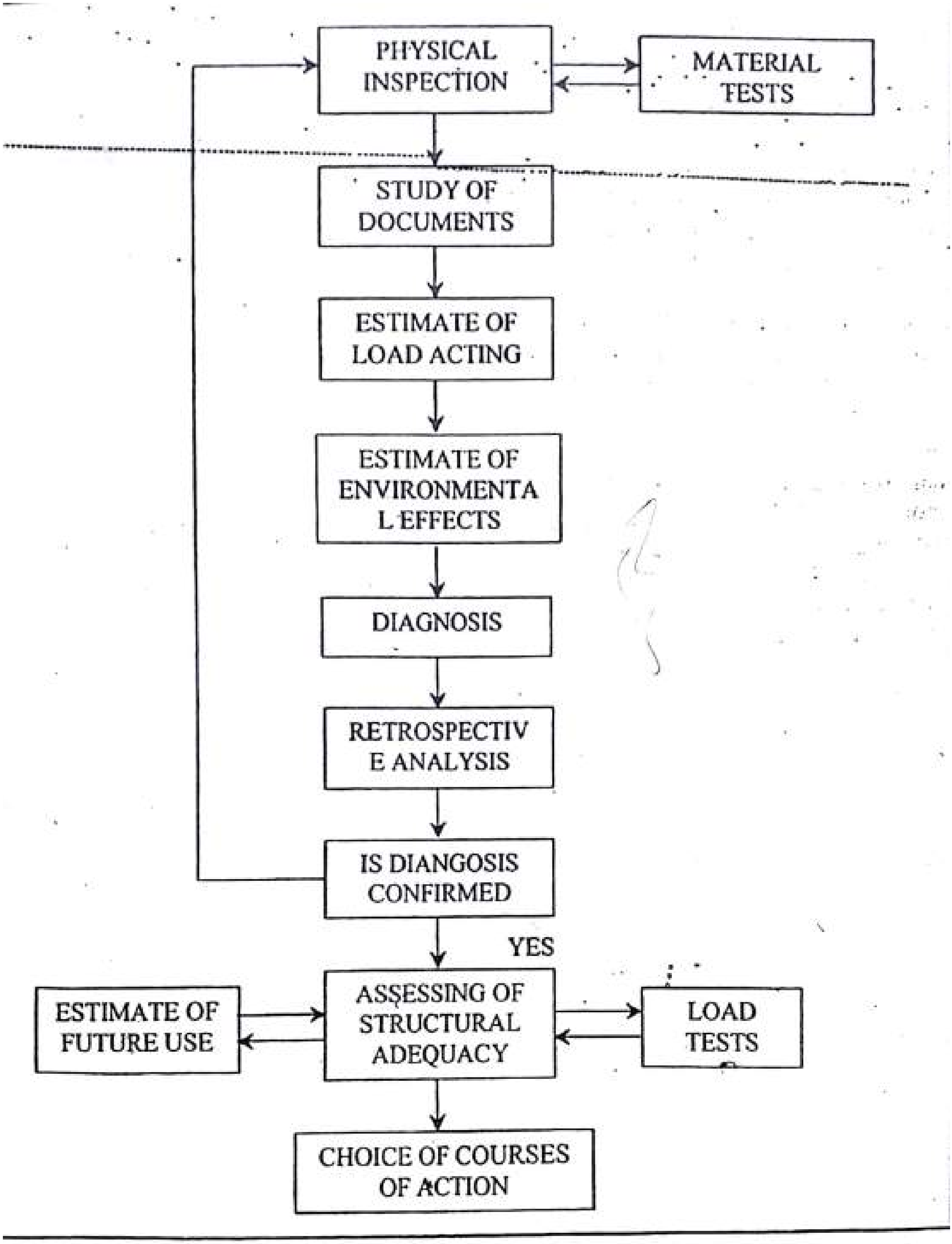

Figure 4.1. Assessment procedure for damage

4.5 DESTRUCTIVE TESTING SYSTEMS

The most common destructive test is load test and is used to assess the strength of concret structural elements. Load testing is sometimes used as an alternative method of assessin structural capacity. Load tests are usually carried out for one of the following reasons:

(a) There are still doubts about the satisfactory performance of the structure under loac after a survey and local testing.

(b) It is difficult or impossible to determine adequate information about the structure and its materials.

(c) Verification of structural analysis in cases where the complexity of the structural form does not lend itself to rigorous analysis

Deficiencies in detail, material or construction are suspected and such deficiencies would mean that the normal procedures or assumptions on which structural analysis is based were not appropriate. After confirming that the reinforcement did not go in to plastic, it was decided to conduct full scale load tests on the beams, so as to determine the load carrying capacity of the cracked beams and compare it with original design strength before any remedial measures are suggested. Accordingly load tests were conducted on most distressed beams. The following aspects were set for during the testing programme.

(a) To conduct the load test on beams up to 1.25 times the designed live load.

(b) To monitor the deflections and recovery of the beams during incremental loading and unloading

(c) To compare the actual deflections with that of theoretical deflections

4.5.1 Assessment of existing concrete structures

It has been learnt that concrete structures require a closer inspection, not only immediately after construction but also periodically at a regular interval. The quality control measures during construction, generally, consist of workability tests on fresh concrete and cube compressive strength of concrete samples, after some specified days of curing. It is a well known fact that the results of the above tests do not reflect the true quality of the concrete, existing in the concrete structure because the quality of a concrete structure depends on many factors such as method of mixing, transporting, placing, compacting and curing of concrete. While concrete members with certain amount of imperfections can satisfy the requirements relating to strength and serviceability, such concrete may not satisfy durability requirements.

(25)

Assessment of quality of concrete is necessary to ensure that the quality of execution i satisfactory and to identify any deficiencies so that they can be rectified. This can b achieved only by conducting some in-situ tests on the structures besides visual inspection The in-situ tests are non-destructive tests and partially destructive tests. These tests measur indirectly the strength of concrete except in the case of core test, where direct evidence or the condition of concrete and a measurement on compressive strength are possible. These testing methods are in use for evaluating existing concrete structures with regard to thei strength apart from assessment and quality control of hardened concrete.

4.5.2 Direct load test

This is a method of assessing the strength of the in-situ concrete member. In most cases thi: test is performed for the proof of structure capacity, not for suspect or critical location. The principal aim in this testing is to demonstrate satisfactory performance under an over load above the design working value. This is usually judged by measurement of deflections unde: this load which may be sustained for a specific period. The selection of specific members or portion of a structure to be tested will depend upon the general features of convenience as well as the relative importance of strength and expected load effect at various locations. Selection of member may often be assisted by non-destructive methods coupled with visual inspection to locate the weakest zones or elements.

There are certain inherent problems in this test. This method is very expensive and time consuming. In in-situ test whether the member under test is actually subjected to the assumed test load during fire is difficult to predict to predict due to load sharing effect. All the direct methods of assessing residual strength of concrete provide regarding average potential cube compressive strength and are not indicative of strength profile across the section. Direct load testing again may not be possible where the loading may damage the structure permanently or where the structural system in too stiff to show any deflection.

4.5.3 Load test on structural element

A live load of 1t/sqm was taken for imposed load calculation. Floor finishing load and partition wall load was also considered. Thus imposed load was taken as 1.25 times live load as per IS 456: 2000 Code recommendations. The load was imposed on the beam as udl spreads over an area of 55.5 sqm. Thus a total load of 84 t was arrived at and imposed in 12 stages of 7t each at an interval of 15 minutes. The load of 7t was imposed with the help of 200 sand bags weighing 35 kg each as first step and increased in steps up to 84t. Dial gauges and electrical strain gauges were fixed at different locations. Separate platform was provided to read and record the deflections from the dial gauges, strain from strain indicator during loading and unloading.

Deflections for the above mentioned loading steps were recorded. Total time for reaching maximum load was 26 hrs and the maximum load was kept for more than 43 hrs during which creep effect was monitored and the time taken for unloading was 32 hrs during which deflection recovery was monitored. The recorded deflections and theoretical deflections were compared.

CHAPTER 5

Non–Destructive Testing Systems

5.1. INTRODUCTION

Non-destructive evaluation is widely employed for inspecting the condition of structures. Non destructive techniques, which are less time consuming and relatively inexpensive, can be used for the following purposes;

(a) Test on actual structures.
(b) Test at several locations.
(c) Test at various stages.
(d) Assess the quality control of actual structures.
(e) Assess the uniformity of the concrete.
(f) Assess the materials used and workmanship with specification.
(g) Assess the poor construction practices.
(h) Assessment of the extent of cracks, voids, honeycombs.
(i) Confirmation of suspected distress due to poor design
(j) Assessment of partial durability
(k) Integrity testing of piles
(l) Monitoring of progressive changes in structure

The results of non-destructive tests are most useful when supplemented by a limited number of destructive test procedures. There are more testing techniques with different principles and applications are available to evaluate the properties of concrete. The concrete material is so complicated that the efficiency and quality cannot be established just by one single test. Most of the tests, which are used, for estimating the parameters of concrete provide an excellent means of establishing and evaluating the uniformity of concrete.

5.2 NON–DESTRUCTIVE TESTING METHODS

Potential and limitations of various non-destructive techniques cited below and are briefly discussed in the following paragraphs to apprise users of their relevance in field application.

(a) Surface Hardness Method
(b) Ultrasonic Pulse Velocity Method
(c) Resonant Frequency Method
(d) Dynamic or vibration method
(e) Pulse Attenuation Method
(f) Pulse Echo Method
(g) Radioactive Method
(h) Nuclear Methods
(i) Magnetic Methods

(j) Electro magnetic methods
(k) Electrical Methods
(l) Acoustic Emission Technique
(m) Radar Technique
(n) Radiography Methods

5.2.1 Surface hardness test

The surface hardness method consists of impacting the concrete surface in a standard manner. Activating a mass by a given energy and measuring the indentation or rebound achieve this. The most commonly and widely used instrument is a "Rebound Hammer". There are several types of hammers having varying impact energy from 0.07 kg m to 3 kg m. the high impact energy is used for mass concrete, road pavements and airport runways. The low impact energy hammers (0.07 to 0.09 kg m) are used for small and low strength materials. A typical rebound hammer is shown in figure 5.1.

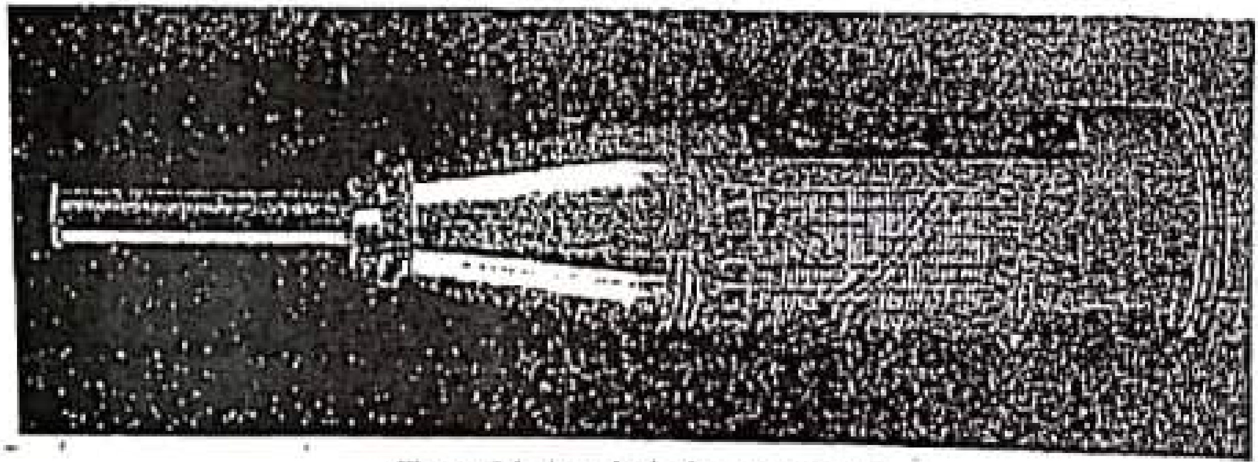

Figure 5.1. A typical rebound hammer

Test procedure:

The test procedure consists of applying the hammer on the concrete surface and observing the rebound reading indicated by a rider over a scale. Before applying the hammer, the surface of the concrete is cleaned and smoothened. A minimum of 10 readings is compared and each reading should not differ by more than 7 units. The average of remaining readings is determined for evaluating the strength. If more than two reading differ from the average by 7 units, than the entire set of readings are taken afresh. The testing procedure in hammer is as shown in figure 5.2.

The procedure for determining the rebound values has been specified in ASTM C 805-85, BIS-13311 Part 2 and also in the latest ASTM specification. Estimation of concrete compressive strength from rebound number is determined from standard calibration curve based on the laboratory results. The calibration curve should be established for each type of concrete. A typical calibration curve is shown in figure 5.3. The main factors that affect the readings are:

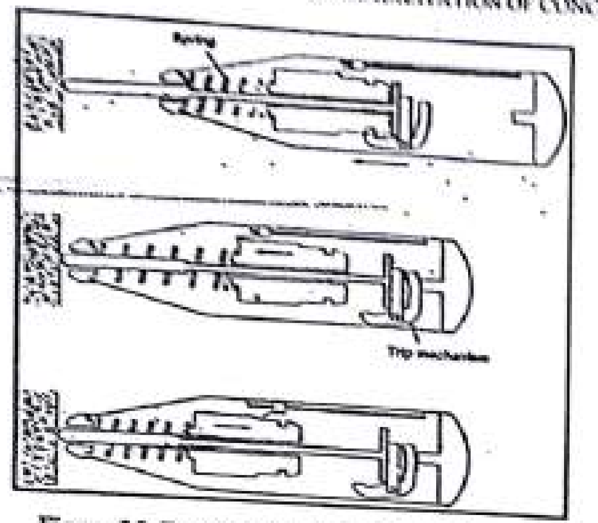

Figure 5.2. Testing procedure of rebound hammer

(a) Size and age of concrete
(b) Surface texture
(c) Concrete mix characteristics
(d) Stress state & temperature
(e) Carbonation level in concrete &
(f) Moisture content

It should be noted that the rebound values reflect the concrete quality up to a depth of 50 mm in the member. In practical situation, the strength prediction can be made to an accuracy of ± 25%. **The application of surface hardness measurements can be used for:**

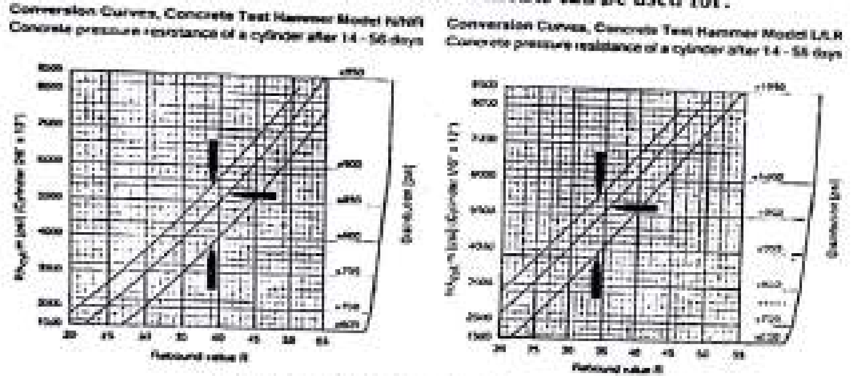

Figure 5.3. Typical calibration curves

(a) Checking the uniformity of concrete
(b) Comparing a given concrete with a specified requirement

(c) Approximate estimation of strength by using laboratory calibrated graphs
(d) Abrasion resistance classification

The following table 5.1 shows the quality of concrete cover from rebound number.

Table 5.1. Quality of cover concrete from rebound number

Average rebound number	*Quality of concrete*
Greater than 40	very good hard layer
30 to 40	good layer
20 to 30	fair
less than 20	poor concrete
0	delaminated

5.2.2 Ultrasonic pulse velocity (UPV) test

The object of this method is basically to measure the velocity of the pulses of longitudinal vibrations passing through concrete. The first report of the measurement of the velocity appeared in USA in the mid of 1940s. It was found that the velocity depended upon the elastic property and geometry of the material. With the development of reliable measurement techniques, the ultrasonic methods have become widely accepted around the world and commercially produced lightweight equipments. If properly used by an experienced operator, a considerable amount of information about the interior of the concrete member can be obtained. Recommendations for the use of this method are given in BS-4408 Part 5, ASTM C 597–71, and BIS-13311 Part 1.

Theory of pulse propagation through concrete:

Three types of waves are generated by an impulse applied to a solid mass. They are longitudinal (compression), shear (transverse) and surface (Reyleigh) waves. Longitudinal waves are fastest and provide more useful information than others. For an infinite, homogenous isotropic elastic medium, the compression wave velocity is given by

$$V = \sqrt{k\, E_d/\rho}.$$

Where,

V = Compression wave velocity km/sec,
$k = (1-\gamma)/[(1+\gamma)(1-2\gamma)]$.
E_d = dynamic modulus of elasticity,
ρ = mass density, and
γ = dynamic Poisson's ratio.

Test procedure:

The velocity of an ultrasonic pulse is influenced by properties of concrete, which determine its elastic stiffness and mechanical strength. Thus the variations in the pulse velocity values reflect a corresponding variation in the state of concrete under test. In order to evaluate the strength of concrete, calibration charts should be established based on laboratory tests. The calibration chart is influenced by a number of factors such as type of cement, cement content, admixtures, type & size of aggregate, curing conditions and age of concrete. Hence, adequate care and caution should be exercised while translating the pulse velocity values in terms of strength. The test consists of transmitting longitudinal vibrations produced by an electro – acoustical transducer from one side of the concrete, receiving the signal from the other side, and measuring the transit time (T) of the pulse using electronic time circuits. The path length (L) of the pulse is measured and velocity (V) is calculated as $V = L/T$.

The measuring equipment consists of an electrical pulse generator, a pair of transducers, an amplifier, and an electronic timing device for measuring the time interval elapsing between the onset of a pulse generated at the transmitting transducer and the onset of its arrival at the receiving transducer. Transducers of 50 to 60 kHz are found to be useful for most of the applications in concrete testing. A typical measuring instrument is as shown in figure 5.4.

Figure 5.4. UPV measuring instrument

The pulse velocity measurements may be used to establish:

(a) The homogeneity of the concrete
(b) The presence of cracks, voids and other imperfections
(c) Changes in the structure of the concrete which occurs with time
(d) The quality of the concrete in relation to standard requirements
(e) The quality of one element of concrete in relation to another
(f) The values of elastic modulii of concrete

There are three possible ways of measuring pulse velocity are shown in figure 5.5:

(a) Direct transmission
(b) Indirect or Surface transmission
(c) Semi–direct transmission

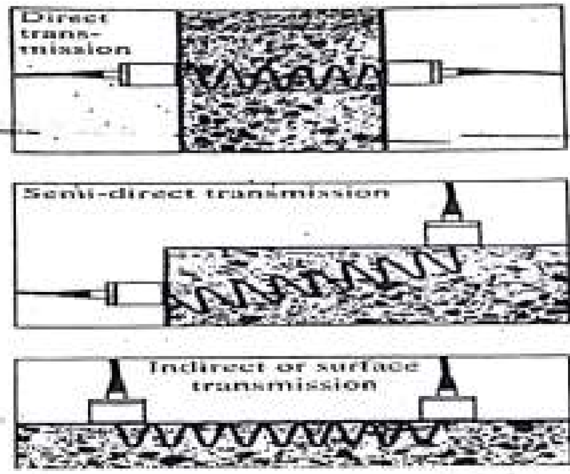

Figure 5.5. Methods of transmission system

The direct transmission method is generally preferred since the maximum energy of the pulse is being directed at the receiving transducer and this gives maximum sensitivity. The indirect transmission arrangement is the least sensitive and for a given path length, produces a signal which has only about 2% or 3% of that produced by direct transmission. This arrangement is used when only one face of the concrete is accessible, when the depth of the surface crack is to be determined or when the quality of the surface concrete relative to the overall quality is of interest. The semi direct transmission arrangement has a sensitivity intermediate between those of the other two arrangements. In this method, there is uncertainly regarding path length. It is generally found to be sufficiently accurate, if the length is measured from centre of transducer faces.

Some of the factors that influence pulse velocity measurements are:

(a) Surface condition
(b) Moisture content
(c) Temperature of concrete
(d) Path length
(e) Shape & size of specimen
(f) Reinforcement

The following figure 5.6 shows the behaviour of ultrasonic pulses in concrete medium under different conditions.

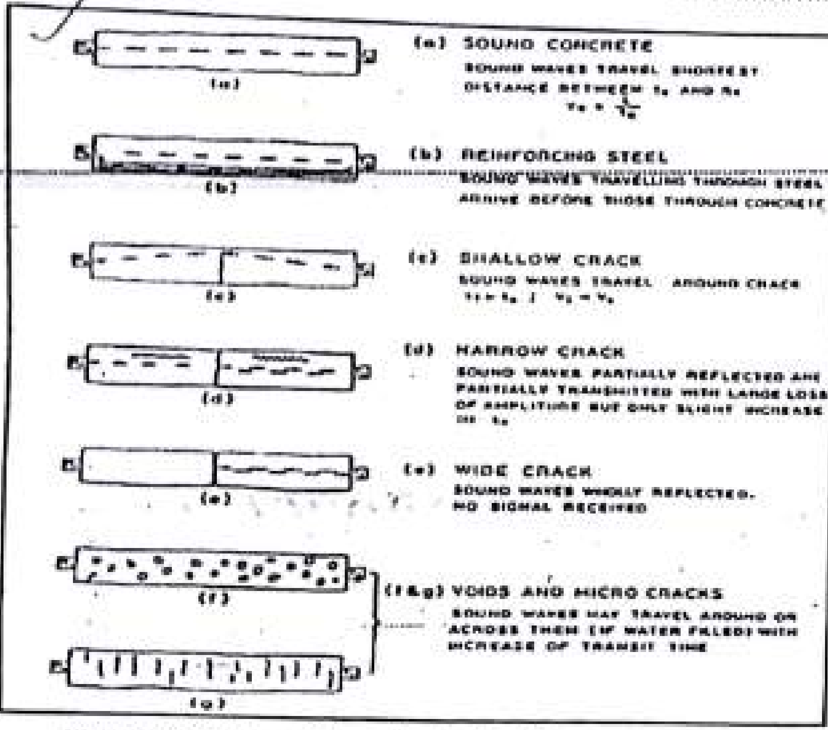

Figure 5.6. Behaviour of ultrasonic pulses in different concrete medium

he following table 5.2 gives the criteria are adopted for certification qualitatively of oncrete under UPV results.

Table 5.2. Quality of concrete from UPV

UPV value km/sec (V)	*Concrete quality*
V greater than 4	Very good
V between 3.5 & 4	Good
V between 3 & 3.5	Poor
V between 2.5 & 3	Very poor
V between 2 & 2.5	Very poor & low integrity
V less than 2	Large voids suspected

dvanced ultrasonic testing equipments:
is category comprises the range of instruments that use sound or stress waves in order to termine the properties of concrete and other materials non-destructively. The first and most idely used System is V-Meter, which utilizes the ultrasonic pulse velocity method for aluating construction materials in the field. Transducers are available for a variety of

frequencies from 24 KHz to 500 KHz. This unit has also been modified to suit the special needs of ceramics users and can be found as the ultra pulse.

The **Portable Impact-Echo System (PIES)** is an advanced instrument for non-destructive detection of flaws and defects in a variety of civil infrastructures ranging from bridges, parking structures and buildings to dams, piles, tunnels, tanks and marine structures. The E-Meter System represents the state of the art in bench top laboratory resonant frequency analysis of materials. This unit comes standard with a test bench designed to handle a variety of modes of vibration, including longitudinal, torsional and flexural. This line of products represents the most effective and efficient way to evaluate concrete and other materials in the field and utilizes the latest in technology in order to guarantee accurate results.

5.2.3 Resonant frequency method
This method is based up on the determination of the fundamental resonant frequency of vibration of a specimen. The equipment used for this is usually known as **SONOMETER.** Resonant frequency methods are mostly used in the laboratory. The size of specimens in these tests is usually limited to 150 mm × 300 mm cylinder or 75 mm × 75 mm × 300 mm prisms. **The resonant frequency test is used for the following purposes:**

(a) For studying the deterioration effects of concrete subjected for repeated cycles of freezing and thawing.
(b) To study the effects due to acidic and alkali reactions
(c) For determining the damage due to fire
(d) To calculate the dynamic young's modulus of elasticity of concrete.

5.2.4 Dynamic or vibration method
These methods are important non-destructive methods used for testing concrete strength and other properties. The fundamental principle of this method is in the propagation of sound velocity through a solid material. A mathematical relationship could be established between the velocity of sound through the specimen and its resonant frequency. From this relationship, the modulus of elasticity of the material is determined. For deriving this relationship the solid mediums are considered to be homogenous, isotopic and perfectly elastic. However these relations are also applied to heterogeneous materials like concrete. The velocity of sound V in a solid material is a function of the square root of the ratio of its modulus of elasticity E and its density ρ. Thus

$$V = f\,[gE/\rho]^{1/2}$$

Where g is the acceleration due to gravity

5.2.5 Pulse attenuation method
This is a wave propagation method in which electromagnetic waves, typically in the frequency ranged 500 MHz to 1000 MHz are allowed to propagate through solids and the methodology is found to be useful for subsurface investigation in civil engineering structures and in particular concrete structures. The basic principle is that the attenuation properties of the electromagnetic waves are influenced by the electrical properties of the solid material tested. The dominant properties are the electrical permittivity, which determines the signal

...city, and the electrical conductivity, which determines the signal attenuation. Reflections ... refractions of the radar wave will occur at inter faces between different materials and the ...al returning to the surface antenna can be interpreted to provide an evaluation of the ...erties and geometry of subsurface features. Limited use of radar has been seen in the ...stigation of tunnel linings in India. It is increasingly recognized that there are many ...ctural applications for the radar system as listed below:

- Determination of major construction features
- Assess element thickness
- Locate reinforcing bars
- Locate moisture
- Locate voids, honey combing/cracking
- Locate chlorides
- Size of voids
- Estimate chloride concentration
- Locate rebar corrosion

...6 Pulse echo method

...siderable developments have taken place in marking commercially available instrument ... pulse echo technique and this has become popular through extensive research and

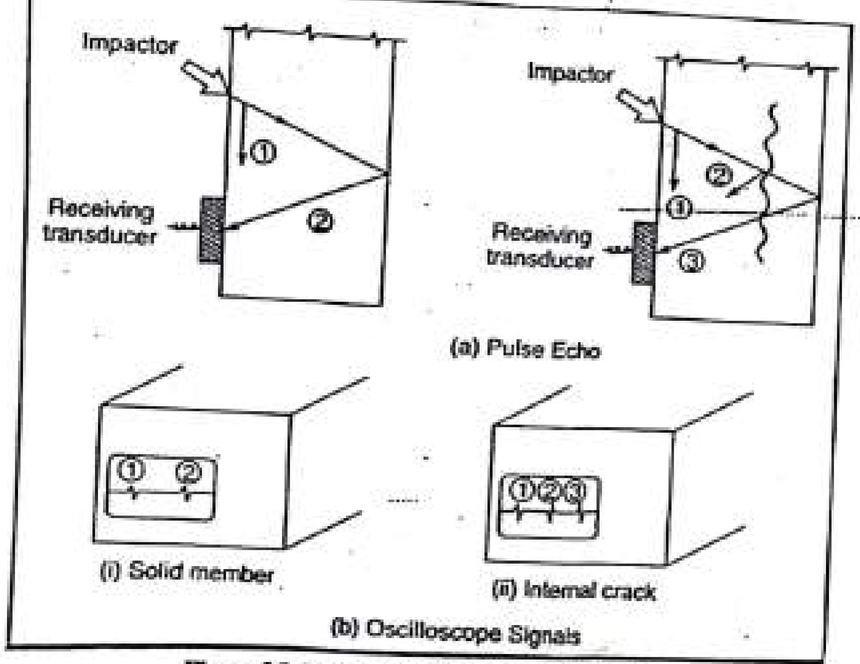

Figure 5.7. Pulse echo method for crack location

practical applications. Originally, pulse echo technique was developed for pile integrity testing and now it is popular for concrete structures and structural elements. An instrument system commercially known as **DOCTER** marked in Denmark is available with a field computer, related software and transducers. The pulse echo and oscilloscope signal techniques are shown in figure 5.7.

The system found to be useful for:

(a) Testing for thickness & flaws in concrete
(b) Testing of wave speed on the surface
(c) Testing for depth of surface opening cracks

5.2.7 Radioactive method

The use of X – rays and gamma rays as non – destructive method for testing properties of concrete is relatively new. X – rays and gamma rays both components of the high-energy region on the electromagnetic spectrum penetrate concrete but undergo attenuation in the process. The degree of attenuation depends on the kind of matter traversed, its thickness, and the wavelength of the radiation. The intensity of the incident gamma rays and the emerging gamma rays after passing through the specimens are measured. These two values are made use of for calculating the density of structural concrete members. Gamma – rays transmission method has been used to measure the thickness of concrete slabs of known density. Gamma radiation source of known intensity is made to pass and penetrate through the concrete. The intensity at the other face is measured. From this thickness of the concrete is calculated.

5.2.8 Nuclear methods

Use of nuclear methods for non – destructive measurement of some properties of concrete is of recent origin. Two principle techniques have been reported, namely **Neutron Scattering Method,** for determining the **moisture content** of concrete and **Neutron Activation Analysis** for the determination of **cement content**. These methods are not suitable for finding out the strength of concrete.

5.2.9 Magnetic methods

Battery operated magnetic devices that can measure the depth of reinforcement cover in concrete and detect the position of reinforcement bars is now available. The apparatus is known as **Cover meter/Pachometer.** This can be used for measuring the cover given in the lightly reinforced sections. Pachometer or covermeter is an electromagnetic device used for determining the location of reinforcing steel embedded in concrete, and to determine the amount of cover over steel. The instrument consists of a search head connected by a cable to a metering unit, which may have a digital or analogue read-out. The purpose of locating the reinforcing steel, prestressing strand, cladding ties etc, in concrete, is to avoid them during core sampling, direct pulse velocity surveys etc, and to make connections for electrochemical measurements. The knowledge of cover depth is essential if it is desired to obtain samples of the concrete at the level of the rebar for chloride ion analysis, pH measurements and the presence of carbonation.

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CHAPTER 6

Semi-Destructive Testing Systems

. INTRODUCTION

tially destructive test are used for assessing the in–situ concrete strength. These tests se localized damage, which do not cause any loss in the performance of the structure. All s in this group are surface zone test which require assess to only one exposed concrete e. Compared to core test the accuracy may not be as good but estimation of strength is nediately available and the testing is less disruptive and damaging. Some of the testing thods are available to assess the quality of concrete–damage of concrete under partial turbance of the surrounding concrete. **The common methods in construction industry given below:**

(a) Penetration Techniques ✓
(b) Pull-out and Pull-off Tests ✓
(c) Core sampling and testing
(d) Break off test
(e) Permeability Test
(f) Half–cell potential survey ✓
(g) Resistivity survey
(h) Carbonation and pH value test
(i) Chloride content test
(j) Abrasion resistance test.

ıddition, some of the tests are used to find the quantity and quality of concrete ingr[illegible]ients are used to help in the damage assessment of existing concrete. **These types of te[illegible] are en below under chemical testing of concrete:**

(a) Determination of cement content
(b) Determination of water content
(c) Determination of water cement ratio
(d) Determination of free chloride content
(e) Determination of pH value of concrete

ıddition, some of the special tests are available to assess the quality of concrete and rebar corrosion-damaged structures and fire damaged structures. These are also discussed in this pter.

.1 Penetration Techniques

: measurement of hardness by probing techniques was first reported during 1954, two hniques were used. In one case, a hammer known as **Simbi** was used to perforate concrete

and depth of borehole was correlated to compressive strength of concrete cubes. In the other technique, the probing of concrete was achieved by blasting with **Spit Pins** and the depth of penetration of the pins was correlated with compressive strength of concrete. The accuracy of the test was found to be ± 25%. Simbi and Spit pins were more affected by the arrangement of coarse aggregate, than the test using rebound hammers.

During 1964 and 1966, a technique known as the **Windsor probes** was advanced for testing concrete in the laboratory and in situ. The Windsor probe is a hardness tester of the surface of the concrete. It is an equipment consisting of a powder activated gun, hardened alloy probes, loaded cartridges, and depth gauge for measuring penetration of probes. The probe is driven in to the concrete by firing of a precision powder charge cartridges. The exposed length is measured by calibrated depth gauge and this is correlated to the strength of concrete cylinder.

The **penetration resistance method** of assessing strength uses a hardened steel probe, which is fired into a concrete surface using a special gun and a standardized explosive charge. The surface is cleared of loose material and a steel plate is positioned adjacent to the probe. A steel cap is screwed onto the probe and the length of probe protruding from the surface is measured using a spring-loaded depth gauge shown in figure 6.1 and the depth of penetration can be determined. It is clear that the forces acting on the probe and the processes occurring within the concrete are extremely complex. As the tip of the probe enters the concrete, it causes local crushing of the surface and creates a shock wave, which results in local spalling. As penetration continues, the forward motion of the probe is resisted by fraction along its sides, concrete is crushed at the tip and the concrete ahead of the probe is compressed in a zone of indeterminate shape and size. The initial kinetic energy of the probe is dissipated by

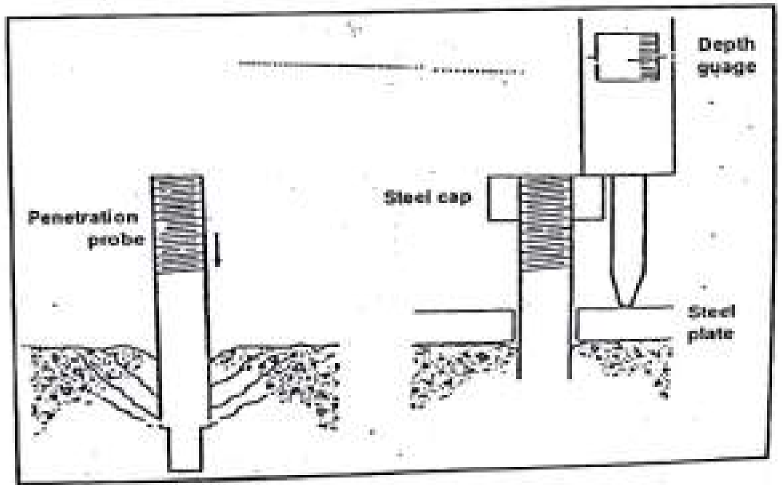

Figure 6.1. Penetration resistance test

Technical: The Windsor HP Probe System is designed to evaluate the compressive strength of concrete in place. It is non-destructive and can be used with equal effectiveness on fresh and mature concrete. 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; the distance required to absorb the specific amount of kinetic energy of the probe determines this. 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.

The combined contributions of both the aggregate and the cement paste to concrete strength are examined by the test. The accuracy of the inferred strengths has been examined in many independent tests and trials. The Windsor HP results correlate well with the concrete strength determinations obtained by conventional means.

For most accurate test results ASTM recommends 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.

6.1.2 Pullout and pull off test

Several tests are available which give estimates of compressive strength by measuring the force required to pull out embedded anchors or to pull off discs glued to the surface. Many of the pull out tests use anchors, which are placed in to the concrete at the time of construction. Pull out test -in this test, either an insert is cast in the concrete or fixed in to a hole, which is drilled in to the concrete. Force required to pull out the insert is measured which is correlated with the compressive strength. Although the results relates to the surface zone only, the advantage is that a more direct measure of strength and a greater depth, compared to the surface, hardness test, and is available. Pull off test -it is based on the measurement of the in-situ tensile strength of concrete by applying a direct tensile force. The method is specially useful in measuring the bond between the overlays. In this test, a metallic disk is glued either to the concrete surface to the surface of partial core. The force required to pull of the disk, causing tensile failure of concrete, is measured and correlated to the strength of concrete. The test requires the care in preparing surface and cause difficulty with damp surfaces.

A pullout test measures the force required to pullout from the concrete a specially shaped rod whose enlarged end has been cast into that concrete. The stronger the concrete, the more is the force required to pullout. The ideal way to use pullout test in the field would be to incorporate assemblies in the structure. These standard specimens could than be pulled out at

any point of time. The force required denotes the strength of concrete. Another way to use pullout test in the field would be to cast one or two large blocks of concrete incorporating pullout assemblies. Pullout test could than be performed to assess the strength of concrete.

Internal fracture test:

It is the pull-out test, which has been most widely used in the field. It was first developed for testing high alumina cement concrete components but further work has been carried out to produce correlation curves for Portland cement concretes. To carry out the test 6mm hole is drilled to a depth of 30-35 mm and the hole is blown out. Holes for individual tests should be at least 100 mm apart, should not be close to an edge and should be drilled away from reinforcement. It is usual to carry out six tests on a member. The average of the six maximum readings is used to read off the mean compressive strength from a standard calibration curve. It is suggested that the method allows concrete strength to be estimated to with in ±30 % at the 95% confidence level. The test causes internal fracture of the concrete and their should be little damage to the surface. After the test the bolt can be sawn of f and the surface made good.

The capo test:

The capo test is a modification of the Lok test which was developed in Denmark. The lok test uses cast- in inserts and confirms to ASTM C 900. The capo test names based on the initial letters of the phrase 'cut and pull out'. To carry out the test, on 18 mm hole is drilled to a depth of 45 mm. The hole is reamed out to form a 25 mm diameter groove at a depth of 25 mm. A ring insert is placed in the hole and expanded in the groove. The insert is connected to the hand-operated hydraulic jack and load-indicating gauge. Connecting to the test equipment is made by a threaded coupling rod 7.2 mm in diameter. The hydraulic jack produces a tensile force in the rod and bears on a reaction ring of internal diameter 55 mm on the concrete surface. The failure surface generated by the test is in the form of the frustum of core from the outer edge of the expanding insert to the inner edge of the reacting edge. The test can be stopped and the load released as soon as the peak is reached. This results in only a fine crack on the surface. Alternatively, the test can be continued until a plug of concrete is removed so that the disc can be recovered and reused.

Limpet test:

The limpet test is a pull off test. An aluminum disc, 50mm in diameter is glued to the concrete surface at the test position using an epoxy resin or other suitable adhesive. The disc is connected to a mechanical loading device by a threaded rod. Load is applied by turning a handle on the side of the machine. Applied load measured by integral strain gauge equipment with digital read-out. The applied force pulls the disc away from the surface with an attached thin layer of concrete. The concrete fails principally in direct tension. It is normal to carry out three tests on a member and calculate a mean pull-off force. The compressive strength can be assessed using suitable calibration curves. Since the layer of concrete detached with the disc is usually only a few mm thick, the results can be strongly influenced by surface effects such as laitance, poor curing or carbonation. This difficulty can be over come by core drilling using a 50 mm diameter barrel to a depth of 20 mm or greater, to penetrate concrete below the surface layer. The central section is not broken off and the aluminium disc is attached to it before pulling in the normal way. This technique is also useful for testing the bond of repairs or overlays. In this case, the core is drilled through the repair or overlay in to the parent concrete.

6.1.3 Core sampling and testing

The core tests are performed under the following situations:

(a) When the standard 28 days cube strength test gives lower strength than acceptable and the primary aim of the core test is to ascertain whether the structural element is of adequate strength.

(b) When it is required to estimate the load carrying capacity of the structure for its safety under change of loading contemplated for the structure.

(c) To detect the segregation, honey combing and to check the bond at construction joint or to verify the thickness of the pavement.

Core test serve as a good support test for any field investigation. Special electric or gasoline driven core drilling machines have been developed to cut cores from the structural members. Cores of 25 mm to 300 mm diameter can be cut including reinforcements. The cores can be tested for compressive strength, chemical analysis, petrographic examination and evaluating physical parameters. The strength testing of cores provide almost a direct evidence on the quality of concrete as it exits on the structure. The final results are influenced by the factors such as: diameter of the core, slenderness ratio, core location, presence of reinforcement and curing condition.

The general procedure includes:

Core location and size - Cores for test should be normally taken at points where minimum strength and maximum stress coincide, for example, from the top surface near mid span for simple beams & slabs or any face near the top of lifts–for columns or walls. In general, the accuracy decreases as the ratio of size of aggregate to core diameter increases, and 150 mm is regarded as the preferred size for aggregates of up to 40 mm, 100 mm cores must not be used if the maximum aggregate size exceeds 25 mm, and this should preferably be less than 20 mm for 75 mm diameter core.

Testing - Each core must be trimmed and capped before visual examination, assessment of voidage, and density determinations.

Trimming–Trimming with a masonry or diamond saw, should give a core of a suitable length with parallel ends, which are normal to the axis of the core. If possible, reinforcement and unrepresentative concrete should be removed.

Capping–Cores should be capped with high alumina cement mortar or sulphur–sand mixture to provide parallel end surfaces normal to the axis of the core. Capping should be kept as thin as possible.

The honey combing, cracks, aggregate distribution, drilling damages and other defects are easily seen on a dry surface. However, for the assessment of type of aggregate, size and then characteristics a wet surface is preferable. Voids between 0.5 to 3 mm are classified as small, 3 to 6 mm sized voids as medium as and greater than 6mm large voids. When the voids are interconnected, they are called honey combing. The precise locations of the reinforcement present in the core must also be recorded.

Density determination:

(a) Measure volume of trimmed core (Vu) by water displacement

(b) Establish density of capping material. (Dc)

(c) Before compression testing, weigh soaked/surface dry capped core in air and water to determine gross weight W_1, and gross volume V_1

(d) If reinforcement is present this should be removed from the concrete after compression testing and the weight Ws and Vs determined.

(e) Calculate saturated density of concrete in the uncapped core from

$$D2 = W1 - Dc\,(V1 - Vu) - Ws/Vu - Vs$$

(f) If no steel is present, Ws and Vs are both zero.

(g) Estimated excess voidage = [(Dp–Da)/(Dp–500)] * 100

When Dp = the potential density based on available values for 28 days old cubes of the same mix.

Da = the actual density from core samples.

Factors influencing the core compressive strength:

(a) Moisture and voids
(b) Length/diameter ratio of core
(c) Diameter of core
(d) Position of cutout concrete on structure
(e) Direction of drilling
(f) Effect of age
(g) Concrete characteristics
(h) Method of capping
(i) Reinforcement

Compression testing:

Compression testing will be carried out at a rate of 15N/mm^2/min in a suitable testing machine and the mode of failure noted. If there is cracking of the caps, or separation of cape and core, the result should be considered as being of doubtful accuracy. Ideally, cracking should be similar all round the circumference of the core, but a diagonal shear crack is considered satisfactory except in short cores or where reinforcement or honeycombing is present.

Estimation of an equivalent cube strength corresponding to a particular core result must initially account for two main factors. There are:

6.1.4 Break off test

A test, which measures the force required to break of a core, has been developed in Norway. The test was originally developed to monitor the early strength of concrete, and utilized a plastic cylindrical insert at the time of casting to from the cores. How ever, it can also be used on drilled cores. A core of 55 mm diameter is drilled to d depth of 70 mm. the annular hole on the surface is enlarged to form a circular group 10 mm wide and 10 mm deep to accept the loading device, as shown in figure 6.3. The load is applied transversely at a depth of 5 mm from the surface. The flexural, tensile stress developed at the base of the core causes it to rupture and the core breaks off.

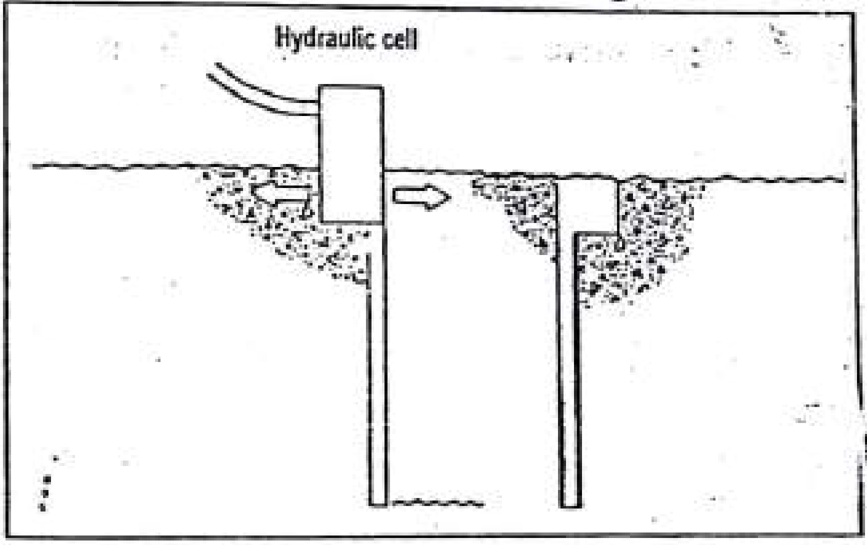

Figure 6.3. Breakoff test

6.1.5 Permeability test

Several types of apparatus have been developed for site use in measuring properties related to permeability. When the test is undertaken on a vertical surface, a means of keeping the cap firmly in contact with the surface has to be provided. Water is introduced into the cap to give a pressure head of 200 mm using a filter funnel. A second port in the cap leads to a horizontal capillary tube. The rate at which water is absorbed into the concrete surface is determined by closing the connecting to the reservoir and measuring the movement of water surface in the capillary tube during a fixed time period. On site the test is usually carried out

10 minutes after filling the cap with water. A diagram of the apparatus is given in Figure 6.4.

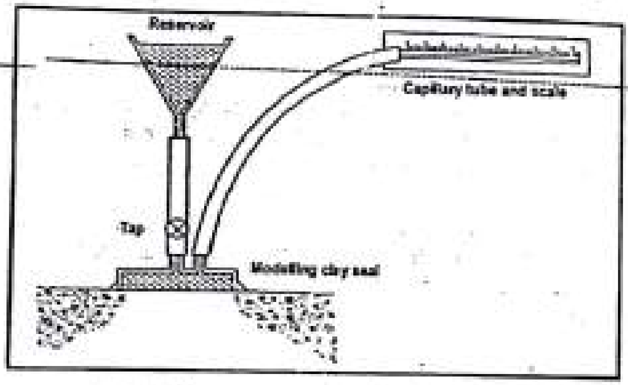

Figure 6.4. Initial surface absorption test

The second test was developed at the building research establishment, U.K in the early 1970 to enable measurements of either air permeation index or water absorption of concrete to be made. Tais is commonly known as the modified Figg's method. In this test, a hole is drilled into the concrete surface and after through cleaning. It is plugged from outside surface by polyether foam and then sealed with a catalyzed silicon rubber. After hardening of rubber, a hypodermic needle is pushed through the silicon plug as shown in figure 6.5. For air permeation, a mercury filled manometer and a hand vacuum pump is attached to the needle. The pressure with in the system is firstly reduced to a standard value and then after isolating the pump, the time for the pressure to raise a standard value is recorded. This time is taken as a measure of the **air permeation index** of concrete. For water absorption, a water head of 100 mm is used. Water is forced in to the assembly using a syringe and one minute after contact between water and concrete, the syringe is shut off. The time for the meniscus in a capillary tube to travel 150 mm is recorded and is taken as a measure of the **water absorption index** of concrete.

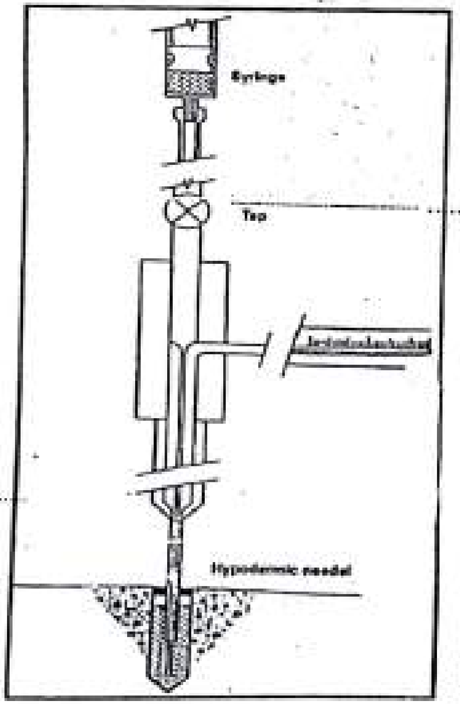

Figure 6.5. Figg's apparatus

A third portable instrument called the Clam uses a circular steel ring bonded to the concrete surface. The main body of the instrument is constant pressure is applied to the surface shown in figure 6.6. The pressure is maintained by means of a piston controlled by a micrometer screw gauge. Readings on the gauge are noted at the beginning and end of a test, which allows the volume of water, which has flowed into the concrete to be calculated.

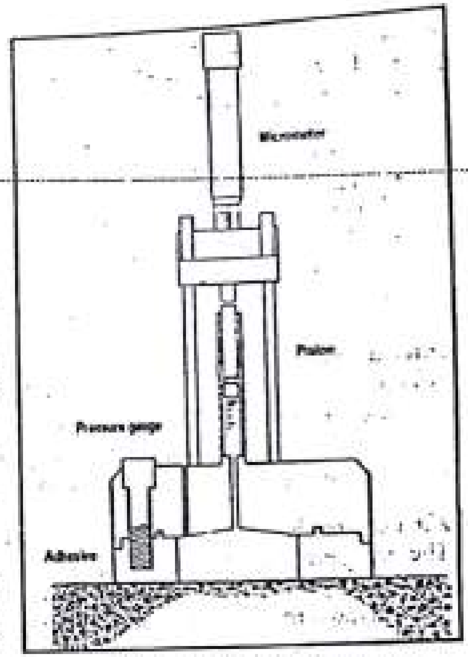

Figure 6.6. Calm apparatus

Advanced permeability tester:
Poroscope-Plus Concrete Air/Water Permeability Tester shown in figure 6.7. The field test for air and water permeability of concrete using the FIGG technique.

Figure 6.7. Poroscope-Plus Concrete Air/Water Permeability Tester

Features:

- Both air and the same instrument measures water permeability of concrete.
- Permeability both at the concrete surface as well as within the concrete mass can be determined.
- Porosity in sealants and surface mortars can be checked.
- The test is non-destructive (only a small plugged hole required) and can be completely carried out on site.
- Each test can be completed in only a few minutes and gives reliable reproducible results.

The test enables meaningful concrete durability prediction to be made.

Internal Test

A hole 10mm diameter × 40mm deep is drilled and plugged leaving a cylindrical test void 10mm diameter × 20mm high situated 20mm below the concrete surface. The time required for air and water to permeate through the test material to the void is used as an index to determine the quality of the concrete under test.

Air Permeability

The air permeability test is always done first since Moisture has a large effect on permeability. Connect the air outlet tube on the instrument to the Luer Connector on the top of the hypodermic needle. Connect the hand operated vacuum pump to the air connection on the top of the instrument and evacuate to greater than 55KPa. The instrument timer in seconds for the vacuum to fall from -55kPa to -50kPa. This time is the Figg number and is a measure of the air permeability of the concrete.

Water Permeability:

Connect the water outlet tube to the Luer socket on the top of the hypodermic and ensure that the fine plastic inner tube is of sufficient lengh to reach the bottom of the test cavitv. After filling the syringe with distilled water connect it to the water inlet on top of the instrument. The water is then forced into the cavity and the air displaced out through the outer tube through the overflow tube which is 4 inches(100 mm) above the surface of the concrete. The cavity is filled when water starts to flow out the overflow tube. The instrument flow sensor and timer then automatically measured the time taken for the water meniscus to travel a distance of 50mm and this time is seconds is displayed on the LCD display of the instrument. The time in seconds is FIGG number for water permeability.

Surface Permeability Test:

Measurements are carried out at the surface by clamping a Stained steel chamber on the smooth surface of the concrete. An exactly dimensioned cup grinding wheel is used to smooth the sealing surface of the concrete if necessary. A measurement of the time required for related amounts of air and water to permeate through the concrete is used as an index of the surface conditions. This time can then be used to determine the condition of any concrete sealant or surface mortar.

Permeability test results have shown that there is a good corrolation with both water/cement ratio and compressive strength of the concrete.

6.1.6 Half–cell potential survey

Corrosion being an electrochemical phenomenon, the electrode potential of steel rebars with reference to a standard electrode under goes changes depending on corrosion activity. A systematic survey on well-defined grid points gives useful information on the presence of probability of corrosion activity. The grid points used for other measurements, namely, rebound hammer and UPV can be used for making the data more meaningful. **The common standard electrodes used are:**

(a) Copper–Copper Sulphate Electrode
(b) Silver–Silver Chloride Electrode
(c) Standard Calomel Electrode

To perform the half–cell potential measurements, electrical connections have to be made with reinforcement and with concrete surrounding it. Electrical connection with the reinforcement is made simply by making a hole in the concrete to expose part of the reinforcement and connecting wire to it. Electrical connection with concrete is made by electrolyte in the $Cu/CuSO_4$ reference electrode which wets both the concrete surface and the conductor to which another wire can be attached. Thus for taking half–cell potential readings, the half–cell *i.e.* $Cu/CuSO_4$ reference electrode is contacted to the surface of the concrete via a sponge soaked in contact fluid and the wires attached to the reinforcement and reference electrode are connected to a high impedance millivoltmeter which gives half–cell potential readings. The measurement consists of giving an electrical connection to the rebar and observing the potential difference between the rebar and a reference electrode in contact with concrete surface. A simple field arrangement is shown in figure 6.8.

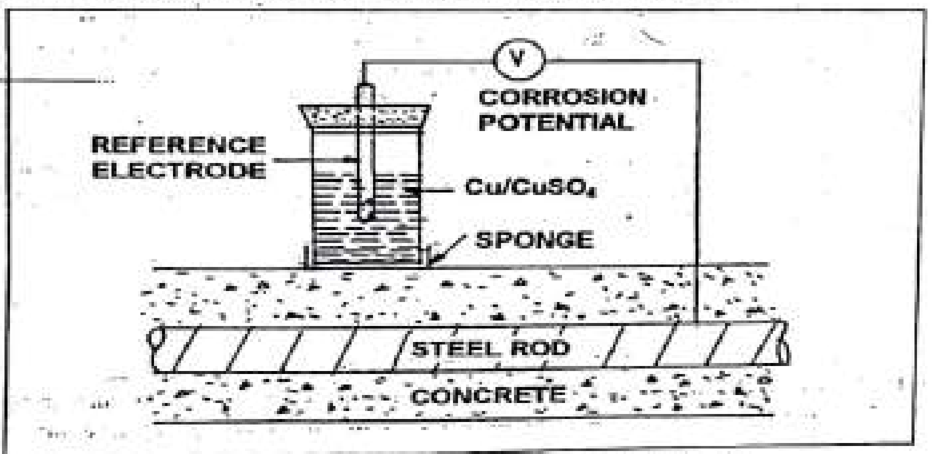

Figure 6.8. Typical setup for potential measurement in rebars

Generally, **the potential value become more and more negative as the corrosion becomes more and more active.** How ever, less negative potential values may also indicate the presence of corrosion activity, if the pH values of concrete are less. The following Table 6.1 shows the general guidelines for identifying the probability of corrosion based on half–cell potential values covered by ASTM C 876-1991.

Figure 6.1. Corrosion risk based on potential readings

Potential readings	*Corrosion*
More negative than –350 mv	> 90%
– 200 mv to –350 mv	50%
More positive than –200 mv	< 10%

However, these interpretation criteria should not be normally utilized under the following conditions:

(a) To evaluate reinforced steel in concrete that has been carbonated/and have highly variable moisture or oxygen content at the level of steel; and
(b) To evaluate galvanized reinforcements which have a range of active/passive potentials completely different of that to bare steel

Advanced testing systems:
The Ohmcor and **Cormap** systems represent more economical methods of evaluating steel reinforcement corrosion. The Cormap Systems use half-cell potential mapping in order to identify areas of probably corrosion. The Ohmcorr System is typically used in conjunction with the Cormap system in order to verify the presence of steel reinforcement corrosion. This older technology is still widely used in the field.

A. CorMap
A simple economical method for identifying areas of probable rebars corrosion.

(a) Easy to use
(b) Detachable electrode extension pieces facilities measurements in hard to reach locations.
(c) High impedance digital meter is designed for tough field conditions
(d) Economical

Technical specifications
Corrosion, which is an electrochemical process, occurs in concrete when oxygen and moisture are present. The actual corrosion is an exchange of energy within different sections of the uncoated reinforcing steel. The relative energy levels can be determined in relation to a reference electrode with a stable electrochemical potential. By connecting a high impedance voltmeter between the reinforcing steel and a reference electrode placed on the concrete surface, a measurement can be made for the half-cell potential at the location of the reference cell. This then is a measurement of the vicinity of the reference cell. The reference cell is

itorage. over 5000 readings

Operating Temperature: 0-50 C
Temperature Reading Range: – 273 - +130 C
Temperature Accuracy: +/-0.5%
Humidity Reading range: 0 - 100%
Humidity Accuracy: +/-5%

Resistivity mapping

corrosion of a specific length of reinforcement is dependent on the algebraic summation e electrical currents originating from the corroding sites on the steel and flowing through moist surrounding concrete to non–corroding sites. Hence the **electrical resistance of** crete plays an important role in determining the magnitude of corrosion at any specific tion. This parameter is expressed in terms of **resistivity in ohm centimeter or kilo ohm imeter** by four-probe resistivity meter shown in figure 6.10.

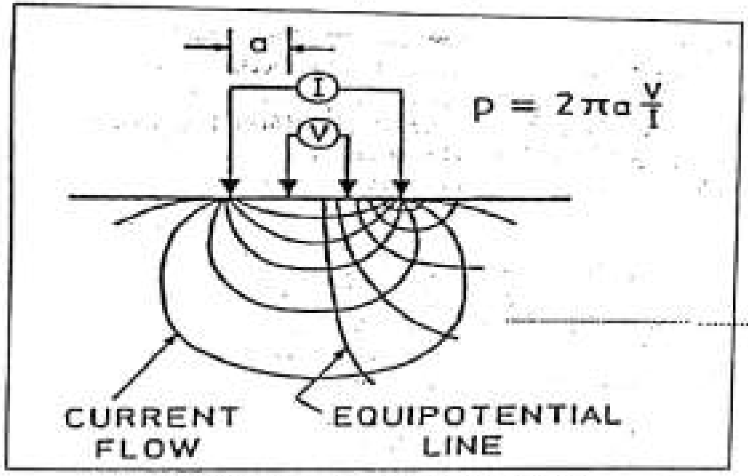

Figure 6.10. Resistivity testing for concrete

method essentially consists of using a 4 probe technique and the probes are inserted into concrete to a depth of 6mm with a spacing of 50mm between the probes in which a own current is applied between two outer probe and the voltage drop between the inner elements is read off allowing for a direct evaluation of resistance R. Using a thematical conversion factor, **resistivity is calculated as $\rho = 2\pi R a$.** The factors, which vern the resistivity values, are:

(a) Constituents of concrete
(b) Chemical contents of concrete
(c) Type of pore structure of concrete

The below Table 6.2 indicates the general guidelines of resistivity values based on which areas having probable corrosion risk can be identified in concrete structures.

Table 6.2. Corrosion risk based on resistivity values

Resistivity ohm cm	*Corrosion probability*
Greater than 20,000	Negligible
10,000 to 20,000	Low
5,000 to 10,000	High
less than 5,000	Very high

Advanced resistivity meter:
RM-8000 OHMCORR - Resistivity Meter to Assess Corrosion Currents in Concrete as shown in figure 6.11.

Features

(a) Assesses damaging corrosion currents in concrete.
(b) Economic and easy to use.
(c) Direct digital readout of resistivity. Measuring from two small holes avoids the problems and errors of surface measurements.

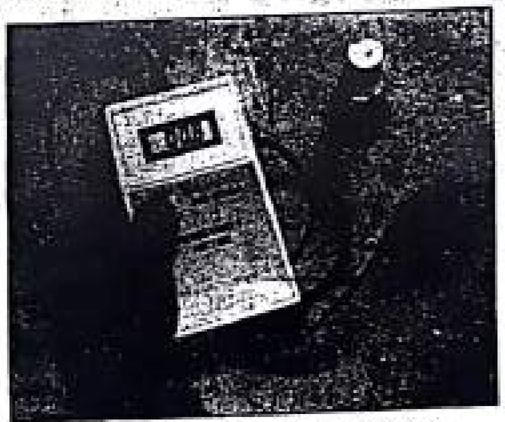

Figure 6.11. Ohmcorr resistivity meter

Used in conjunction with Cor-Map system to produce resistively plots. The OHMCORR, when used in conjunction with The CORMAP system provides an economic and sound means of diagnoses of corrosion in reinforced concrete

Technical Information

The electrical conductivity of concrete is an electrolytic process that takes place by the movements of ions in the cement matrix. This ionic movement will take place when

2.3 Durability:

Definition:

Durability of concrete may be defined as the ability of concrete to resist weathering action, chemical attack, and abrasion while maintaining its desired engineering properties. Different concretes require different degrees of durability depending on the exposures environment and properties desired.

For example, concrete exposed to tidal seawater will have different requirements than an indoor concrete floor. Concrete ingredients, their proportioning, interactions between them, placing and curing practices, and the service environment determine the ultimate durability and life of concrete.

Some important degradation mechanisms in concrete structures include the following:

1. Freeze-thaw damage (physical effects, weathering).
2. Alkali-aggregate reactions (chemical effects).
3. Sulphate attack(chemical effects).
4. Microbiological induced attack(chemical effects).
5. Corrosion of reinforcing steel embedded in concrete (chemical effects).
   a)carbonation of concrete
   b) chloride induced.
6. Abrasion (physical effects).
7. Mechanical loads(physical effects).

2.3.1 Effect of freezing and thawing:

- The most severe climate attack on concrete occurs when concrete containing moisture is subjected to cycles of freezing and thawing.
- The capillary pores in the cement are of such a size that water in them will freeze when the ambient temperatures is below 0^0C.

- The gel pores are so small that water in them does not freeze at normal winter temperatures.
- As water when freezing expands by 9% of its volume, excess water in the capillaries has to move.
- Since the cement paste is relatively impermeable, high pressures are necessary to move the excess water even over quite small distance.
- For normal strength concrete it has been found that movement of the order of 0.2mm is sufficient to require pressures which approach the tensile strength of the paste.

- Concrete can be protected from freeze – thaw damage, by the entrainment of appropriate quantities of air distributed through the cement paste with spacing between bubbles of not more than about 0.4mm.
- The air bubbles must remain partially empty so that they can accommodate the excess water moved to them.
- This will generally be the case since the bubbles constitute the coarsest pore system and are therefore the first to lost moisture as the concrete dries.
- Fully saturated concrete, if permanently submerged, will not need protection against freezing, but concrete which as been saturated and is exposed to freezing, as for example in the tidal range, may not be effectively protected by air-entrainment.

2.3.2 Effect of Temperature:

- The temperature difference within a concrete structure, result in differential volume change.
- When the tensile strain due to differential volume change exceeds the tensile strain capacity of concrete, it will crack.
- The temperature differentials associated with the hydration of cement, affect the mass concrete such as in large columns, piers, footings, dams etc. Whereas the temperature differentials due to changes in the ambient temperature can affect the whole structure.
- The liberation of the heat of hydration of cement causes the internal temperature of concrete to rise during the initial curing period, so that is is usually slightly warmer than its surroundings.
- In thick sections and with rich mixes the temperature differential may be considerable. As the concrete cools it will try to contract.

- Any restraint on the free contraction during cooling will result in tensile stresses which are proportional to the temperature change, coefficient of thermal expansion, effective modulus of elasticity and degree of restraint.
- The more massive the structure, the greater is the potential for temperature differential and degree of restraint.
- Thermally induced cracking can be reduced by controlling the maximum internal temperature, delaying the onset of cooling by insulating the formwork and exposed surfaces, controlling the rate of cooling, and increasing the tensile strain capacity of the concrete.
- Special precautions need to be taken in the desing of structures in which some portions are exposed to temperature changes while the other portions of structures are either

partially or completely protected.

- A drop in temperature may result in the cracking of the exposed element while increase in temperature may cause cracking in the protected portion of the strcture.
- Temperature gradients cause deflection and rotation in structural members; if these are restrained serious stresses can result.
- Allowing for movement by using properly designed contraction joints and correct detailing will help alleviate these problems. If the cracks do form.
- Remedial measures are similar to those for cracks that form after a structure in service.

2.3.3 Effect of chemical:

- The most important constituent of concrete namely cement is alkaline; so it will react with acids or acidic compounds in presence of moisture, and in consequence the matrix becomes weakened and its constituents may be leached out. The concrete may crack, as a result of expansive reactions between aggregate containing active silica and alkalies derived from cement hydration, admixture or external sources(e.g. curing water, ground water, alkaline solutions stored). The alkali – silica reaction results in the formation of a swelling gel, which tends to draw water from other portions of concrete. This causes local expansion nd accompanying tensile stresses which if large may eventually result in the complete deterioration of the structure. Control measures include proper selection of aggregate, use of low-alkali cement and use of pozzolana. Typical symptoms in unreinforced and highly reinforced concrete are *map cracking*, usually in a rough hexagonal mesh pattern and gel excluding from cracks.
- The alkali-carbonate reactions occurs with certain limestone aggregate and usually results in the formation of alkali-silica product between aggregate particiles and the surrounding cement paste. The problem may be minimized by avoiding reactive aggregate, use of smaller size aggregate and use of low-alkali cement.
- When the sulphate bearing waters come in contact with the concrete, the sulphate penetrates the hydrated paste and reacts with hydrated calcium aluminate to form calcium suphoaluminate with a subsequent large increase in volume, resulting in high tensile stresses causing the deterioration of concrete. The blended or pozzolana cements impart additional resistance to sulphate attacks.
- The calcium hydroxide in hydrated cement paste will combine with carbon dioxide in the air to form calcium carbonate which occupies smaller volume tan the calcium hydroxide resulting called *carbonation shrinkage*. This situation may result in significant surface

- The calcium hydroxide in hydrated cement paste will combine with carbon dioxide in the air to form calcium carbonate which occupies smaller volume tan the calcium hydroxide resulting called *carbonation shrinkage.* This situation may result in significant surface grazing and ay be especially serious on freshly placed concrete surface kept warm during winter by improperly vented combustion heaters.

Factors which increase concrete vulnerability to external chemical attacks are,

1. High porosity
2. High permeability and absorption resulting from too high W/C ratio.
3. Unsatisfactory grading of aggregate.
4. Cement compaction.
5. Improper choice of cement type for condition of exposure.
6. Inadequate curing period.
7. Exposure to alternate cycles of wetting and drying and to the lesser extended of heating and cooling.
8. Increased fluid velocity which may bring about both replenishment of the aggressive species and increases in the rate of leaching.
9. Suction forces which may caused by drying on one or more faces of a section.
10. Unsatisfactory choice of shape and surface to volume ratio of concrete structure.

2.3.4 Effect of Corrosion:

2.3.4.1 Formation of white patches

CO_2 reacts with $Ca(OH)_2$ in the cement paste to form $CaCO_3$. The free movement of water carries the unstable $CaCO_3$ towards the surface and forms white patches. It indicates the occurrences of carbonation.

2.3.4.2 Brown patches along reinforcement

When reinforcement starts corroding, a layer a ferric oxide is formed. This brown product resulting from corrosion may permeate along with moisture to the concrete surface without cracking of the concrete.

2.3.4.3 Occurrence of cracks

The increase in volume exerts considerable bursting pressure on the surrounding concrete resulting in cracking. The hair line crack in the concrete surface lying directly above the reinforcement and running parallel to it is the positive visible indication that reinforcement is corroding. These cracks indicate that the expanding rust had grown enough to split the concrete.

reinforcement and running parallel to it is the positive visible indication that reinforcement is corroding. These cracks indicate that the expanding rust had grown enough to split the concrete.

2.3.4.4 Formation of multiple cracks

As corrosion progresses, formation of multiple layers of rust on the reinforcement which in turn exert considerable pressure on the surrounding concrete resulting in widening of hair cracks. In addition, a number of new hair cracks are also formed. The bond between concrete and the reinforcement is considerably reduced. There will be a hollow sound when the concrete is tapped at the surface with a light hammer.

2.3.4.5 Snapping of bars

The continued reduction in the size of bars results in snapping of the bars. This will occur in ties/stirrups first. At this stage, size of the main bars is reduced.

2.3.4.6 Buckling of bars and bulging of concrete

The spalling of the cover concrete and snapping of ties causes the main bars to buckle. This results in bulging of concrete in that region. This follows collapse of the structure. When corrosion of reinforcement starts, the deterioration is usually slow but advances in geometrical progression. Corrosion can also cause structural failure due to reduced C/S and hence reduced load carrying capacity. It is possible to arrest the process of corrosion at any stage by altering the corrosive environment in the vicinity of the reinforcement.

2.4 Design Errors and Construction Errors:

2.4.1 Design Errors

Design errors may be divided into two general types:

1. Those resulting from inadequate structural design
2. Those resulting from lack of attention to relatively minor design details.

Each of the two types of design errors is discussed below.

(1) Inadequate structural design.

(a) Mechanism. The failure mechanism is simple – the concrete is exposed to greater stress than it is capable of carrying or it sustains greater strain than its strain capacity.

(b) Symptoms. Visual examinations of failures resulting from inadequate structural design will usually show one of two symptoms.

1. First, errors in design resulting in excessively high compressive stresses will result in spalling. Similarly, high torsion or shear stresses may also result in spalling or cracking.
2. Second, high tensile stresses will result in cracking.

To identify inadequate design as a cause of damage, the locations of the damage should spalling. Similarly, high torsion or shear stresses may also result in spalling or cracking.

2. Second, high tensile stresses will result in cracking.

To identify inadequate design as a cause of damage, the locations of the damage should be compared to the types of stresses that should be present in the concrete. For example, if spalls are present on the underside of a simple-supported beam, high compressive stresses are not present and inadequate design may be eliminated as a cause. However, if the type and location of the damage and the probable stress are in agreement, a detailed stress analysis will be required to determine whether inadequate design is the cause. Laboratory analysis is generally not applicable in the case of suspected inadequate design. However, for rehabilitation projects, thorough petro graphic analysis and strength testing of concrete from elements to be reused will be necessary.

(c) Prevention.

Inadequate design in prevented by thorough and careful review of all design calculations. Any rehabilitation method that makes use of existing concrete structural members must be carefully reviewed.

(2) Poor design details

A structure may be adequately designed to meet loadings and other overall requirements, poor detailing may result in localized concentrations of high stresses in otherwise satisfactory concrete. These high stresses may result in cracking that allows water or chemicals access to the concrete. In other cases, poor design detailing may simply allow water to pond on a structure, resulting in saturated concrete. In general, poor detailing does not lead directly to concrete failure; rather, it contributes to the action of one of the other causes of concrete deterioration described in this chapter. Several specific types of poor detailing and their possible effects on a structure are described in the following paragraphs. In general, all of these problems can be prevented by a thorough and careful review of plans and specifications for the project. In the case of existing structures, problems resulting from poor detailing should be handled by correcting the detailing and not by simply responding to the symptoms.

(a) Abrupt changes in section.

Abrupt changes n section may cause stress concentrations that may result in cracking. Typical examples would include the use of relatively thin sections such as bridge decks rigidly tied into massive abutments or patches and replacement concrete that are not uniform in plan dimensions.

(b) Insufficient reinforcement at reentrant corners and openings.

Reentrant corners and openings also tend to cause stress concentrations that may cause cracking. In this case, the best prevention is to provide additional reinforcement in areas where stress concentrations are expected to occur.

concentrations are expected to occur.

(c) Inadequate provision for deflection.

Deflection in excess of those anticipated may result in loading of members or sections beyond the capacities for which they were designed. Typically, these loadings will be induced in walls or partitions, resulting in cracking.

(d) Inadequate provision for drainage.

Poor attention to the details of draining a structure may result in the ponding of water. This ponding may result in leakage or saturation of concrete. Leakage may result in damage to the interior of the structure or in staining and encrustations on the structure. Saturation may result in severely damaged concrete if the structure is in an area that is subjected to freezing and thawing.

(e) Insufficient travel in expansion joints.

Inadequately designed expansion joints may result in spalling of concrete adjacent to the joints. The full range of possible temperature differentials that a concrete may be expected to experience should be taken into account in the specification for expansion joints. There is no single expansion joint that will work for all cases of temperature differential.

(f) Incompatibility of materials.

The use of materials with different properties (modulus of elasticity or coefficient of thermal expansion) adjacent to one another may result in cracking or spalling as the structure is loaded or as it is subjected to daily or annual temperature variations.

(g) Neglect of creep effect.

Neglect of creep may have similar effects as noted earlier for inadequate provision for deflections. Additionally, neglect of creep in prestressed concrete members may lead to excessive prestress loss that in turn results in cracking as loads are applied.

(h) Rigid joints between precast units.

Designs utilizing precast elements must provide for movement between adjacent precast elements or between the precast elements and the supporting frame. Failure to provide for this movement can result in cracking or spalling.

(i) Unanticipated shear stresses in piers, columns, or abutments.

Through lack of maintenance, expansion bearing assembles are allowed to become frozen, horizon-tal loading may be transferred to the concrete elements supporting the bearings. The result will be cracking in the concrete, usually compounded by other problems which will be caused by the entry of water into the concrete.

2.4.2 Construction Errors:

Failure to follow specified procedures and good practice or outright carelessness may lead to a number of conditions that may be grouped together as construction errors. Most of these errors do not lead directly to failure or deterioration of concrete. Instead, they enhance the adverse impacts of other mechanisms. Each error will be briefly described along with preventative methods. In general, the best preventive measure is a thorough knowledge of what these construction errors are, plus an aggressive inspection program. It should be noted that errors of the type described in this section are equally as likely to occur during repair or rehabilitation projects as they are likely to occur during new construction.

(a) Adding water to concrete. Water is usually added to concrete in one or both of the following circumstances:

1. First, water is added to the concrete in a delivery truck to increase slump and decrease emplacement effort. This practice will generally lead to concrete with lowered strength and reduced durability. As the w/c of the concrete increases, the strength and durability will decrease.
2. In the second case, water is commonly added during finishing of flatwork. This practice leads to scaling, crazing, and dusting of the concrete in service.

(b) Improper alignment of formwork.

Improper alignment of the formwork will lead to discontinuities on the surface of the concrete. While these discontinuities are unsightly in all circumstances, their occurrence may be more critical in areas that are subjected to high-velocity flow of water, where cavitations erosion may be induced, or in lock chambers where the "rubbing" surfaces must be straight.

(c) Improper consolidation.

Improper consolidation of concrete may result in a variety of defects, the most common being bugholes, honeycombing, and cold joints.

"Bugholes" are formed when small pockets of air or water are trapped against the forms. A change in the mixture to make it less "sticky" or the use of small vibrators worked near the form has been used to help eliminate bugholes.

Honeycombing can be reduced by inserting the vibrator more frequently, inserting the vibrator as close as possible to the form face without touching the form, and slower withdrawal of the vibrator. Obviously, all of these defects make it much easier for any damage-causing mechanism to initiate deterioration of the concrete.

Frequently, a fear of "overconsolidation" is used to justify a lack of effort in consolidating concrete. Overconsolidation is usually defined as a situation in which the consolidation effort causes all of the coarse aggregate to settle to the bottom while the paste rises to the surface. If this situation occurs, it is reasonable to conclude that there is a problem of a poorly proportioned concrete rather than too much consolidation.

(d) Improper curing.

Curing is probably the most abused aspect of the concrete construction process. Unless concrete is given adequate time to cure at a proper humidity and temperature, it will not develop the characteristics that are expected and that are necessary to pro-vide durability. Symptoms of improperly cured concrete can include various types of cracking and surface disintegration. In extreme cases where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

(e) Improper location of reinforcing steel.

This section refers to reinforcing steel that is improperly located or is not adequately secured in the proper location. Either of these faults may lead to two general types of problems.

1. First, the steel may not function structurally as intended, resulting in structural cracking or failure. A particularly prevalent example is the placement of welded wire mesh in floor slabs. In many case, the mesh ends up on the bottom of the slab which will subsequently crack because the steel is not in the proper location.

2. The second type of problem stemming from improperly located or tied reinforcing steel is one of durability. The tendency seems to be for the steel to end up near the surface of the concrete. As the concrete cover over the steel is reduced, it is much easier for corrosion to begin.

(f) Movement of formwork

Movement of formwork during the period while the concrete is going from fluid to a rigid material may induce cracking and separation within the concrete. A crack open to the surface will allow access of water to the interior of the concrete. An internal void may give rise to freezing or corrosion problems if the void becomes saturated.

(g) Premature removal of shores or reshores.

If shores or reshores are removed too soon, the concrete affected may become overstressed and cracked. In extreme cases there may be major failures.

(h) Settling of the concrete.

During the period between placing and initial setting of the concrete, the heavier components of the concrete will settle under the influence of gravity. This situation may be aggravated by the use of highly fluid concretes. If any restraint tends to prevent this settling, cracking or separations may result. These cracks or separations may also develop problems of corrosion or freezing if saturated.

(i) Settling of subgrade.

If there is any settling of the subgrade during the period after the concrete begins to become rigid but before it gains enough strength to support its own weight, cracking may also occur.

(j) Vibration of freshly placed concrete.

Most construction sites are subjected to vibration from various sources, such as blasting, pile driving, and form the operation of construction equipment.

Freshly placed concrete is vulnerable to weakening of its properties if subjected to forces which disrupt the concrete matrix during setting.

(k) Improper finishing of flat work.

The most common improper finishing procedures which are detrimental to the durability of flat work are discussed below.

(1) Adding water to the surface. Evidence that water is being added to the surface is the presence of a large paint brush, along with other finishing tools. The brush is dipped in water and water is "slung" onto the surface being finished.

(2) Timing and finishing. Final finishing operations must be done after the concrete has taken its initial set and bleeding has stopped. The waiting period depends on the amouts of water, cement, and admixtures in the mixture but primarily on the temperatures of the concrete surface. On a partially shaded slab, the part in the sun will usually be ready to finish before the part in the shade.

(3) Adding cement to the surface. This practice is often done to dry up bleed water to allow finishing to proceed and will result in a thin cement-rich coating which will craze or flake off easily.

2.5 Effect of Cover Thickness

There is a substantial experience which relates durability and the amount of water. The thicker the cover over the steel is, the longer it will take the chloride ions to reach the steel and reduce the pH and passivity provided by the cement. However, excessive cover can led to the development of a few wide cracks under overstress, whereas a thinner cover results in many small cracks.

As opposed to the above mentioned facts, which appear to justify the rigid rules on cover, are the fallowing facts.

- Ships built during World War I and II had covers of only about 20mm, yet they did not suffer corrosion steel.

- In the erstwhile USSR, many floating dry-docks have been built with covers of 15 and 20mm with highly successful durability over many years of adverse exposure.

It is confirmed opinion that the impermeability of the cover is of major importance. The thickness should be related to the steel bar diameter and the maximum size of the coarse aggregate.

The general factors affecting permeability, such as cement content, water/cement ratio, compaction and consolidation of the concrete, and curing are important. While many fee that prestressing steel should have a greater cover than non-stressed steel, because of the more serious consequences of corrosion. Prestressed concrete pilling by hundreds of thousands are rendering completely successful service with only 4-6cm of cover. Other factors affecting cover are the tolerances of placement of steel and forms, and the depths of honeycombs and bug holes and other surface defects.

Lack of adequate cover contributes much to corrosion in an aggressive environment. A well compacted and continuous, even if thin, cover of good quality concrete on reinforcement is sufficient to protect it from corrosion. The following are the reinforcement thickness of covers for various levels of exposure.

- For normal exposure : At least 50mm thickness
- For moderate exposure : At least 40mm thickness
- For mild exposure : At least 30mm thickness
- For normal exposure : At least 20mm thickness

Cover Meter

When a metallic object is placed in the varying magnetic field of coil, the field induces eddy currents in the object. These eddy currents in turn produce an additional magnetic field in the vicinity of the magnetic object. A magnetic field gets superimposed and the magnetic field near the coil also gets modified in the presence of metal. This modification has the same effect as would be obtained if the characteristic of the coil itself had been changed. The change depends upon the electrical conductivity, dimension, magnetic permeability, presence of discontinuity such as crack, frequency of the field of the coil, size and shape of the coil, and the distance of the coil from the metallic object.

It is possible to measure the cover thickness for a known diameter by keeping all other upon the electrical conductivity, dimension, magnetic permeability, presence of discontinuity such as crack, frequency of the field of the coil, size and shape of the coil, and the distance of the coil from the metallic object.

It is possible to measure the cover thickness for a known diameter by keeping all other parameters constant. By placing the soil at two different distances from the rebar, both the cover thickness and the diameter of the rebar can be found.

2.6 Effect of Cracking

The formation of cracks is dangerous for protection against corrosion. Once concrete cracks, the external depassivating agents can penetrate deep into concrete and set off the process of corrosion. Cracks running transversely to the reinforcement are less harmful than the longitudinal cracks along the reinforcement.

Thus in the order to induce the process of corrosion and to keep it going, at least one of the following conditions must exist in any RC structure.

- Chloride ion concentration in excess of the threshold value at the interface of the reinforcement and concrete or sufficient advancement of the carbonation front to destroy the passivity of the ferric oxide surface layer of the reinforcement.
- Adequate moisture in the concrete to facilitate the movement of chloride ions and provide a conduction path between the anodic and the cathodic areas on the steel.
- Sufficient oxygen supply to the cathodic areas in order to maintain such areas in a depolarized condition.
- Difference in electrochemical potentials at the surface of the reinforcement.
- Low values of electrical resistivity of concrete.
- Relative humidity in the range 50-70%.
- Higher ambient temperature.

3.5.4. Water–cement ratio

The water–cement ratio plays a predominant role in the permeability characteristics of concrete and therefore, the draft IS 456–2000 stipulates water–cement ratio requirement for different conditions of exposure as given in Table 3.4.

Table 3.4. Maximum water-cement ratio for different exposure conditions

S.No	Exposure	Plain concrete maximum free w/c	Reinforcement concrete minimum free w/c
1	Mild	0.60	0.55
2	Moderate	0.60	0.50
3	Severe	0.50	0.45
4	Very severe	0.45	0.45
5	Extreme	0.40	0.40

3.6 CORROSION PROTECTION TECHNIQUES

The detailed analysis of the factors that influence corrosion mechanism and process indicates that corrosion protection requires a multiple approach. There are many possible approaches as listed below:

(a) Coating to reinforcement
(b) Galvanized reinforcement
(c) Improving metallurgically by addition of certain elements
(d) Using stainless steel
(e) Using non-ferrous reinforcement
(f) Using corrosion inhibitors
(g) Coating to concrete
(h) Cathodic protection, either by means of impressed unit or by sacrificial anodes.
(i) Electrochemical chloride removal
(j) Improving the cover concrete.

3.6.1 Coating to reinforcement

The objective of coating to steel rebar is to provide a sufficiently durable barrier to aggressive materials such as chlorides. Initially, the bar in shot blasted to remove mill-scale. This ensures an adequate bond between the epoxy & the steel. The bar is then heated to a controlled temperature before passing through a spray booth. Hence, electro-statically charged epoxy powder particles are deposited evenly on the surface of the bar. Shortly after spraying, the epoxy starts to use and hardens sufficiently for the bars to be handled. The coating thickness typically varies from 130 micron to 300 micron.

3.6.2 Galvanized reinforcement

Galvanized reinforcement consists of standard black bar, hot dipped in molten zinc. This process forms a coating which is metallurgically bonded to the surface of the parent metal. The surface of the zinc reacts with calcium hydroxide in the concrete to form a passive layer,

preventing corrosion. However, it is a more difficult process to control as uneven coatings can form and it is more expensive.

3.6.3 Improving metallurgically by adding certain elements

The bars can be improved for its corrosion resistance by adding certain elements such as chromium and copper during the formation/process itself.

3.6.4 Stainless steel reinforcement

Stainless steel is the name given to a family of corrosion resistant steels containing a minimum of 12 % chromium. On contact with air, the chromium forms a thin oxide layer on the surface of steel. This is passive and resists corrosion. The austenic group of steels has superior corrosion resistance and is the most suitable for use in reinforced concrete.

3.6.5 Non-ferrous reinforcement

Currently a number of manufacturers who are developing non-ferrous reinforcement as an alternative to the conventional steel in traditional structures. A range of man -made fibers is used, the most common being glass, carbon and aramid. The fibers are used either in the form of ropes or combined with suitable resins to form rods. It may be seen that aramid, glass and carbon fibers have ultimate tensile stresses well in excess of that of reinforcing steel and well resistance to corrosion.

3.6.6 Corrosion inhibitors

Certain admixtures can be made to inhibit corrosion of the reinforcement in the presence of chlorides. Example:

(a) The addition of calcium nitrate extends the time to corrosion initiation.
(b) Total corrosion of samples with calcium nitrate is substantially less
(c) The corrosion rate, once corrosion is initiated is less with calcium nitrate.

3.6.7 Concrete coatings

A concrete surface coating of silane-siloxane type was evaluated for its corrosion performance. By several test conducted and the results show the chloride penetration depth is minimized in coated specimen compared with uncoated specimen.

3.6.8 Cathodic protection

Cathodic protection is a technique by which the electrical potential of the steel is increased to a level at which corrosion cannot take place. It is widely used for both steel and concrete offshore structures, while on land it has been used for the protection of pipelines and similar structures. Two different methods are employed, an impressed current and the use of sacrificial anodes. In the first the structure is connected to the negative terminal of a DC power source, ideally using an anode, which does not corrode. In the second the reinforcement is connected to anodes with a more negative corrosion potential than steel, such as Zinc or aluminium. In both cases electrical continuity of the reinforcement is required. The principal of cathodic protection have been used to remove chlorides from contaminated concrete.

3.6.9 Electrochemical chloride removal

This is another emerging area with lot of potentialities. This technique needs only a temporary installation lasting few days. 20 to 50 % of the chloride present in concrete can be removed. An electrical current in the range of 1 to 5 A/m^2 is needed.

3.6.10 Improving the concrete

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

(a) High cement content
(b) Low water/cement ratio
(c) Suitable minimum thickness of cover to the reinforcement
(d) Careful curing.

4.8.4.1 Methods of Corrosion Protection

The following are some of the methods for protecting steel from corrosion

- Protective coatings for reinforcement
- Cathodic protection
- Corrosion Resistant steel
- Corrosion inhibitors

4.8.5. Protective coatings for reinforcement

This is an effective means to combat corrosion in such environment where ordinary concrete with surface coating is not able to protect reinforcement against corrosion. The surface coating for the reinforcement will increase the protection against corrosion.

There are several methods of providing protective coating to the reinforcement. The important ones are:

i. Cement Slurry Coating

- Cement Slurry Coating provides short-term protection until placement in concrete.
- Several methods have been developed for an effective corrosion protection using cement slurry.
- One such coating is a mixture of cement, condensed silica and polymer dispersion.
- This mixer found to be impermeable to water, chlorides and carbon-di-oxide.

ii. Epoxy Coating

- Epoxy coating is formed by application of an epoxy resin with appropriate curing agents catalysts, pigments and flow control agents.
- Fusion bonding using the electrostatic process is the recent development.
- Fusion bonded epoxy coating provides long-term protection against corrosion.
- Though the cost is relatively high, it is the one which is the most effective in high alkaline and chloride contaminated environment.

iii. Plastic Coating

- Similar to epoxy coating, the plastic coatings are very effective in preventing corrosion of reinforcement even in high alkaline or chloride contaminated environment.
- However, the reduction in bond between plastic coated bar and the concrete is quite substantial and hence plastic coating cannot be

considered as a solution for prevention of corrosion which cannot be solved by conventional methods.

iv. Galvanizing

- Galvanizing gives protection to the reinforcement against corrosion, by means of metallic coating such as zinc.
- However, in case of corrosion due to excessive chlorides, the effect of galvanizing protection is reduce and hence is not advisable in highly chloride contaminated environments.

Cathodic protection

- Cathodic protection interferes with the natural action of the electrochemical cells that are responsible for corrosion.
- Cathodic protection can be effectively applied to control corrosion of surfaces that are immersed in water or exposed to soil.
- Cathodic protection in its classical form cannot be used to protect surfaces exposed to the atmosphere.
- The use of anodic metallic coatings such as zinc on steel(galvanizing) is, however, a form of cathodic protection, which is effective in the atmosphere.

3.3 CORROSION PROCESS

The corrosion process of reinforcement embedded in concrete has two distinct periods namely, **initiation period and propagation period.** Figure 3.1 shows these periods schematically as service life of a structure. The initiation period, during which the metal, have been embedded in concrete remains passive whilst, with the concrete, environmental changes are taking place that may ultimately terminate passively. The propagation period, during which begins at the moment of depassivation and involves the propagation of corrosion at a significant rate, until a final state is reached, when the structure is no longer considered acceptable on grounds of structural integrity, serviceability or appearance.

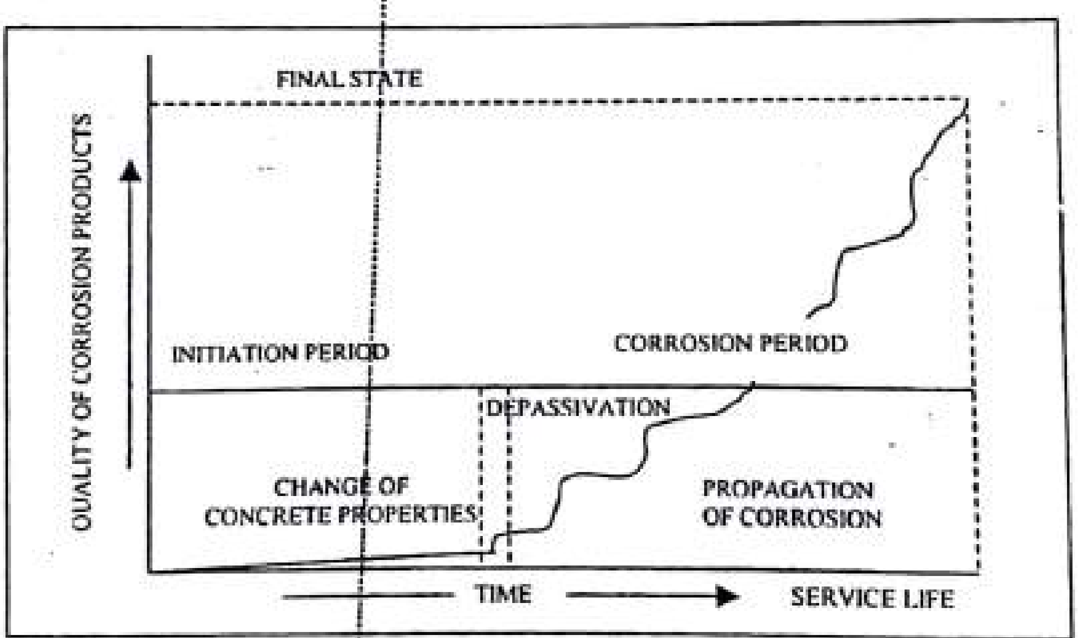

Figure 3.1. Service life model for corrosion affected structures

3.3.1 Initiation period

That is the time taken to initiate corrosion, which can be caused either by ingress of carbon dioxide or chloride ions. The reinforcing steel embedded in concrete is well protected by providing adequate cover thickness and good quality concrete with certain minimum cement content.

Influencing factors:

The factors, which influence the corrosion initiation, are:

(a) The environment to which the structure is exposed
(b) The cover thickness
(c) Quality of cover concrete in terms of its alkalinity, permeability and diffusion characteristics
(d) The type of steel
(e) Critical chloride in concrete and
(f) Presence of cracks

3.3.2 Propagation period

After the initiation of corrosion, the propagation begins and this period has two distinct processes. One is that the corrosion follows **an electrochemical** process and the other is the **physical process** due to which damage to concrete occurs. During the propagation period, the corrosion progresses at a rate depending on the availability of oxygen and moisture.

Electrochemical process:

The **electrochemical process corrosion** can be considered as the metallurgy in reverse. Steel is produced from the basic iron ore which is oxide in nature. Energy is added to make the ore into steel and during the electrochemical process by corrosion, electrons get liberated dissipating the energy added and thereby the steel goes back to its oxidised form. This is illustrated in figure 3.2.

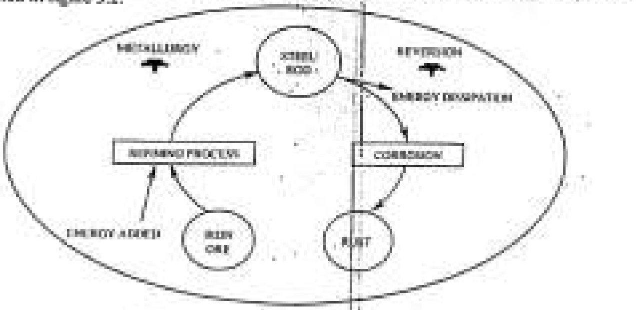

Figure 3.2. Metallurgy in reverse is corrosion

In respect of reinforcing steel, **the electrochemical process can occurs in two situations.** Immediately after production in the factory, the rods come out in light blue colour. During transportation and storage, a thin oxide film gets formed and this acts as a passive layer. However, during handling, it is likely that the passive layer may get mechanically destroyed creating locally depassivated spots. Such spots in the presence of water and oxygen create galvanic cells, forming anodic and cathodic sites and highly localised corrosion takes place. Such corrosion is known as localised pitting corrosion. The pitting corrosion in embedded steel is schematically illustrated in figure 3.3.

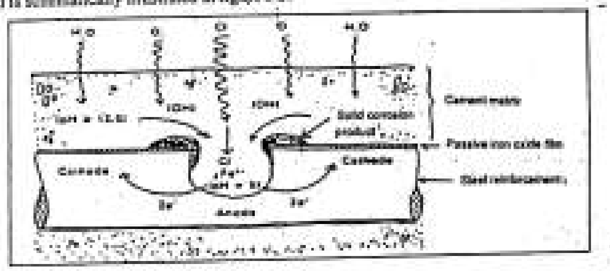

Figure 3.3. Occurrence of pitting corrosion

The process follows an electrochemical phenomenon creating a potential gradient and current flow between the anodic and cathodic location. It is necessary that the reinforcing rods are well protected during storage and this can be achieved by keeping the rods under covered sheds, placing them on wooden supports and providing a cement slurry coating. **Another situation of electrochemical process is when the rod is embedded in concrete.** Corrosion process of rebar in concrete is illustrated in figure 3.4. In this situation, the electrochemical process progress by forming the anodic and cathodic sites, involving chemical reactions.

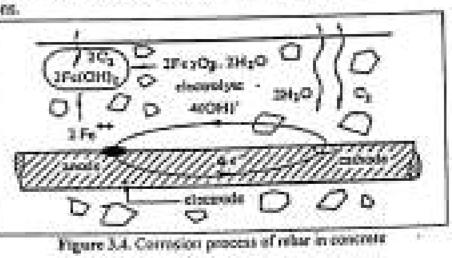

Figure 3.4. Corrosion process of rebar in concrete

A moist concrete matrix forms an acceptable electrolyte and the steel reinforcement provides the anode and cathode. Electrical current flows between the cathode and anode, and the reaction results in an increase in metal volume as the Fe(Iron) is oxidized into $Fe(OH)_2$ and $Fe(OH)_3$ and precipitates as Fe OH (rust).

The electrochemical equations as given below:

$$Fe \rightleftharpoons 2e^- + Fe^{++} \quad \text{(ANODE)}$$

$$1/2O_2 + H_2O + 2e^- \rightleftharpoons 2(OH^-) \quad \text{(CATHODE)}$$

$$\rightarrow Fe(OH)_2$$

$$4Fe(OH)_2 + 2H_2O + O_2 \longrightarrow 4Fe(OH)_3$$

RED RUST

$$3Fe + 8OH^- \longrightarrow Fe_3O_4 + 8e^- + 4H_2O$$

BLACK RUST

The electrochemical process is greatly influenced by the pH value of concrete and the chloride. Water and oxygen must be present for the reaction of take place.

1.0 Maintenance :

Maintenance is preventive in nature. Activities include inspection and works necessary to fulfill the intended function or to sustain original standard of service. The maintenance of structure is done to meet the following objective

- Prevention of damages due to natural agencies and to keep them in good appearance and working condition.
- Repair of the defects occurred in the structure and strengthen them, if necessary.

1.0.1 The Maintenance work is broadly classifies as

a) Preventive Maintenance

b) Remedial Maintenance

c) Routine Maintenance

d) Special Maintenance

a) Preventive Maintenance

- The maintenance work done before the defects occurred or damage developed in the structure is called preventive maintenance.
- It includes thorough inspection, planning the programs of maintenance and executing the work
- It depends upon the specifications, condition and use of structure.

b) Remedial Maintenance

- It is the maintenance done after the defects or damage occurs in the structure. It involves the following basic steps.
  - Finding the deterioration
  - Determining the causes
  - Evaluating the strength of the existing structure
  - Evaluating the need of the structure
  - Selecting and implementing the repair procedure

c) Routine Maintenance

- It is the service maintenance attended to the structure periodically.
- The nature of work done and interval of time at which it is done depends upon specifications and materials of structure, purpose, intensity and condition of use.
- It includes white washing, parch repair to plaster, replacement of fittings and fixtures, binding of road surface.

e) Special Maintenance

- It is the work done under special condition and requires sanction and performed to rectify heavy damage.
- It may be done for strengthening and updating of the structure to meet the new condition of usage or to increase its serviceability.

1.0.3 Facets of maintenance:

Maintenance operations have many facets such as

a) Emergency maintenance: Necessitated by unforeseen breakdown drainage or damage caused by natural calamity like fire, floods, cyclone earthquake etc.

b) Condition Based maintenance: Work initiated after due inspection

c) Fixed time maintenance: Activities repeated at predetermined intervals of time.

d) Preventive maintenance: This is intended to preserve by preventing failure and detecting incipient faults (Work is done before failure takes place)

e) Opportunity maintenance: Work did as and when possible within the limits of operation demand.

f) Day-to-Day care and maintenance

g) Shut down maintenance: Thorough overhaul and maintenance after closing a facility.

h) Improvement plans: This is essentially maintenance operation wherein the weak links in the original construction are either replaced by new parts or strengthened.

1.0.4 Importance of Maintenance

- Improves the life of structure
- Improved life period gives better return on investment
- Better appearance and aesthetically appealing
- Better serviceability of elements and components
- Leads to quicker detection of defects and hence remedial measures
- Prevents major deterioration and leading to collapse
- Ensures safety to occupants
- Ensures feeling of confidence on the user
- Maintenance is a continuous cycle involves every element of building science namely

  - Structural
  - Electrical wiring
  - Plumbing-water-supply-sanitation
  - Finishes in floors and walls
  - Roof terrace
  - Service platform/verandah
  - Lifts
  - Doors windows and other elements

12.1.1 Need for strengthening or Retrofitting.

Concrete structures need to be strengthened for any of the following reasons:

1. Load increases due to higher live loads, increased wheel loads, installations of heavy machinery, or vibrations.
2. Damage to structural parts due to aging of construction materials or fire damage, corrosion of the steel reinforcement, and/or impact of vehicles.
3. Improvements in suitability for use due to limitation of deflections, reduction of stress in steel reinforcement and/or reduction of crack widths.
4. Modification of structural system due to the elimination of walls/columns and/or openings cut through slabs.

12.2 STRUCTURAL CONCRETE STRENGTHENING or Retrofitting

Many buildings that originally were constructed for a specific use now are being renovated or upgraded for a different application that may require higher load-carrying capacity. As a result of these higher load demands, existing structures need to be reassessed and may require strengthening to meet heavier load requirements. In general, structural strengthening may become necessary because of code changes, seismic upgrade, deficiencies that develop because of environmental effects (such as corrosion), changes in use that increase service loads, or deficiencies within the structure caused by errors in design or construction. The structural upgrade of concrete structures can be achieved using one of many different upgrading methods such as span shortening, external composites, externally bonded steel, external or internal post-tensioning systems, section enlargement, or a combination of these techniques. Similar to concrete repair, strengthening systems must perform in a composite manner with an existing structure to be effective and to share the applied loads. The strengthening is generally done by adopting (i) Jacketing Technique (ii) Externally Bonding Technique.

Strengthening or stiffening of members such as columns and girders is usually achieved either by replacing poor quality or defective material by better quality material or by adding additional material to the member. In either case, the new material will usually be reinforcing steel, high quality concrete, thin steel plates and straps or various combinations of these materials. The main difficulty in this type of operation is to achieve continuity of structural action between the original material and the new material. The various techniques of bolting, gluing, dowelling and keying have been developed to provide positive force transfer and composite action.

Column strengthening:

One of the simplest and most effective methods for strengthening a column in an existing building is to partially loaded the column by jacketing between floors and then insert two or more props to carry portion of the axial load. The props are usually rolled steel sections which may subsequently be encased in concrete to improve fire protection and appearance. The disadvantages of this method are that considerable floor space is lost, and that the props may not be effective in transferring moment unless positive connection details are introduced at both ends, for example in the form of end plates bolted through holes drilled in the floors.

Strengthening and stiffening of beams and girders:
Commonly used methods for strengthening flexural members include:

(a) Provision of additional concrete on the compressive face
(b) Addition of tensile reinforcement with a cast-in place or gunited cover
(c) Bolting or providing steel plates or straps to the surface of the member

The main problem of strengthening, stiffening and repair operation is to ensure good composite action between old and new material. Bolting of new concrete or steel to the existing member is effective but time consuming and expensive. For this reason, gluing procedures have been developed which provide strong and durable connections between concrete and surface straps and plates.

Strengthening and stiffening of slabs:
The simplest and cheapest procedure of strengthening and stiffening slabs is by providing additional props either in span or in the vicinity of existing columns and walls. However, this solution will often be unacceptable for architectural reason, even through props placed quite close to existing columns may be sufficient to bring the structure to an acceptable standard of serviceability.

12.2.1 Jacketing technique:
Jacketing is the process of fastening a durable material over concrete and filling the gap with a grout that provides needed performance characteristics. The materials used for jacket are metals, rubber, plastics, ferrocement and concrete. A typical jacketing process is shown in

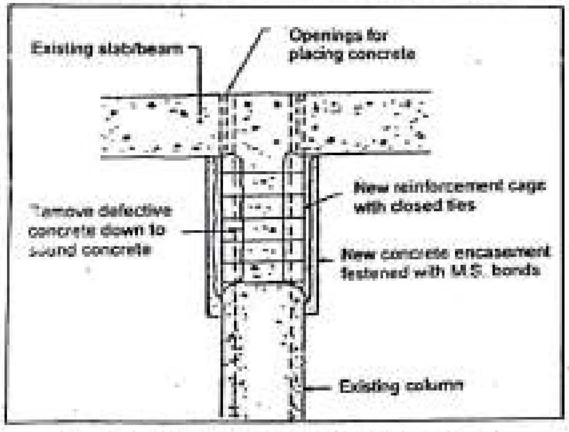

Figure 12.2. Process of jacketing technique

figure 12.2. This restores structural values, protects the reinforcements from exposure to the harmful elements and improves the appearance of the original concrete. Jacketing materials may also be secured to concrete by means of bolts, screws, nails or adhesives by bond with the existing concrete, or by gravity. The method of securing employed, will depend up on the pressure, the material used and the positioning of the jacketing material. Fibre glass reinforced plastics, ferrocement and other materials such as poly propylene can be used for jacketing.

The column jacket can also be used for increasing the punching shear strength of column-slab connections by using it as a column capital. When the jacket is provided around the periphery of the column, it is termed a collar. In most of the applications, the main function of the collar is to transfer vertical load to the column. Circular reinforcement can be used for load transfer. The practice of transferring load through dowel bars embedded into columns or shear keys has a disadvantage in that they require drilling of holes for dowels or cutting shear keys which are costly and time consuming, and can damage the existing column. Reinforcement encircling the column can be used to transfer the load through shear friction. The expansion of collar as it slides along the roughened surface causes the tensioning of circular reinforcement resulting in radial compression, which provides normal force needed for load transfer. The shear transfer strength is provided by both frictional resistance to sliding and dowel action of reinforcement crossing the crack.

The collar can also be used as mid-column bearing surface, acting as circumferential beam to distribute the concentrated load around the column. The collar is subjected to shear and bending along the collar circumference as well as direct bearing stress under concentrated load. Thus in addition to shear transfer reinforcement, the collar should be provided with reinforcement for shear and moment within the collar. The repair can be used as an alternate load path from the column to the collar and then to the connecting structural component. Column collars can be provided below the slab to act as column capital to improve punching shear strength of the slab-column connection.

12.2.2 Externally bonding technique
In this, suitable laminates are pasted externally in the flexural zone to improve upon the strength and deformation characteristics of distressed structural members. The laminates may be fabricated from any one of the following material: Steel, Stainless steel, FRP, Ferrocement and SIFCON and SIMCON. The laminates are bonded to the distressed structural elements using epoxy resin. **Design criteria suggested by researchers worked in plate bending techniques:**

(a) The plate/laminates width to thickness ratio should not be greater than 50.
(b) The central axis depth should not be greater than 0.4 times the effective depth of member.

Strengthening with external bonding of plates has the following limitations:

(a) Difficulty in manipulating the plates in field conditions
(b) Deterioration of the bend steel concrete interface

(c) Lack of proper formation of joints

A typical process of plate bonding technique is shown in figure 12.3 using steel plates.

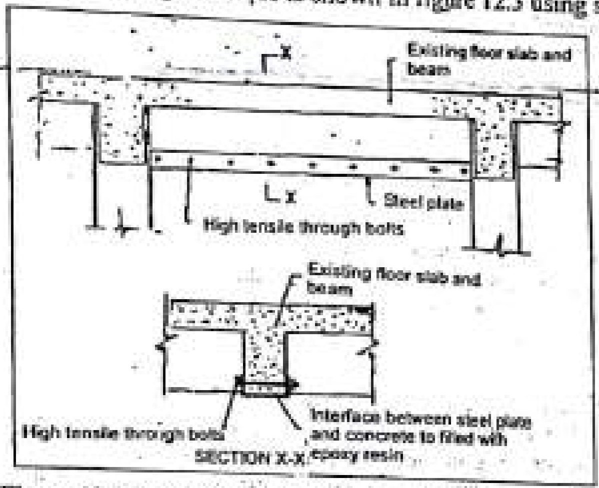

Figure 12.3. Process of plate bonding technique by steel plates

12.2.3 Externally bonded mild steel plates

The technique of gluing mild steel plates to the soffit of reinforced concrete beams and slabs can be applied to improve the structural performance of the existing structures. It increases the strength and rigidity and also reduces the flexural crack width in the concrete. The technique can be used to enhance the load carrying capacity either due to increase in load requirements or design or construction deficiencies. This technique has many advantages when applied to bridges. The plates can be easily handled and placed, this allowing the procedure to be carried out relatively quickly with minimal disruption to traffic. However, this technique is not recommended for application to bridges with unformed concrete or those affected by corrosion unless the source of the problem is rectified. The solution of an appropriate adhesive and surface preparation of the bonding surface is the most critical parameters for the successful implementation of this technique. The practical considerations to facilitate the placement of plates include the temperature requirements for mixing, applying and curing adhesive should be consistent with the existing conditions in the field and the adhesive should develop its strength rapidly.

Bonded steel elements - Strengthening concrete members by using bonded steel plates was developed in the 1960s. With this method, steel elements are glued to the concrete surface by a two-component epoxy adhesive to create a composite system and improve shear or flexural strength. The steel elements can be steel plates, channels, angles, or built-up members. In addition to epoxy adhesive, mechanical anchors typically are used to ensure that the steel element will share external loads in case of adhesive failure. The exposed steel elements must be protected with a suitable system immediately following installation. And regardless

of the specified corrosion protection system, its long-term durability properties and maintenance requirements must be considered fully. A roof was strengthened with FRP and steel plates externally as shown in figure 12.4.

Figure 12.4. Externally bonded steel plates with FRP

The roof consisted of prestressed concrete, hollow planks. Installation of the skylights required cutting openings in the planks that would reduce their load-carrying capacity. This issue was resolved by designing a hybrid strengthening system composed of FRP fabric and steel elements. The externally bonded FRP strengthened the planks adjacent to the one to be cut, while the steel elements tied the plank to the adjacent ones, thus creating a new unit consisting of three planks with adequate capacity. In addition to the fast application of this system, this was a cost effective solution that also was aesthetically pleasing.

12.3 STRENGTHENING WITH EXTERNAL REINFORCEMENT

Fibre composites are becoming increasingly important in the construction Industry with a number of applications in many areas. The fibres such as glass, carbon and aramid can be introduced in a certain position; volume and direction in the matrix to obtain the maximum efficiency. Other advantages offered by the fibre reinforced and greater efficiency, in construction when compared with the more conventional materials. **Some important applications of FRP are as follows:**

(a) Use of FRP castings to confine new concrete beams and columns or to strengthen beams in seismic area.

(b) Development of prestressing tenders made of parallel alignment of continuous fibres.

(c) Strengthening of concrete beams with uni-directional fibre reinforced concrete sheets bonded on their tension faces by the use of epoxy adhesives.

Composites:

Fiber reinforced polymer (FRP) systems are high-strength, lightweight reinforcement in the form of paper thin fabric sheets, thin laminates, or bars that are bonded to concrete members with epoxy adhesive to increase their load carrying capacity. These systems have been used extensively in the aerospace, automotive, and sport-equipment industries, and now are becoming a mainstream technology for the structural upgrade of concrete structures. Important characteristics of FRPs for structural repair and strengthening applications include their non-corrosive properties, speed and ease of installation, lower cost, and aesthetic appeal.

As with any other externally bonded system, the bond between the FRP system and the

2.4. Strengthening Procedure

In the NSM-steel technique, strengthening bars were placed into grooves cut in the concrete cover of the RC beams and bonded using epoxy adhesive groove filler. The installation of the strengthening steel bars began with cutting groove with the dimensions 1.5 $d_b \times 1.5\, d_b$ (where d_b is the diameter of the tension reinforcement) into the concrete cover of the beam specimens at the tension face in the longitudinal direction. The groove was made using a special concrete saw with a diamond blade. A hammer and a hand chisel were used to remove any remaining concrete lugs and to roughen the lower surface of the groove. The groove was cleaned with a wire brush and a high-pressure air jet. The details of the groove are shown in Figure 1. The groove was half-filled with epoxy and then the steel bar was placed inside the groove and lightly pressed. This forced the epoxy to flow around the inserted steel bar. Epoxy was used to fill the groove and the surface leveled. The bonded length of the NSM steel bars was 1900 mm. In ensuring the epoxy achieved full strength, the beam was allowed to cure for one week.

2.5. Test Setup

The instrumentation of the beams is presented in Figure 2. In measuring the deflection at beam midspan, one TML linear variable differential transducer (LVDT) was used. A number of gauges were used to measure strains. Two 5 mm strain gauges were attached to the middle of the internal tension bars. A 30 mm strain gauge was placed on the top surface of the beam at midspan. Demec gauges were attached along the depth of the beam at midspan. All beams were tested in four-point bending using an Instron Universal Testing Machine at heavy structural lab. The experiments were carried out using two controlling techniques. The first was load control, used for strain hardening. Commencing from the strain softening region, displacement control loading was maintained until failure. All data were recorded at 10 s intervals. The rate of the actuator was set to 5 kN/min during load control and 1.5 mm/min during displacement control. A Dino-Lite digital microscope was used to measure crack widths on the beams during testing.

3. Results and Discussion

The flexural capacities of tested beams were evaluated by a static four point bending test, causing the larger portion of the beams to take maximum stress. The key factors considered in this studies are cracking load, ultimate load, crack width and spacing, concrete compressive strain, tensile strain in the main steel reinforcement, sectional strain, and mode of failures. The experimental performance of all tested beams in terms of flexural load capacities are shown in Table 4.

12.5 EXTERNAL POST-TENSIONING

The external post-tensioning technique has been used effectively to increase the flexural and shear capacity of both reinforced and prestressed concrete members since the 1950s. With this type of upgrading, active external forces are applied to the structural member using post-tensioned (stressed) cables to resist new loads. Because of the minimal, additional weight of the repair system, this technique is effective and economical, and has been employed with great success to correct excessive deflections and cracking in beams and slabs, parking structures, and cantilevered members.

The post-tensioning forces are delivered by means of standard prestressing tendons or high-strength steel rods, usually located outside the original section. The tendons are connected to the structure at anchor points, typically located at the ends of the member. End-anchors can be made of steel fixtures bolted to the structural member, or reinforced concrete blocks that are cast into place. The desired uplift force is provided by deviation blocks, fastened at the high or low points of the structural element. Prior to external prestressing, all existing cracks are epoxy-injected and spalls are patched to ensure that prestressing forces are distributed uniformly across the section of the member.

Figure 12.7 illustrates an external post-tensioning system used to strengthen ~~prestressed~~ double tees damaged by vehicular impact. Four double tee stems on an overpass located were damaged when the driver of an over-height truck failed to observe the posted height restriction. The four stems suffered excessive concrete cracking and spalling, and damage occurred to some of the internal prestressing steel.

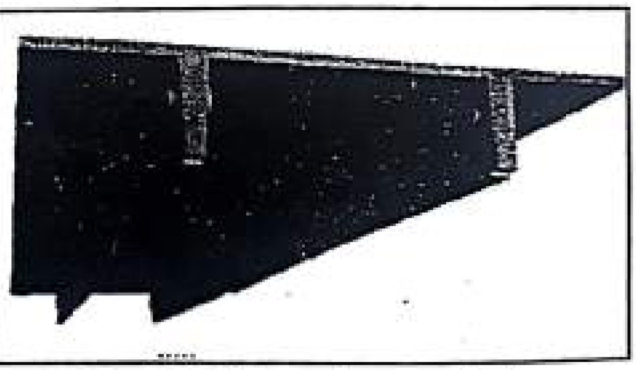

Figure 12.7. External post-tensioning technique

Proposed solutions included replacing the damaged double tees with new ones and installing a steel frame underneath for support. Both options would render the overpass out of service for a longer-than-desired period. The option of an external post-tensioning system was more economical, required less time to complete, and allowed for a strengthening system that provided active forces. Therefore, it was more compatible with the existing construction. After all cracks were injected, the sides of the stems were formed and new concrete was cast to restore the integrity of the stems. The strengthening system then was installed, and — after the concrete cured — the external strands were stressed according to the engineer-specified forces.

12.6 SECTION ENLARGEMENT

This method of strengthening involves placing additional bonded reinforced concrete to an existing structural member in the form of an overlay or a jacket. With section enlargement, columns, beams, slabs, and walls can be enlarged to increase their load-carrying capacity or stiffness. A typical enlargement is approximately 2 to 3 inches for slabs and 3 to 5 inches for beams and columns.

Figure 12.8 depicts details of a section enlargement used to increase the capacity of a main girder in a university parking garage. The girder was re-evaluated because of a change in the required loading and found to be deficient in flexure and shear. To correct the deficiency, additional flexural and shear steel were added. The entire beam then was formed and a 4-inch jacket of concrete was cast to enlarge the section.

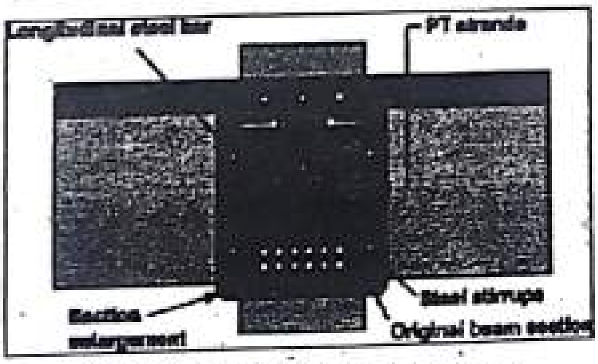

Figure 12.8. Section enlargement technioue

(b) Construction joints are clean and free from defective concrete.
(c) Forms are grout-tight, amply strong, and set to their true alignment and grade.
(d) All reinforcement steel and embedded parts are clean, in their correct position, and securely held in place
(e) Adequate concrete delivery equipment and facilities are on the job, ready to go, and
(f) Capable of completing the placement without addition unplanned construction.

12.9 GUIDELINES FOR SEISMIC REHABILITATION OF EXISTING BUILDINGS

Advances in earthquake-related technology during the past few decades have led to a realization that seismic risk to life and property can be reduced significantly by improving seismic performance of existing seismically deficient buildings. Detailed post-earthquake investigations of building failures have provided engineers with considerable information concerning the details of building design and construction that enhance earthquake resistance.

12.9.1 Seismic vulnerability

The vulnerability of a building subjected to an earthquake is dependent on seismic deficiency of that building relative to a required performance objective. The seismic deficiency is defined as a condition that will prevent a building from meeting the required performance objective. Thus, a building evaluated to provide full occupancy immediately after an event may have significantly more deficiencies than the same building evaluated to prevent collapse. Depending on the vulnerability assessment, a building can be condemned and demolished, rehabilitated to increase its capacity, or modified so that the seismic demand on the building can be reduced. Thus, structural rehabilitation of a building can be accomplished in a variety of ways, each with specific merits and limitations related to improving seismic deficiencies.

12.9.2 Common seismic deficiencies

Regardless of the evaluation method used, failure to meet the stipulated performance objective implies certain seismic deficiencies. These deficiencies are described below:

Global strength:
Global strength typically refers to the lateral strength of the vertically oriented lateral force-resisting system. For degrading structural systems characterized by a negative post yield slope on the pushover curve, a minimum strength requirement may apply. In certain cases, the strength will also affect the total expected inelastic displacement and added strength may reduce nonlinear demands into acceptable ranges. A deficiency in global strength is common in older buildings either due to a complete lack of seismic design or a design to an early building code with inadequate strength requirements. If prescriptive equivalent lateral force methods or linear static procedures have been used for evaluation, inadequate strength will directly relate to unacceptable demand-to-capacity ratios within elements of the lateral force-resisting system.

Global stiffness:
Global stiffness refers to the stiffness of the entire lateral force-resisting system although the lack of stiffness may not be critical at all levels. For example, in buildings with narrow walls, critical drift levels occur in the upper floors. Conversely, critical drifts most often occur in the lowest levels in frame buildings. Stiffness must be added in such a way that drifts are efficiently reduced in the critical levels. Although strength and stiffness are often controlled by the same existing elements or the same retrofit techniques, the two deficiencies are typically considered separately. Failure to meet evaluation standards is often the result of a building placing excess drift demands on existing poorly detailed components.

Configuration:
This deficiency category covers configuration irregularities that adversely affect performance. In codes for new buildings, these configuration features are often divided into plan irregularities and vertical irregularities. Plan irregularities are features that may place extraordinary demands on elements due to torsional response or the shape of the diaphragm. Vertical irregularities are created by uneven vertical distribution of mass or stiffness between floors that may result in concentration of force or displacement at certain levels. In older existing buildings, such irregularities are seldom taken into consideration in the original design and, therefore, normally require rehabilitation measures to mitigate.

Load path:
A discontinuity in the load path, or inadequate strength in the load path, may be considered overarching because this deficiency will prevent the positive attributes of the seismic system from being effective. The load path is typically considered to extend from each mass in the building to the supporting soil. For example, for a panel of cladding, this path would include its connection to the supporting floor or floors, the diaphragm and collectors that deliver the load to components of the primary lateral force resisting system (walls, braces, frames, etc.), continuity of these components to the foundation, and finally the transfer of loads between foundation and soil. Many load path deficiencies are difficult to categorize because the strength deficiency may be considered to be part of another element. For example, an inadequate construction joint in a shear wall could be considered a load-path deficiency or a shear wall deficiency in the category of global strength.

Inadequate component detailing:
Detailing, in this context, refers to design decisions that affect a component's or system's behavior beyond the strength determined by nominal demand, often in the nonlinear range. An example of a detailing deficiency is poor confinement in concrete gravity columns. Often in older concrete buildings, the expected drifts from the design event will exceed the deformation capacity of such columns, potentially leading to degradation and collapse. Although the primary gravity load design is adequate, the post elastic behavior is not, most often due to inadequate configuration and spacing of ties. Identification of detailing deficiencies is significant in selection of mitigation strategies because acceptable performance often may be achieved by local adjustment of detailing rather than by adding new lateral force-resisting elements.

In the case of gravity concrete columns, acceptable performance can be achieved by enhancing deformation capacity by adding confinement rather than by reducing global deformation demand by adding lateral force-resisting elements.

Diaphragm deficiencies:
The primary purpose of diaphragms in the overall seismic system is to act as a horizontal beam spanning between lateral force-resisting elements. Diaphragm deficiencies include such factors as inadequate shear or bending strength, stiffness, or reinforcing around openings or reentrant corners. Inadequate local shear transfer to lateral force-resisting elements or missing or inadequate collectors are categorized as load path deficiencies.

Foundation deficiencies:
Foundation deficiencies can occur within the foundation element itself, or due to inadequate transfer mechanisms between foundation and soil. Element deficiencies include inadequate bending or shear strength of spread foundations and grade beams, inadequate axial capacity or detailing of piles and piers, and weak and degrading connections between piles, piers and caps. Transfer deficiencies include excessive settlement or bearing failure, excessive rotation, inadequate tension capacity of deep foundations, or loss of bearing capacity due to liquefaction.

Other deficiencies:
Deficiencies that do not fit into one of the categories described above may include:

(a) Geologic hazards
(b) Adjacent buildings
(c) Deteriorated structural materials

12.9.3 Strategies for rehabilitation schemes

Technical Considerations:
Selected techniques must eliminate deficiencies, preferably more than one deficiency. First one should consider enhancing existing elements such as shear walls, moment frames, and bracing frames. The deformation compatibility between new elements and existing elements must be considered. In some cases, the application of base isolators or damping devices is the most efficient way to eliminate deficiencies.

Non-technical considerations:
The solution chosen for rehabilitation is almost always dictated by building-user oriented issues rather than by merely satisfying technical demands. There are five basic issues that are of concern to building owners or users:

(a) Construction cost,
(b) Seismic performance,
(c) Short term disruption of occupants,
(d) Effects on long-term functionality of building, and
(e) Aesthetics, including consideration of historic preservation.

All of these characteristics are always considered, but an importance will eventually be put on each of them, either consciously or subconsciously, and a combination of weighting factors will determine the scheme chosen.

Cost:
Construction cost is always important and is balanced against one or more other considerations deemed significant. However, sometimes other economic considerations, such as the cost of disruption to building users, or the value of contents to be seismically protected, can be orders-of-magnitude larger than construction costs. Thus, cost may be the only criterion applied when choosing among equivalent rehabilitation options.

Seismic performance:
Prior to the emphasis on performance based design, perceived qualitative differences between the probable performance of difference schemes would be used to assist in choosing a scheme. Specific performance objectives are often set prior to the development of schemes. Objectives that require a limited amount of damage or continued occupancy will severely limit the retrofit methods that can be used and may control the other issues.

Short-term disruption of occupants:
When seismic rehabilitation is done at the time of major building remodeling, disruption issue is minimized. However, in cases where the building is partially or completely occupied, this parameter commonly becomes dominant and controls the design.

Effects on long-term functionality of building:
This characteristic is often judged less important than others. The planning flexibility is only subtlety changed. However, it can be significant in building occupancies that need open spaces such as retail spaces and parking garages.

Aesthetics:
In historic buildings, considerations of preservation of historic fabric usually control the design. Performance objectives are controlled by limitations imposed by preservation. In non-historic buildings, aesthetics is commonly stated as a criterion, but is often sacrificed, particularly in favor of minimizing cost and disruption to tenants.

12.9.4 Rehabilitation techniques

Different building types require different mitigation techniques for a specific seismic deficiency. Depend on building types and associated seismic deficiencies, alternative recommendations are made to satisfy the performance objective of rehabilitation. Rehabilitation techniques are being developed for common building types. They are:

Wood light-frames: One- and two-detached dwellings of one or more stories in height.

Multi-story, multi-unit residential wood frames: Large residential buildings with commercial space at the ground floor.

Glass fibre

corrosion resistance, and it is uncommon for civil engineering application. Of the three fibers, the E-glass is the most common reinforcement material used in civil structures. Some typical properties of the glasses are shown in table 9.1. It is produced from lime-alumina-borosilicate which can be easily obtained from abundance of raw materials like sand. The fibers are drawn into very fine filaments with diameters ranging from 2 to 13 $\times 10^{-6}$ m. The glass fiber strength and modulus can degrade with increasing temperature. Although the glass material creeps under a sustained load, it can be designed to perform satisfactorily. The fiber itself is regarded as an isotropic material and has a lower thermal expansion coefficient than that of steel.

Table 9.1. Typical properties of E & S glass

Typical Properties	*E-Glass*	*S-Glass*
Density (g/cm^3)	2.60	2.50
Young's Modulus (GPa)	72	87
Tensile Strength (Gpa)	1.72	2.53
Tensile Elongation (%)	2.4	2.9

Glass fibres possess high tensile strength and modulus of elasticity, but serious concern is expressed regarding their durability in an alkaline environment. For low w/c pastes, compressive strength is reduced by about 20% and for higher w/c ratio the decrease can be as high as 30%. Uniaxial tensile strength increases with age and amount of fibre. Aggregate grading does not influence the strength. In spite of the enhanced mechanical properties, question of the durability of alkaline resistant glass fibre concrete composite in alkaline environment remain unresolved.

Glass fibre reinforced concrete: "GFRP"

For the precast concrete industrial sector, the use of light-weight composite components (*e.g.* 60-80 kg/m² for main slab elements and 30-50 kg/m² for secondary slab elements), able to withstand to structural loads, would represent a significative improvement with respect to the current production. In fact the possibility to manufacture precast elements weighting about one third of the conventional concrete products (*e.g.* 250-300 kg/m² for main and 100-150 kg/m² for secondary slab elements), would allow to achieve some important objectives:

(a) A substantial reduction of overall costs;
(b) A major improvement in the safety of the structure with respect to the seismic risk;
(c) The possibility to construct buildings also on low bearing capacity soils.

The reduction in weight and of carriage costs also allows to think about exportation of structural elements produced in an European factory to a foreign Country. An opportunity has been recognized by the industrial proposes to realize these light weight structural elements using Glass-fibre Reinforced Concrete (GFRC). Until now its main application has been in architectural cladding, in the form of panels 10-20 mm thick. Due to the tendency of GRC to *age*, or loose ductility with time, it never has been considered for *structural* applications. Although the estimated cost of such innovative, light-weight GRC components (*i.e.* 100-150 ECU/m² for main slab elements) is comparable or lightly higher than conventional ones, a reduction of overall costs of about 30% could be foreseen due to the following reasons:

(a) A slender carrying structure of the building (foundations, pillars, beams, etc.) is required;
(b) A complete automation of the whole production cycle is allowed;
(c) Carriage and assembly costs are minimal;
(d) The surface finishing (*e.g.* thermal insulation, waterproofing, painting, etc.) of precast elements can be carried out directly at the factory and no further operations are required on site.

Applications:
Potential applications of structural GRC components are copious also limiting our consideration to the precast sector; some of these have already been identified by the proposers and represent an interesting market opportunity:

(a) Main and secondary elements for slabs (*e.g.* in industrial sheds or in building retrofitting);
(b) Facade panels, wall panels and roof elements (*e.g.* in stand-alone technical shelters for mobile phone repeaters and in prefabricated houses).

The time-to-market for the above-mentioned or similar applications is estimated to require at least one year after the end of the project, because it will be necessary to modify the current manufacturing plants for GRC production or to install new ones.
The lack of specific standards for designing and testing of these components could represent an obstacle to a wide acceptance of new products. To avoid this risk, one of the actions carried out by the consortium members in the exploitation phase will consist of a collaboration with national and international Standardisation Bodies in order to ensure the issuing of a specific normative in the shortest possible time.

9.12.2 Polypropylene
Polypropylene fibres have high tensile strength and low modulus of elasticity and they are dependent on the isotacticity molecular weight and other structural features of PP. The elastic properties would be influenced by the extent to which calcium hydroxide interacts with fibre at the interface and therefore dependent on the fibre type. The brittleness of the composite is probably also affected by the amount and size of the calcium hydroxide crystals present. Further crystallisation of the calcium hydroxide in the contact zone may actually result in a weakening of the bond between the fibre and matrix. Surface modification of PP can result in improving interfacial bond. Reinforcement of cement matrices with continuous fibres, fibrillated filaments, fibrillated films; tape or woven fabric generally results in increased flexural and tensile strength.

9.12.3 Natural fibres
Some investigation have already been carried out on the use of natural fibres from coconut husk, sisal, sugar cane bagasse, bamboo, akara, plantain and musamba in cement paste, mortar, and concrete. These investigations have shown encouraging results. Flexural strength increases with fibre addition to a maximum and then decreases. The decrease at higher-fibre content is due to incomplete compaction and increased porosity. The decrease in the strength for longer fibres was mainly due to bundling effects of the fibres. Impact strength depends on curing period and fibre volume fraction for both jute and their fibre-reinforced concretes. Durability is of major importance in evaluating the suitability of natural fibres for inclusion in cement matrix. Coir fibre exhibits ductile failure characteristics while most of the other fibres exhibit brittle failure. Coir also shows the greater resistance to alkali attacks. Resin coatings provide a reasonable measure of protection against alkali attack.

9.12.4 Aramid fibers "AFRP"
These are synthetic organic fibers consisting of aromatic polyamides. The aramid fibers have excellent fatigue and creep resistance. Although there are several commercial grades of aramid fibers available, the two most common ones used in structural applications are Kevlar® 29 and Kevlar® 49. Typical properties of aramid fibres are shown in table 9.2.The Young's Modulus curve for Kevlar® 29 is linear to a value of 83 GPa but then becomes slightly concave upward to a value of 100 GPa at rupture; whereas, for Kevlar® 49 the curve is linear to a value of 124 GPa at rupture. As an anisotropic material, it's transverse and shear modulus are an order of magnitude less than those in the longitudinal direction. The fibers can have difficulty achieving a chemical or mechanical bond with the resin.

Table 9.2. Typical properties of aramid fibres

Typical Properties	*Kevlar 29*	*Kevlar 49*
Density (g/cm³)	1.44	1.44
Young's Modulus (GPa)	83/100	124
Tensile Strength (GPa)	2.27	2.27
Tensile Elongation (%)	2.8	1.8

Experimentation was conducted on the use of a new aramid fiber for the reinforcement of Portland cement-based concrete and slurry. The study showed that synthetic fibers manufactured by cutting an epoxy-impregnated, braided bundle made of aramid filaments act similarly to steel fibers in reinforcing both portland cement-based concrete and slurry matrices. The advantage of epoxy-coated aramid over steel is in the lack of corrosion problems, and over polypropylene in the higher performance.

9.12.5 Carbon fibres "CFRP"
The graphite or carbon fiber is made from three types of polymer precursors—polyacrylonitrile (PAN) fiber, rayon fiber, and pitch. The tensile stress-strain curve is linear to the point of rupture. Although there are many carbon fibers available on the open market, they can be arbitrarily divided into three grades as shown in Table 9.3. They have lower thermal expansion coefficients than both the glass and aramid fibers. The carbon fiber is an

iisotropic material, and its transverse modulus is an order of magnitude less than its ngitudinal modulus. The material has a very high fatigue and creep resistance.

Table 9.3. Typical properties of carbon fibres

Typical Properties	*High Strength*	*High Modulus*	*Ultra-High Modulus*
Density (g/cm^3)	1.8	1.9	2.0-2.1
Young's Modulus (GPa)	230	370	520-620
Tensile Strength (GPa)	2.48	1.79	1.03-1.31
Tensile Elongation (%)	1.1	0.5	0.2

ince its tensile strength decreases with increasing modulus, its strain at rupture will also be uch lower. Because of the material brittleness at higher modulus, it becomes critical in joint id connection details, which can have high stress concentrations. As a result of this ienomenon, carbon composite laminates are more effective with adhesive bonding that iminates mechanical fasteners. Wide spread use of carbon fibre in cement has been limited ie to cost consideration. Initially it was used in the pipe manufacturing only. Alternative ses are now being exploited as a result of the development of less expensive discontinuous bre. Although discontinuous randomly distributed carbon fibres are less efficient than ontinuous aligned fibres, the properties of composites containing these carbon fibres are gnificantly improved. Tensile and flexural strengths increase with fibre content and they e generally less than for those with continuous fibres. At low water-cement ratios, the rengths are similar. Compressive strength of carbon fibre reinforced cements generally ecreases with fibre addition.

9.6 ORGANIC POLYMERS

Organic polymers are complex chemical compounds derived mainly from the petrochemical industry. These materials are often referred to as resins and the principal resins used in the construction industry are epoxide, polyurethane, polyester, acrylic, polyvinyl acetate and styrene- butadiene.

9.6.1 Epoxide resins

The formulators market epoxide resins and have the special properties required for the specific use to which they will be put. **The basic characteristics of epoxide resins include the following:**

(a) Outstanding adhesive qualities to such materials as concrete and steel
(b) Resistance to wide range of acids, alkalies and other chemicals
(c) Rather vulnerable to organic solvents
(d) Low shrinkage
(e) High coefficient of thermal movement compared with concrete.
(f) High compressive, tensile and flexural strength
(g) Appreciable loss of strength at high temperatures
(h) High rate of gain of strength, which can be varied to suit the particular application
(i) Rather poor resistance to fire compared with concrete and clay bricks
(j) To obtain satisfactory results, the site conditions under which the resin will be applied must be stated to the formulator.

9.6.2 Polyurethanes

Polyurethanes, like epoxide resins, are products of the petrochemical industry, They can be obtained as elastomers, solid and rigid materials and as flexible coatings. They are very durable in external conditions and retain their gloss well. Polyurethanes can be specially formulated to meet specific site requirements. They can be combined with epoxies and will withstand relatively high temperatures as well as sudden changes in temperature, *i.e.* thermal shock.

9.6.3 Polyester resins

Polyester resin can be divided into two classes:

(a) The so-called saturated polyester resins such as polyethylene terepthalate or terylene
(b) The unsaturated polyester resins which can be cross-linked

3.1 Special Concretes:

Note: For more Special Concretes please refer "Concrete Technology" by M.S. Shetty. From Page 444 with the topic "Special concrete and concreting methods.

3.2 Special Mortar:

The following are the different types of special mortars available, they are

- Cement-clay mortar
- Light-weight and heavy mortars
- Decorative mortar
- Air-entrained mortar
- Gypsum mortar
- Fire-resistance mortar
- Packing mortar
- Sound absorbing mortar
- X-ray shielding mortar

3.2.1 Cement-clay mortar

- Here clay is introduced as an effective finely ground additive in quantities ensuring a cement-clay proportion of not over 1:1. The addition of clay improves the grain composition, the water retaining ability and the workability of mortar and also increases the density of mortar.
- This type of mortar has better covering power and can be used in thin layers.

3.2.2 Lightweight and Heavy mortars

Light weight mortars:

- These are prepared form light porous sands fro pumice and other fine aggregates. They are also prepared by mixing wood powder, wood shavings or saw dust with cement mortar or lime mortar.
- In such mortars, fibres of jute coir and hair, cut into pieces of suitable size, or asbestos fibres can also be used.
- These mortars have bulk density less than $15KN/m^3$.

Heavy weight mortars:

Heavy weight mortars:

- These are prepared from heavy quartz or other sands.
- They have bulk density of 15 KN/m^3 or more.
- They are used in load bearing capacity.

3.2.3 Decorative mortars:

These mortars are obtained by using-

- Colour cements or pigments and
- Fine aggregate of appropriate color, texture and surface.

3.2.4 Air-entrained Mortar

- The working qualities of lean cement-sand mortar can be improved by entraining air in it(air serves as a plasticizer producing minute air bubbles which helps in flow characteristics and workability)/
- The air bubbles increase the volume of the binder paste and help to fill the voids in the sand.
- The air entraining also makes the mortar weight and a better heat and sound insulator.

3.2.5 Gypsum Mortar

- These mortars are prepared from gypsum binding materials such as building gypsum and anhydrite binding materials.

3.2.6 Fire Resistant Mortar

- It is prepared by adding aluminous cement to a finely crushed power of fire-bricks(Usually proportion being one part of aluminous cement to two parts of powder of fire-bricks).

- This mortar being fire resistance, is used with fire-bricks for lining furnaces, fire places, ovens etc.

3.2.7 Sound Absorbing mortar

- These mortarts may have binging materials such as cement, lime, gypsum slag etc and aggregate(light weight porous materials(such as pumice, cinders etc.
- The bulk density of such a mortar varies from 6 to 12KN/m^3.
- Noise level can be reduced by using sound absorbing plaster formed with the help of sound absorbing mortar.

3.3 Concrete Chemicals

Admixtures are used to modify the properties of fresh and hardened concrete. They are classified as chemical and mineral admixtures. Chemical admixtures are used in construction industry for building strong, durable and waterproof structures. Depending on their use, chemical admixtures are used for the following four main purposes.

1. Some chemicals are mixed with concrete ingredients and spread throughout the body of concrete to favorably modify the moulding and setting properties of the concrete mix. Such chemicals are generally known as *chemical admixtures* Admixtures are added to concrete to give it certain desirable properties in either the fresh or the hardened state. Most admixtures result in modifying more than one intended property.
2. Some chemicals are applied on the surfaces of moulds used to form concrete to effect easy mould-releasing operation.
3. Some chemicals are applied on the surfaces of concrete to protect it during or after its setting.
4. Some chemicals are applied to bond or repair broken or chipped concrete.

3.4 Special Cements for accelerated Strength Gain

In repairs of certain strucutures, particularly roadways and bridges, it may be desired that early strength gain should be as rapid as possible. The engineer may, as a first approach, consider admixtures so that ordinary types of Portland cement can be used. The chief chemical admixture now used for this purpose is superplasticizer.

Formerly high doses of calcium chloride were advocated but this procedure has been rejected on the basis of corrosion, problems associated with calcium chloride use. The time of setting of Portland cement concrete and its strength gain may be shortened by the use of calcium aluminate cement. Because of problems associated with the conversion, under hot humid conditions, of the calcium aluminate hydrates from one form to another, and the resultant strength losses, other types of cements have been preferred.

Regulated set cement is a modified Portland cement which contains a substantial amount of calcium fluoro-aluminate. The cement meal contains a substantial amount of fluorite as a substitute for limestone. The burning process has a problem due to the release of small amounts of fluoro compounds. When prepared and ground the initial and final set of this type of cement occurs almost simultaneously and therefore the time between mixing and set is often referred to as the handling time. As a rule, this varies between 2 to 45 minutes.

The strength level is adjusted by controlling the amount of calcium fluoro aluminate in the cement. The time of set is reduced and the compressive strength gain increased in regulated cement mortars and concrete by an increase in the cement content of the mix, reduction of the water/cement ratio, increases temperature of the mix and increase in curing temperature.

The chemical reactions of this type of cement are much more energetic than those of Portland cements. For that reason retardation is necessary. Conventional retarders for Portland cement are not effective in controlling the set of regulated set cement. However, citric acid is used in the mix as a retarder. Where practical, the setting action can be effectively controlled by reducing the mix temperature. Such reductions in the temperature of the mix is also advantageous, as the heat of hydration is considerably higher than that of Portland cement concrete.

Special cements based on chemical reactions which are completely different from those of normal Portland or similar cements are now part of the technology. These include fast-setting magnesium phosphate and aluminium –phosphate cements, which when used for concrete patching for pavements allow traffic flow after only 45 minutes.

4.1 Rust Eliminators

Cement paste normally provides a highly alkaline environment that projects embedded steel against corrosion. Concrete with a low water/cement ratio, well compacted and well cured, has a low permeability and hence minimizes the penetration of atmospheric moisture as well as other components such as oxygen, chloride ion, carbon dioxide and water, which encourage corrosion of steel bar.

In very aggressive environments, the bars may be coated with special materials developed for this purpose. Coating on reinforcing steel, therefore, serves as a means of isolating the steel from the surrounding environment. Common metallic coatings contain galvanizing zinc. High chloride concentration around the embedded steel corrodes the zinc coating, followed by corrosion of steel.

Hence, this treatment used for moderately aggressive environments. For high corrosive atmospheres caused by chloride ions from the de-icing salts applied to protect against sodium chloride and calcium chloride, usually near seashores, epoxy coating is applied to protect steel reinforcing bars from corrosion. Such bars have acceptable bond and creep characteristics. The coat normally applied is 150 um thick. The reinforcement is epoxied in the factory itself, where the steel rods are manufactured. Such reinforcement are known as fusion-bonded epoxy coated steel. Steel manufacturers also manufacture CTD bars with better corrosion resistance, termed as Corrosion Resistance Steel (CRS). The performance of the CRS CTD bars is better in resisting corrosion compared to plain CTD bars. However, the use of CRS CTD bars will only delay the process of corrosion. It will not prevent corrosion once for all.

4.2 Polymer Resin based Coating

These are generally of two types,

1. Resins blended with organic solvents and
2. Solvent free coating

Solvent-based coatings are subdivided into single and two component coatings. The coatings on drying produce a smooth dense continuous film that provides a barrier to moisture and mild chemical attack of the concrete. Because of the resistance to moisture penetration, staining, and ease of cleaning, they are preferred for locations of high humidity and those in which a lot of soiling occurs.

Most products are low solids content materials which require multiple coats to produce a continuous film over concrete, since the materials are thermoplastic, and have a significant degree of extensibility they are capable of bridging minor cracks which may develop in the concrete surface if they are applied in sufficient thickness. The number of coats required depends on the surface texture, porosity and the targeted dry film thickness. Although some of the newer products have some moisture tolerance, enabling them to be applied over damp surfaces, in normal usage they should be applied over dry surfaces. Due to their relative in permeability to water vapor, they could blister When applied to concrete surfaces with high moisture content or where the opposite surface of the concrete is in constant contact with moisture. Careful control of wet film thickness is therefore necessary during application.

Two component polymer coatings consist of a solution of a compounded polymer with or without solvent and a reactive chemical component called the curing agent hardener or catalyst. The materials are usually mixed just prior to use in accordance with the manufacturer's instructions. When using two components polymer based coatings the following items are of importance to the application of the materials.

1. Most produces are supplied as a kit containing the two components in the required proportions. Therefore, in order to realize the full potential of the product the correct mix ratio of the two components must be used.

2. To ensure a complete reaction of the two components they must be mixed thoroughly.

3. Some two component material require an induction period of 15 to 40 min after mixing. Therefore, such products cannot be used immediately after mixing.

4. Viscosity reduction by the use of thinners should be resorted to only after the manufacturers are consulted.

5. The storage temperature of solvent based coatings is not critical. They should be stored are a temperature 16 to 32°c just prior to use.

4.3. Formed Concrete

If a sufficiently portion of concrete is removed, it can best be replaced with concrete placed in forms. This concrete can be placed without a bonding agent and without grout on the prepared surface of the old concrete. US bureau of reclamation suggests that this method should be used

(i) When the depth of the repair exceeds 150 mm,

(ii) For holes extending right though the concrete section

(iii) For holes in unreinforced concrete with area greater than $0.1m^2$ and over 100 mm deep, and (iv)For holes in reinforced concrete which have an area greater than $0.05m^2$ and which extend deeper than the reinforcement.

There are some essential requirements that apply to the use of formed concrete as a replacement material, regardless of its location in the structure.

(i) The concrete should be made from the best possible materials and with the lowest possible water/cement ratio.

(ii)To keep shrinkage to a minimum, the aggregate size should be large as can be accommodated and the water content as low as possible.

(iii)The mix should be designed so that no bleeding occurs in order to ensure that the replacement material remains in intimate contact with old concrete located above it.

(iv)The hole to be filled must be shaped so that there are no feather edges and with a depth normal to the finished seurface of atleast 40mm. It must also be shaped so that sir is not trapped.

(v)Forms must be robust and firmly fixed so that they withstand any applied pressure and do not allow grout leakage.

(vi)Old concrete, against which new concrete is to be placed, must be sound, completely clean and saturated and the surface must be free from moisture.

4.4. Mortar and Dry Pack

Dry packing is the hand placement if a low W/C ratio mortar which is subsequently rammed in to place to produce a dense mortar plug having tight contact to the existing concrete.

Because of the low W/C ratio, there is e patch remains little shrinkage and the patch remains tight, with good durability, strength and water tightness. Dry pack should be used for filling holes having a depth equal to, or greater than, the least surface dimension of the repair area; for cone belt, she bolt, core holes and grout-insert holes; for holes left by the removal of form ties; and for narrow slots cut for repair of cracks. Dry pack should not be used for relatively shallow depressions where lateralrestraint cannot be obtained, for filling behind reinforcement, or for filling holes that extend completely through a concrete section. For the dry pack method of concrete pair, holes should be sharp and square at the surface edges, but corners within the holes should be rounded, especially when water tightness is required. The interior surfaces of holes left by cone bolts and she bolts sould be roughened to develop and effective bond; this can be done with a rough stub of 7/8 inch steel wire rope, a notched tapered reamer, or a star drill. Other holes should be undercut slightly in several places around the perimeter. Holes for dry pack should have a minimum depth of 1 inch.

4.5. Vacuum Concrete:

It is well known that high water/cement ratio is harmful to the overall quality of concrete, whereas low water/cement ratio does not give enough workability for concrete to be compacted hundred percentage. Generally, higher workability and higher strength or very low workability and higher strength do not go hand in hand. Vacuum process of concreting enables to meet this conflicting demand. This process helps a high workable concrete to get high strength.

In this process, excess water used for higher workability, not required for hydration and harmful in many ways to the hardened concrete is withdrawn by means of vacuum pump, subsequent to the placing of the concrete. The process when properly applied produces concrete of quality. It also permits removal of formwork at an early age to be used in other repetitive work.

It essentially consists of a vacuum pump, water separator and filtering mat. The filtering consists of a backing piece with a rubber seal all round the periphery. A sheet of expanded metal and then a sheet of wire gauge also form part of the filtering mat. The top of the suction mat is connected to the vacuum pump. When the vacuum pump operates, suction is created within the boundary of the suction mat and the excess of water is sucked from the concrete through the fine wire gauge or muslin cloth. At least one face of the concrete must be open to the atmosphere to create difference of pressure. The contraction of concrete caused by loss of water must be vibrated.

The vacuum processing can be carried out either from the top surface or from the side surface. There will be only nominal difference in the efficiency of top processing or side processing. It has been seen that the size of the mat should not be less than 90cm X 60cm. smaller mat was not found to be effective

4.6. Rate of extraction of water:

The rate of extraction of water is dependent upon the workability of mix, maximum size of aggregate, proportion of fines and aggregate, cement ratio. In general, the following general tendencies are observed.

- The amount of water, which may be withdrawn, is governed by the initial workability or the amount of free water. A great reduction in the water/cement ratio can, therefore, be obtained with higher initial water/cement ratio.
- If the initial water/cement ratio is kept the same the amount of water which can be extracted is increased by increasing the maximum aggregate size or reducing the amount of fines in the mix.
- Although the depression of the water/cement ratio is less, the lower the initial water/cement ratio, the final water/cement ratio is also less, the lower the initial value.

- The ability of the concrete to stand up immediately after processing is improved if a fair amount of fine material is present, if the maximum aggregate size is restricted to 19mm and if a continuous grading is employed.
- Little advantage is gained y prolonging the period of treatment beyond 15 to 20 minutes and a period of 30 minutes is the maximum that should be used.

4.7. The Gunite or Concrete

Gunite can be defined as mortar conveyed through a hose and pneumatically projected at a high velocity onto a surface. Recently this method has been further developed by the introduction of small sized coarse aggregate into the mix deposited to obtain considerably greater thickness in one operation and to make the process economical by reducing the cement content. Normally fresh material with zero slump can support itself without sagging or peeling off. The force of the jet impacting on the surface compact the material. Sometimes use of set accelerators to assist overhead placing is practiced. The newly developed "Redi-set cement" can also be used for shotcreting process.

There is not much difference between guniting and shotcreting. Gunite was first used in the early 1900 and this process is mostly used for pneumatical application of mortar of less thickness, whereas shotcrete is a recent development on the similar principle of guniting for achieving greater thickness with small coarse aggregates.

There are two different processes in use, namely the "Wet-mix" process and the "dry-mix" process. The dry mix process is more successful and generally used.

4.7.1 Dry-mix Process

The dry mix process consists of number of stages and calls for some specialized plan. A typical small plant set-up is shown.

- This material is carried by compressed air through the delivery hose to a special nozzle. The nozzle is fitted inside with a perforated manifold through which water is sprayed under pressure and intimately mixed with the sand/cement jet.
- The wet mortar is jetted from the nozzle at high velocity onto the surface to gunited.

4.7.2 Wet-mix process

In the wet-mix process the concrete is mixed with water as for ordinary concrete before

4.7.2 Wet-mix process

In the wet-mix process the concrete is mixed with water as for ordinary concrete before conveying through the delivery pipe line to the nozzle, at which point it is jetted by a compressed air, onto the work in the same way, as that of dry-mix process.

The wet-mix process has been generally discarded in favour of dry-mix process, owing to the greater success of the latter.

The dry-mix method makes use of high velocity or low velocity system fully. The high velocity gunite is produced by using the small nozzle and a high air pressure to produce a high nozzle velocity of about 90 to 120 meters/sec. This results in exceptional good compaction. The lower velocity gunite is produced using large diameter hose for large output. The compaction will not be very high.

General use of Shotcrete

1. It is useful where considerable savings and peculiar adaptability is needed and it is more suitable than conventional placing methods.
2. Shuttering and formwork need be erected only on side of the work and hence there will be considerable saving in the shuttering costs.
3. It can be conveyed over a considerable diameter pipe, makes this process suitable for sites where access is difficult.
4. The maximum rate of deposition is about 15 m^3 hr for the dry process but this can be exceeded with the wet process.
5. The low water-cement ratio, the thinness of the section deposited and the fact that normally only one side of the concrete is covered, necessitates careful attention to curing more than with normal concrete.
6. The normal specifications with respect to cement, aggregate and water, also apply for shotcrete, but it is desirable that the aggregate should be harder to allow for attrition.
7. Admixtures can be used in shotcrete to produce the same effects as in ordinary concrete.
8. The drying shrinkage will depend on the water content and may, therefore, be expected to be fairly low for the dry process. The creep of the dry shocrete is similar to that of high quality normally placed concrete but shrinkage and creep of wet shocrete is likely to be high.
9. The durability or resistance to frost action and other agencies of dry shocrete is good.
10. About half of the entrained air is likely to be lost while spraying.

Epoxy Injection:

The Injection of polymer under pressure will ensure that the sealant penetrates to the full depth of the crack. The technique in general consists of drilling hole at close intervals along the length of cracks and injecting the epoxy under pressure in each hole in turn until it starts to flow out of the next one. The hole in use is then sealed off and injection is started at the next hole and so on until full length of the crack has been treated. Before injecting the sealant, it is necessary to seal the crack at surface between the holes with rapid curing resin.

For repairs of cracks in massive structures, a series of holes (Usually 20mm in dia and 20mm deep spaced at 150 to 300mm interval) intercepting the crack at a number of location are drilled. Epoxy injection can be used to bond the cracks as narrow as 0.05mm. It has been successfully used in the repair of cracks in buildings, bridges, dams and other similar structures. However, unless the cause of cracking is removed, cracks will probably recur possibly somewhere else in the structure. Moreover, in general this technique is not very effective if the cracks are actively leaking and cannot be dried out.

Epoxy injection is a highly specialized job requiring a high degree of skill for satisfactory execution. The general steps involved are as follows.

i. **Preparation of the surface**: The contaminated cracks are cleaned by removing all oil, grease, dirt and fine particles of concrete which prevent the epoxy penetration and bonding. The contaminants should preferably be removed by flushing the surface with water or a solvent. The solvent is then blown out using compressed air, or by air drying. The surface cracks should be sealed to keep the epoxy from leaking out before it has cured or gelled. A surface can be sealed by brushing an epoxy along, the surface of cracks and allowing it to harden. If extremely high injection pressures are needed, the crack should be routed to a depth of about 12mm and width of about 20mm in V-shape, filled with an epoxy, and stuck off flush with the surface.

ii. **Installation of entry ports**: The entry port or nipple is an opening to allow the injection of adhesive directly into the crack without leaking. The spacing of injection ports depends upon a number of factors such as depth of crack, width or crack and its variation with depth, viscosity of epoxy, injection pressure etc. and choice must be based on experience. In case of V-grooving of the cracks, a hole of 20mm dia and 12 to25mm below the apex of V-grooved section, is drilled into the crack. A tire-calue stren is bonded with an epoxy adhesive in the hole. In case the cracks are not V-grooved, the entry port is provided by bonding a fitting, having a hat-like croos-section with an opening at the top for adhesive to enter, flush with the concrete face over the crack.

iii. **Mixing of epoxy:** The mixing can be done either by batch or continuous methods. In batch mixing, the adhesive components are premixed in specified proportions with a mechanical stirrer, in amounts that can be used prior to the commencement of curing of the material. With the curing of material, pressure injection becomes more and more difficult. In the continuous mixing system, the two liquid adhesive components pass through metering and driving pumps prior to passing through an automatic mixing head. The continuous mixing system allows the use of fast-setting adhesives that have short working life.

iv. **Injection of epoxy:** In its simplest form, the injection equipment consists of a small reservoir or funnel attached to a length of flexible tubing, so as to provide a gravity head. For small quantities of repair material small hand-held guns are usually the most economical. They can maintain a steady pressure which reduces chances of damage to the surface seal. For big jobs power-driven pumps are often used for injection. The pressure used for injection must be carefully selected, as the use excessive pressure can propagate the existing cracks, causing additional damage. The injection pressures are governed by the width and depth of cracks and the viscosity of resin and seldom exceed 0.10Mpa. It is preferable to inject fine cracks under low pressure in order to allow the material to be drawn into the concrete by capillary action and it is a common practice to increase the injection pressure during the course of work to overcome the increase in resistance against flow as crack is filled with material. For relatively wide cracks gravity head of few hundred millimeters may be enough.

v. **Removal of surface seal:** After the injected epoxy has occurred; the surface seal may be removed by grinding or other means as appropriate. Fittings and holes at the entry ports should be painted with an epoxy patching compound.

SHORING is the term given to a method of temporarily supporting buildings by a framing of timber acting against the walls of the structure. If the frame consists of more than one shore, it is called a system; if of two or more systems, it becomes a series.

The forces that tend to render a building unstable are due primarily to gravity, but owing to the various resistances set up by the tying together of the building, the force does not always exert itself vertically downwards.

This instability may arise from various causes, the most common being the unequal settlement of materials in new buildings, the pulling down of adjoining buildings, structural alterations and defects, and alterations or disturbances of the adjacent ground which affect the foundations. The pulling down of an adjoining building would, by removing the corresponding resistance, allow the weight of the internal structure of the building to set up forces which at first would act in a horizontal direction outwards. Structural defects, such as an insufficiently tied roof truss, would have the same effect. Structural alterations, such as the removal of the lower portion of a wall in order to insert a shop front would allow a force due to gravity to act vertically downwards.

To resist these forces, three different methods of shoring are in general use, and they are known as flying or horizontal shores, raking shores, and underpinning.

Flying Shores.—Where the thrusts acting upon the wall are in a horizontal direction, flying or raking shores are used to give temporary support. The most direct resistance is gained by the first-named, the flying or horizontal shore. There are, however, limits to its application, as, owing to the difficulty of obtaining sound timber of more than 50 to 60 feet in length, a solid body is necessary within that distance, from which the required purchase can be obtained.

It is a method of shoring generally used where one house in a row is to be taken down, the timbers being erected as demolition proceeds, and taken down again as the new work takes its place.

Fig. 47 shows a half-elevation of two general systems of construction.

The framing, as at A, may be used alone where the wall to be supported is of moderate height and the opening narrow, but larger frames should be combined, as at B.

The framework C is for wide openings and walls of considerable height.

The wall plates, 9 in. by 2 in. or 9 in. by 3 in., are first fixed vertically on the walls by wall hooks. Then, in a line with the floors, rectangular holes 4 in. by 3 in. are cut in the centre of wall plates. Into these holes, and at least 4½ inches into the brickwork, needles (also known as tossles and joggles) of the same size are fitted, leaving a projection out from the wall plate of 5 in. or 6 in., sufficient to carry the shore of about 7 in. by 7 in.

The shore, prior to being fixed, has nailed on its top and under sides straining pieces 2 inches thick, and of the same width as the shore. To tighten, oak folding wedges are driven at one end between the shore and wall plate.

To stiffen the shore, and to further equalise the given resistance over the defective wall, raking struts are fixed

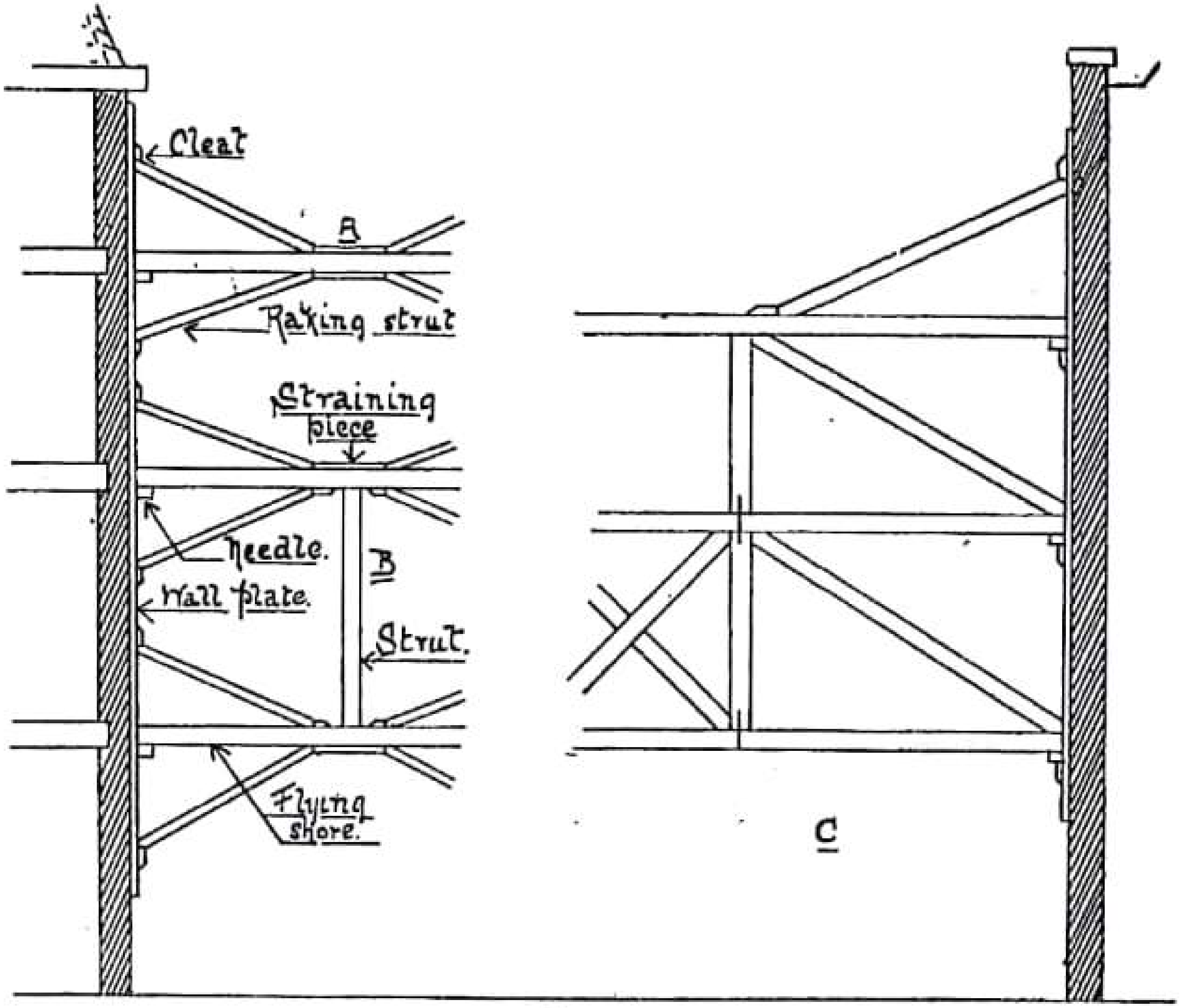

FIG. 47

between the straining pieces, and cleats are nailed above and below the shore. These raking struts are tightened by driving wedges between their ends and the straining pieces.

The cleats previous to, and in addition to being nailed, should be slightly mortised into the wall plate. This lessens the likelihood of the nails drawing under the pressure.

A Raking Shore consists of a triangulated system of timber framing, and is used to support defective walls where the resistance to the threatened rupture has to be derived from the ground surrounding the building.

In its simplest form a raking shore is a balk of timber of varying scantlings, but as a rule of square section, inclined from the ground to the defective wall. The angle of inclination is taken from the horizon, and should vary between 60 and 75 degrees. In settling this the space available at the foot of the wall has to be taken into consideration, especially in urban districts where the wall abuts on the footpath.

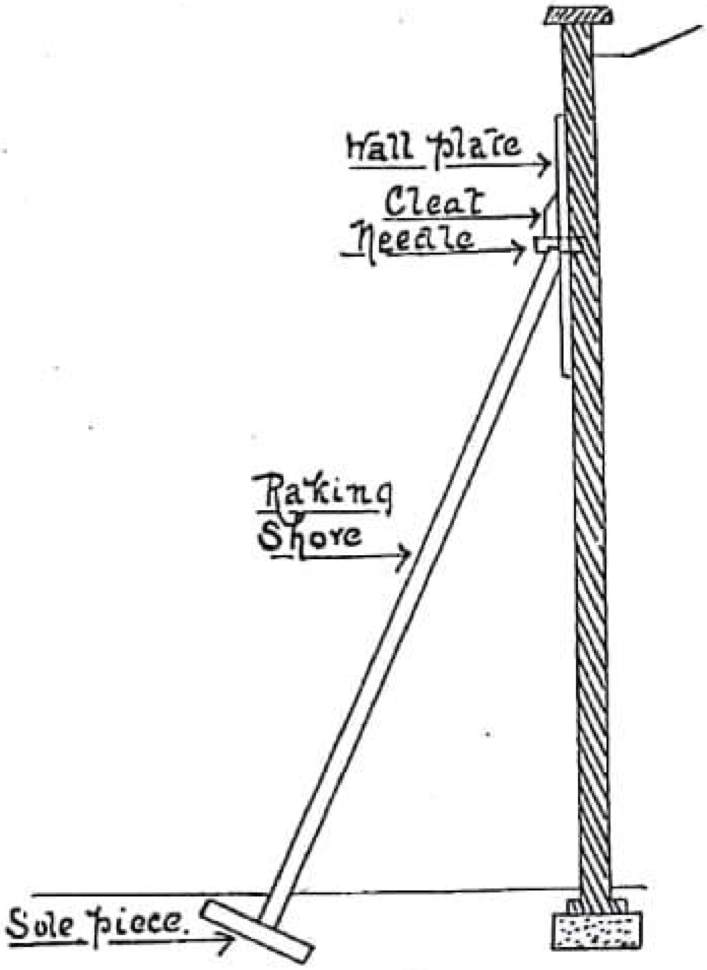

FIG. 48

Fig. 48 shows a raking shore in its simplest form, but usually two or more shores are used (see fig. 49).

The following table from Mr. Stock's book[1] shows the general rule and also the scantlings to be used:

For walls from	15 to 30 feet high	2	shores are necessary	in each system
"	30 " 40 "	3	"	"
"	40 and upwards	4	"	"

[1] *A Treatise on Shoring and Underpinning, and generally dealing with Dangerous Structures.* By Cecil Haden Stock. Third edition, revised by F. R. Farrow. (B. T. Batsford.)

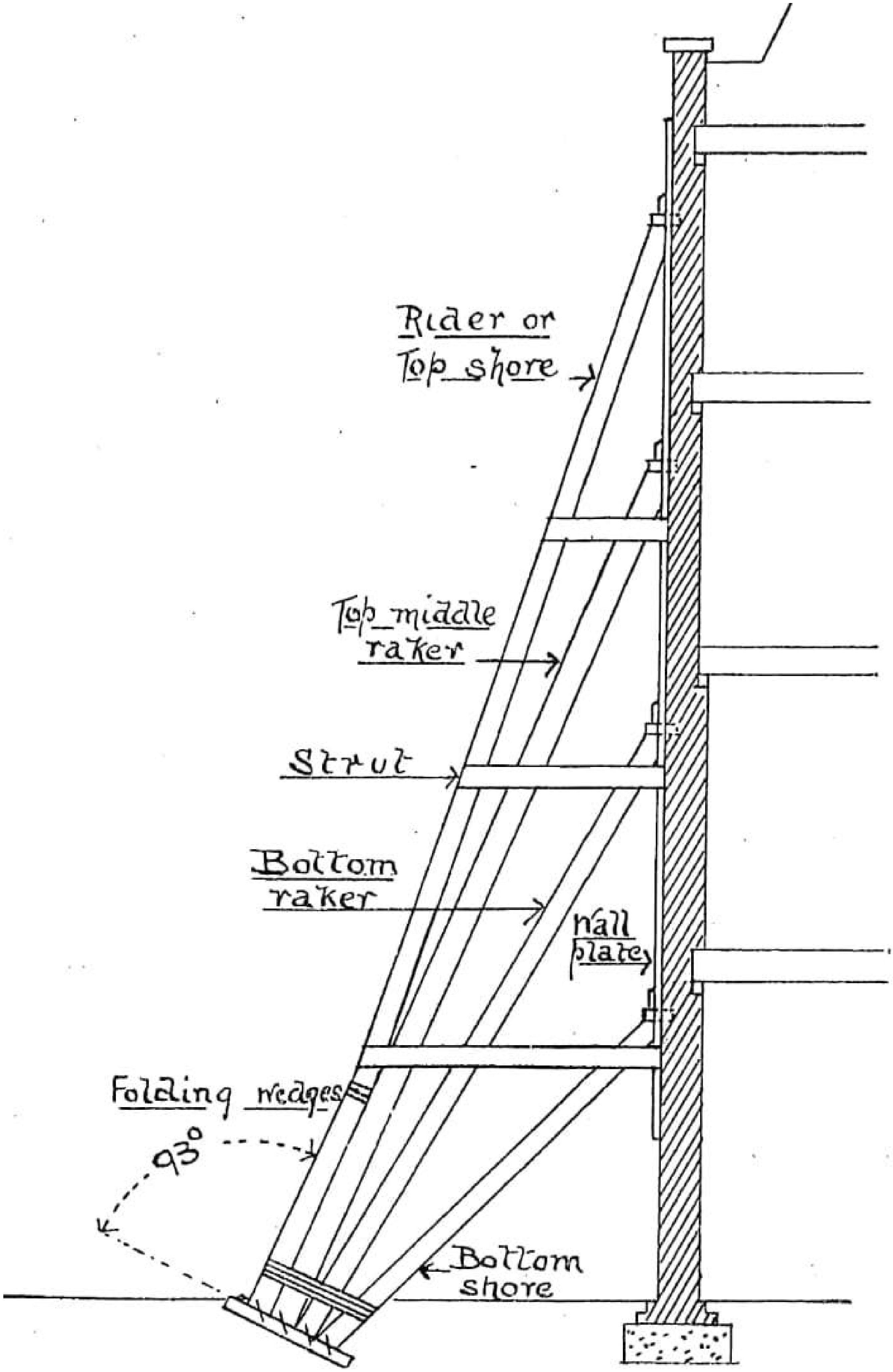

FIG. 49.—EXAMPLE OF RAKING SHORE

Underpinning.—Underpinning is necessary to carry the upper part of a wall, while the lower part is

[redacted]

removed ; for instance, the insertion of a shop front, or the repairing of a foundation. It is only kept in position until a permanent resistance to the load is effected. Underpinning is, as a rule, unnecessary when the opening to be made is of less width than five feet. This method of shoring is a simple operation, but yet requires great care in its execution.

The first thing to be done is to remove from the wall all its attendant loads. This is accomplished by strutting from the foundation floor upwards from floor to floor until the roof is reached (see fig. 53).

Header and sole plates 9 in. by 2 in. are put in at right angles to the joists in order to give bearing to the struts.

The portion of the wall to be taken down having been marked out, small openings are made, slightly above the proposed removal, at from 5 to 7 feet apart, and through these, at right angles to the face of the wall itself, steel joists or balk timbers 13 in. by 13 in., called needles, are placed. These are supported at each end by vertical timbers 13 in. by 13 in., called dead shores, which again rest upon sleepers.

The sleepers serve as a bed to the dead shores to which they are dogged, and by distributing the weight over a larger area, they prevent the dead shores sinking under the pressure. The dead shores, if well braced, may be of smaller scantling.

Where it is impossible to arrange for the dead shores to be in one length, the lower pieces are first fixed. They must be of uniform length, and across their top end a transom is carried to support the upper pieces, the bottom ends of which must stand directly over the top ends of the lower pieces (see fig. 53).

Having placed all the timbers in position, and before the tightening up takes place, the windows or

other openings in the wall are strutted to prevent any twisting which may take place. This is done as shown

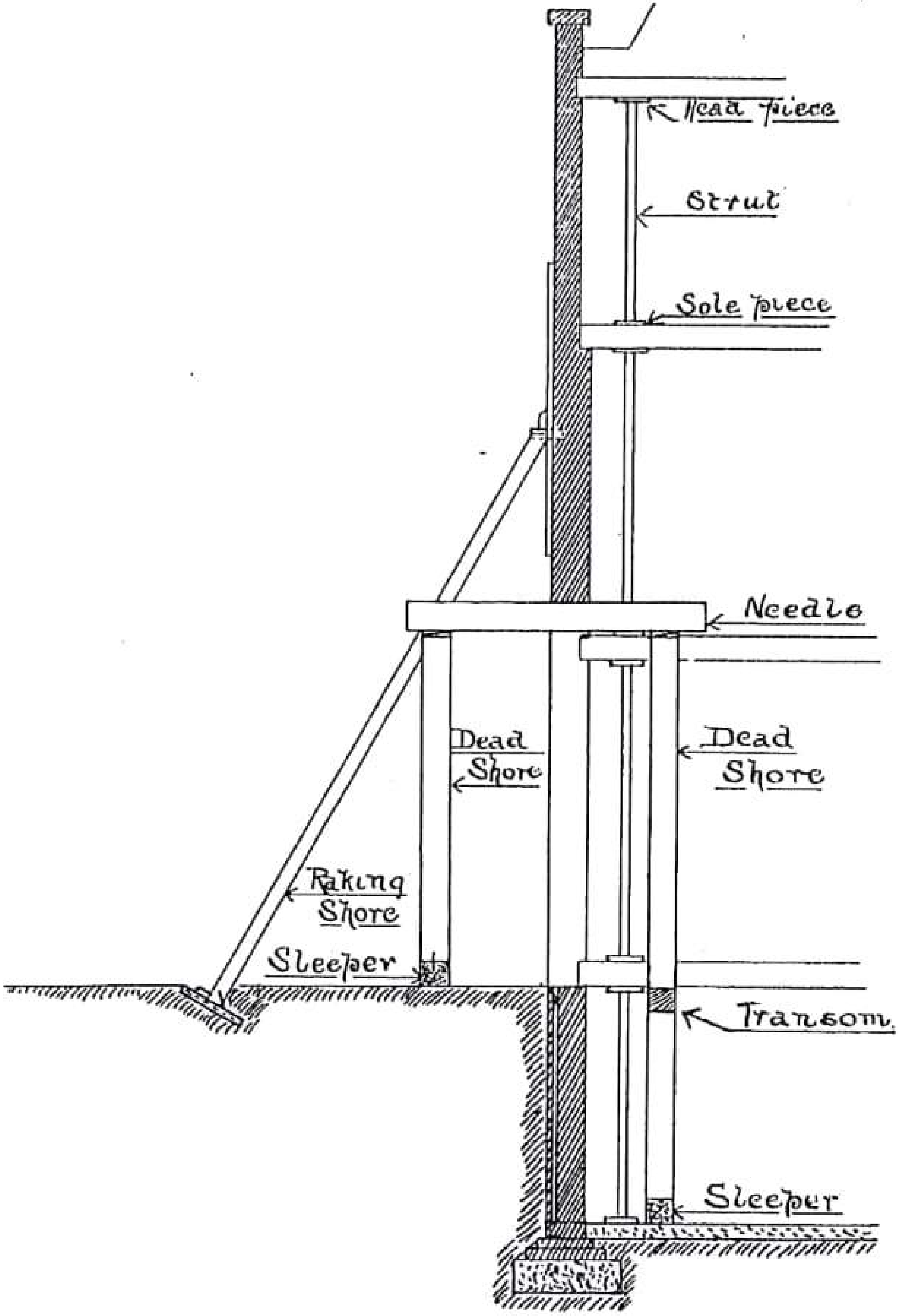

FIG. 53.—EXAMPLE OF UNDERPINNING

on fig. 54, but small windows do not require the centering.