Chapter two

Literature review

2.1: Introduction

We come across many old buildings needing major repairs or go early in to a state of dilapidation condition to make them unfit for occupation. However, if a building has given about 25 to 30 years of service without much maintenance or major repair, then it is reasonable to expect that it would need some structural repair soon. The main cause for this is weathering and ageing effect or inadequate maintenance and care. However, generally at an age of less than 10 years; many poorly designed and/or constructed buildings are found to be in a very bad structural and general health condition needing major structural repairs. This premature deterioration is largely due to poor construction or inappropriate design and /or neglect of timely repairs. The premature deterioration of structure is an economic burden not only on owners but also to the municipalities and nation as a whole. The longer life of structures enables better utilization of natural resources. Reduction in waste due to demolition enhances energy conservation. Such philosophy will be in harmony with the environment and help in achieving sustainability of system. It would also lead to benchmark standards of maintenance and upkeep of buildings. of course the buildings/structures should also be appropriately designed for resistance to natural disasters like earthquakes, cyclones, floods etc.

2.2 Expected Service Life of Structures

There is very little literature available on the subject of expected service life of structures. The lifespan of RC generally is taken as 100 years. However, there are some expected as well as prevalent conventions about design life span, which are given here [1]:

- Monumental Structures like Temple, Mosque or Church etc - 500 to 1000 years.
- Steel Bridges, Steel Building or similar structures - 100 to 150 years.
- Concrete bridges or High rise building or stone bridges etc - 120 years.
- Residential houses or general office/commercial buildings etc - 60 to 80 years.
- Concrete pavements - 30 to 35 years.
- Bituminous pavements - 8 to10 years.

The functional utility as well as aesthetics of building are important in the architectural design process. But, durability is the most important in structural design and laying down specifications for construction for achieving service life of structure. Thus, all three aspects (via architectural, structural and construction) are important. Yet, many times, there are clashes between the three.

2.3 factors for durability of concrete for adoption during design / construction stages: [1].

a. Environment or exposure condition,
b. Cover to embedded steel,
c. Type and quality of constituent materials,
d. Cement content and water/ cement ratio,

e. Workmanship,
f. Shape and size of the Structural member.

(a) Exposure to highly aggressive environment

Relatively new but severely distressed buildings are observed in the locations, close to sea or creek. Corrosion of reinforcement and distress in concrete in the structural members (columns, beams, slabs) in some of them could be alarming. It is mainly due to high concentration of chlorides and/or sulphates in the ground water and saline environment around the structure. Moisture from soil could rise. The absence of damp proof course at the DPC level could allow the dampness to rise quite high. This combined with substandard materials and workmanship could get further aggravate deterioration process. Thus, extra care must be taken in materials selection and have proper control on quality of concrete in structures, close to sea, as shown in Fig.(2.1) below.

Fig. [2.1]: Sulphate attack on pipe pedestal.

(b) Reinforced Cement Concrete

Concrete came into existence in last 200 years or so and it was expected that concrete should behave like stone structures. But with the introduction of steel reinforcement in concrete, its structural utility has been enhanced for taking tensile loads. Hence, reinforced cement concrete has now become one of the most important materials of construction all over the world. However, because of its heterogeneous character, the durability of structure has assumed much importance in last three decades. Durability is affected because of poor quality of its constituent materials and workmanship could lead to early deterioration. Chlorides may also be present in these materials which lead to early and faster corrosion Reinforcement. Poor quality of concrete materials also affects quality of concrete:

- Quality of cement,
- Type and quality of coarse aggregates,
- Fineness of sand and silt contents therein,
- Concrete mix design etc.

(c) Construction

Concrete could become a treacherous construction material, if not manufactured correctly and not compacted fully. It may show low strength and high permeability. Though, it does not show signs of immediate weakness, but only after about five to ten years of construction, (depending upon the environmental conditions), signs of deterioration become visible. Therefore, for executing good construction at site, we need:

- Good quality of construction material,
- Appropriate methodology,
- Trained manpower,
- Appropriate machinery.

2.4 Causes of Early Deterioration of Concrete Structures

Newly constructed RC structures are failing in a fraction of its design life span. Therefore, the causes of premature deterioration in relatively new buildings are different as compared to those for old buildings. Hence, the Approach for the repair (or restoration) of such buildings should be quite different from that of old buildings. The process of repair should start with a thorough visual survey, followed by non-destructive testing of the structural elements and chemical tests on concrete and ground water. Generally, the main reason is poor or incorrect design and/or poor quality of materials. There may be several other reasons also as described below [2].

1- Poor workmanship & quality of construction

Substandard workmanship in RC can be in the form of honey combing (insufficient compaction of concrete) or inadequate cover to reinforcement (improper placement of bars) or both. These lead to early corrosion of reinforcement especially in thin structural member. In masonry and plaster, poor workmanship could be in the form of loosely fitted masonry joints (within walls or between external walls and beams / columns), poor lines and levels and hollow plaster etc. They lead to excessive seepage. Some of these deficiencies may become evident only after the full loading of structure has been put to use like 5 to 7 years.

Further, necessary steps must be taken at construction site which calls for increase in number and proper mix of knowledge, skills and attitudes, are shown in Fig. (2.2) and (2.3) below.

Fig. [2.2]: Poor Concreting in column.

Fig. [2.3]: Poor quality of concrete in cover and ingress of moisture.

2- Effect of Climate

Climate plays a significant role in the decay of structure. Prolonged exposure to polluted environment and acid rain can deteriorate concrete or dissolve bricks and will also corrode embedded or exposed metal ties and fastenings. High levels of moisture and excessive fluctuations in heating and cooling can promote the movement of soluble salts. Salt movement is characterized by patches of white crystals in the surface of walls and can cause considerable damage. Frost can also contribute to the decay due to freezing and expansion of embedded water and resultant cracking of surface concrete.

3- Inadequate cement quantity:

IS:456-2000 has laid down from durability considerations, the minimum cement content in concrete, irrespective of strength depending upon the exposure conditions to which the structure. Many design and construction engineers overlook this codal requirement and even unscrupulous contractors use less cement than that specified. The minimum quantity of cement is needed not only to coat the fine and coarse aggregate particles but also to fill the voids between the aggregate particles and to provide a thicker film of cement grout for easy workability. Thus, the aggregate particles slide over each other, during compaction of concrete [2].

4- Excessive water cement ratio

For concrete to be durable there is a maximum w/c ratio specified for mixing concrete, as well as to give proper workability and concrete strength. Again the IS 456:2000 has laid down upper limits of water cement ratio. Normally, it needs about 11.5 litters of water (say w/c = 0.23) per 50 kg bag of cement for hydration process. However, the concrete will be stiff and un-compactable with this quantity of water. Therefore, additional water and super-plasticizer is added to make the concrete workable. This extra water (not needed for Hydration) eventually evaporates and leaves minute capillary pores which permit the ingress of moisture and pollutants which lead to slow corrosion of steel bars and ultimate disintegration of the concrete. The objective is to add only the water as per concrete mix design. Many a times, the concrete is made at site and contractors use more than the permissible water to make concrete workable because of following:

- The contractors may not have or like to use vibrators.
- Masons want fluid concrete for easy placing and compaction of the concrete.
- The sand with excessive silt (clay) demands more water to maintain workability (high slump) desired by the workers.
- The coarse aggregates (stone metal) are excessively flaky instead of cubical, due to bad crushers. The extra surface area of the flaky stone particles, demand extra water to maintain easy workability desired by workers.

5- Inadequate concrete cover:

The smiths who fix reinforcement bars are neither trained to bend the bars accurately nor to fix them effectively to ensure that the specified cover is left between bars and the formwork (shuttering). Quite often, not only the bars themselves touch form work but also the binding wire loose ends and the steel bars are seen at the surface of the concrete and they are subjected to early carbonation of concrete.

6- Honeycombed or Un-vibrated concrete:

Honeycombed concrete is a major source of weakness in concrete and cause of safety concern especially in multi-storied buildings, due to inadequate vibration/compaction in columns, walls, beams and slabs. Use of form vibrator is essential for narrow walls, partitions and architectural fins.

7- Cold joints or bad construction joints:

Most construction site personnel do not plan properly the sequence of pouring concrete to minimize the number of construction joints. They do not take adequate precautions to eliminate cold joints. A cold joint is a joint where fresh concrete is placed against a previous un-compacted concrete which has already hardened due to lapse of concrete setting time. Therefore, the fresh concrete will not homogeneously merge with the older concrete, as shown in Fig. (2.4) below.

Fig. [2.4]: Less cover in slab bottom and cold joint.

8- Alkali-Aggregate Reactivity:

Sometimes, chemical reaction occurs between reactive siliceous minerals or carbonates, present in aggregate and the alkaline hydroxides derived from hydrated cement. The result of reaction is formation of alkali-silicate gel of 'unlimited swelling' type. These reactions are called; (I) Alkali-silica reaction, and (II) Alkali-carbonate reaction. Because the gel is confined by the surrounding cement paste, so the internal pressure develops and causes cracking and disruption of concrete. Under most conditions, this very slow reaction causes excessive expansion and cracking of concrete after few years. Aggregates containing particular varieties of silica are susceptible to attack by alkalis (Na_2O and K_2O) originating from cement, admixtures or other sources, producing an expansive reaction. Aggregates petro-graphically known reactive type or aggregates which, on the basis of past history or laboratory experiments are suspected to have reactive tendency are needed to be avoided in concrete or used only with cements of low alkalis [not more than 0.6 percent as sodium oxide (Na_2O)]. Use of pozzolanic cement and certain pozzolanic admixtures [i.e. Fly ash] may be helpful in controlling alkali aggregate reaction. Alternately use non-reactive aggregate from alternate sources [2].

9- Initially rust steel bars:

Often steel bars are stored in open areas, exposed to rain and atmospheric moisture resulting in rusting of them. The corrosion process starts rapidly in the presence of moisture (especially in coastal areas). The steel bars are rarely wire-brushed and cleaned thoroughly before being placed in shuttering prior to concreting. In other cases, due to suspension of work due to reasons whatsoever, structural frame remains exposed to sun, rain and misuse for a long duration. Such prolonged exposure to weather can Cause rusting of rebars and carbonation to adversely affecting durability of the building frame.

10- Congested reinforcement bars:

Sometimes design engineer provides too many steel bars in the narrow and slender RC columns, walls or beams which lead to practically no cover in concrete or even space for inserting a needle vibrator to ensure full compaction. This results in honeycombed concrete. Through honeycombing, the moisture and atmospheric pollution enters the steel bars and thus starting the corrosion process.

11- Porous cover blocks:

The cover blocks are invariably made at site with no attention to the correct mix proportion or the specified water cement ratio. The cover blocks are usually fixed to the steel bars at about one meter centres and if they are porous they become the starting source of decay of concrete as they permit the ingress of moisture which corrodes the steel. If dense concrete cover blocks cannot be made then it is preferable to use plastic cover blocks which are now available.

2.5 Corrosion of reinforcement

Corrosion of reinforcement in concrete is an electro-chemical process. The following reaction will take place during the corrosion process of steel:

Iron + oxygen + water → products of corrosion

This electro-chemical reaction is composed of four partial processes, which occur at the same time, see Fig. (2.5)[8].

- Oxidation of iron (anodic process). Iron has a natural tendency to oxidize, where it liberates electrons and forms ferrous ions;
  ($Fe \rightarrow Fe^{2+} + 2\ e^{-}$).
  Subsequent hydrolysis produces acidity;
  ($Fe^{2+} + 2H_2O \rightarrow Fe(OH)_2 + 2H^{+}$).
- Reduction of oxygen that consumes these electrons and produces alkalinity (cathodic process):
  $O_2 + 2H_2O + 4e^{-} \rightarrow 4OH^{-}$
- Transport of electrons within the metal from the anodic areas where they become available, to the cathodic regions where they are consumed.
- The flow of current inside the concrete from the anodic regions to the cathodic ones, transported by ions in the pore solution.

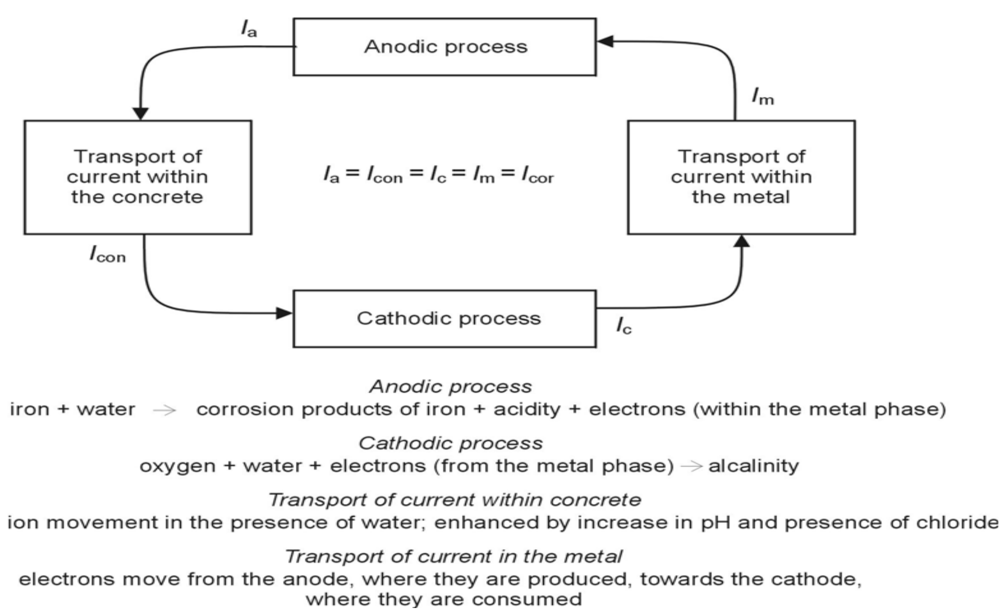

Fig. [2.5]: Mechanism of corrosion of steel in concrete [8].

2.5.1 Modelling of Reinforcement Corrosion

The process of corrosion can be divided into two different periods: the initiation and the propagation period see Fig. (2.6). during the initiation phase chloride ions penetrate from the surface into the concrete cover and reach the reinforcement. When the concentration of the chloride at the surface of the reinforcement reaches the critical level, breakdown of the protective layer of the reinforcement will occur. This indicates the termination of the initiation period. After initiation has occurred the propagation period starts. Corrosion propagation will occur only if water and oxygen are present on the surface of the reinforcement. As can be seen from the Fig. (2.6), during the propagation period the corrosion penetration depth increases rapidly in time resulting in a reduction of the reinforcement area.

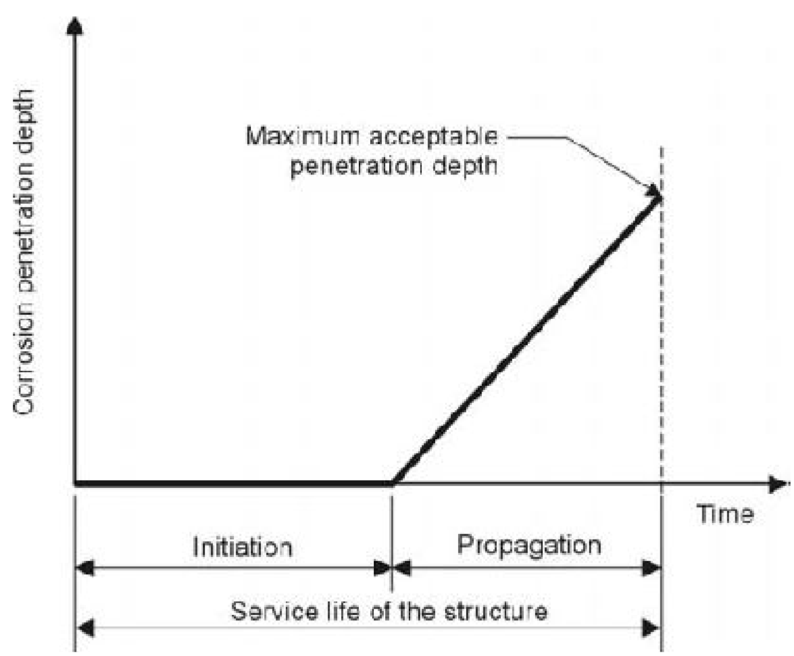

Fig. [2.6]: Initiation and propagation periods for corrosion in a reinforced concrete structure [8].

During those periods certain events occur which continue until a Limit State (LS) is reached. Those limit states are represented in Fig. (2.7).

1. *Depassivation of the reinforcement*. This event marks the end of initiation phase. The cross sectional area of the reinforcement in this limit state is unchanged.

2. *Formation of cracks*. The cross section of the reinforcement reduces progressively due to the corrosion process after the reinforcement has become depassivated. In addition, the corrosion products cause internal pressure thus cracks will start to appear in the concrete. The limit state 2, formation of cracks (see Fig. 2.7), is reached when a visible crack width of 0.3 mm appears at the concrete surface [19].

3. *Spalling of the concrete cover*. Corrosion continuing after cracking has occurred may lead to spalling of the concrete cover. Referring to Fig. (2.7), the limit state, spalling of the concrete cover is reached when a crack width of approximately 1.0 mm [19].

4. *Collapse of the structure*. Collapse of the concrete structure will occur if the load carrying capacity of the element is reduced sufficiently due to ongoing corrosion.

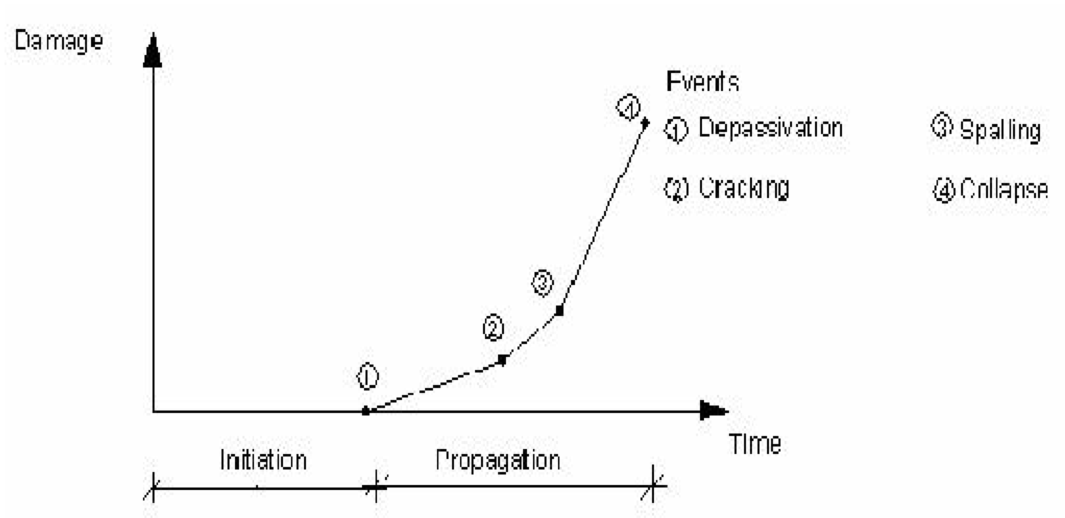

Fig. [2.7]: Events related to the service life.

The corrosion rate in the presence of chlorides is mainly determined by the amount of moisture in concrete. The lower the moisture content, the lower the corrosion rate. In structures that are exposed to the atmosphere, the corrosion rate can vary from several tens of µm/year to localised values of 1mm/y. The corrosion rate is also dependent on the concrete resistivity. When corrosion is started, there will be a lower corrosion rate in blended cements, which have a higher resistivity compared to Portland cement [8].

2.5.2 Corrosion consequences of concrete structures

Corrosion has two effects. First, corrosion reduces the cross-section area of the reinforcement, which influences the load-carrying capacity of the structure.

Second, the corrosion products have a larger volume than the steel, causing internal pressure. Due to the internal pressure caused by the expansion of rust products cracks will be formed in the concrete and the concrete cover will spall, see Fig. (2.8) [9].

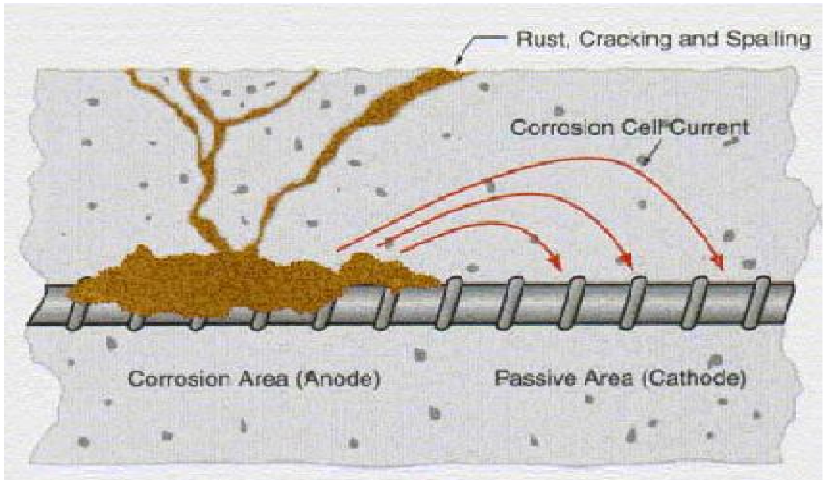

Fig. [2.8]: Consequences of reinforcement corrosion in concrete [9].

2.5.3 Chloride induced corrosion

Corrosion by chlorides is mostly localized. Such localized corrosion is also called pitting corrosion. Pitting corrosion is caused by chlorides that locally damage the protective oxide film on the reinforcement in alkaline concrete. Areas that are no longer protected act as anodes (active zones) while the surrounding areas are still passive, see Fig. (2.9). A very aggressive environment will develop inside the pits as the chloride content will increase in that area and the alkalinity is lowered. This causes an acceleration of the corrosion [8].

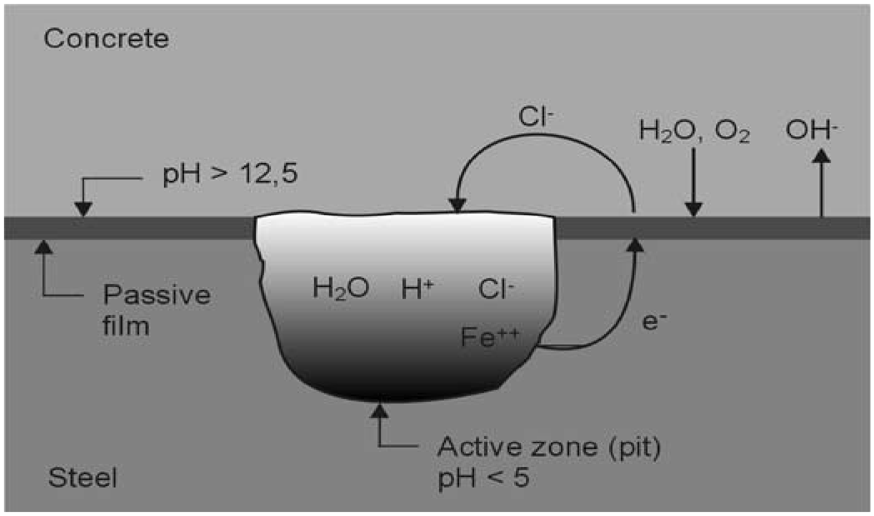

Fig. [2.9]: Pitting Corrosion [8].

2.5.6 Uniform and pitting corrosion

A high alkalinity concrete environment forms a passive layer around the reinforcement, which significantly protects the bar from corrosion attack. The corrosion is not initiated as long as the passive layer is sustained. Two processes, carbonation of concrete and chloride attack, may destroy the protection layer. Depending on the type of deterioration process, corrosion of reinforcement may take different forms, ranging from a very widespread to a very local damage, known as uniform and pitting corrosion, respectively. When there is uniform or pitting corrosion of a steel bar, the effective reinforcement area is either evenly or locally reduced. The reduction of reinforcement area, or diameter, is most accurately obtained by direct measurements. For a corroded structure with cover spilling, the remaining bar diameter can be measured on the exposed bars after removal of the rust layer. For less corroded structures where the cover has not yet spelled off, small parts of the cover could be removed at non-critical locations, and afterwards repaired. An alternative to direct measurement is to estimate the corrosion penetration based on corrosion rate and time of corrosion initiation. Then the reduction of the effective reinforcement diameter due to uniform corrosion is calculated [12];

$$\emptyset = \emptyset_0 - 2x \qquad \ldots\ldots\ldots\ldots..(2.1)$$

Where;

$\emptyset$ $\equiv$ is the remaining effective diameter of the reinforcement ,

$\emptyset_0$ $\equiv$ is the original diameter, and x is the corrosion penetration.

2.5.7 Bond between corroded reinforcement and concrete

The bond between reinforcement bars and concrete is influenced by several parameters, such as concrete cover, transverse reinforcement, concrete strength, lateral pressure, and the yielding and spacing of the reinforcement bars [14], as shown in Fib (2000). Low bond stresses are resisted mainly by chemical adhesion. Additional bond stresses lead to a failure of the chemical adhesion and cause transverse cracks that radiate from the tip of the ribs, Fig. (2.10). For high loads, the cracks spread radically and the bond stresses extend outwards into the concrete. The stress can be divided into longitudinal bond stress and normal splitting stress. The splitting stresses are resisted by ring stresses in the concrete around the bar, see Fig. (2.11).

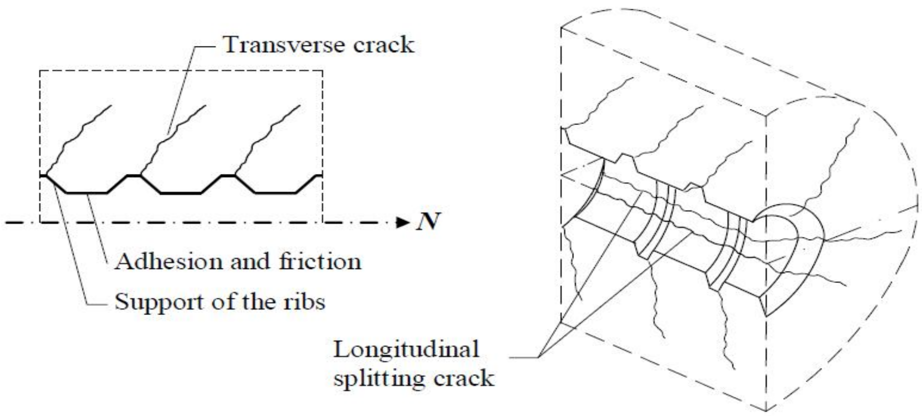

Fig. [2.10]: cracking caused by bond, modified by magmusson (2000) from vandewalle (1992).

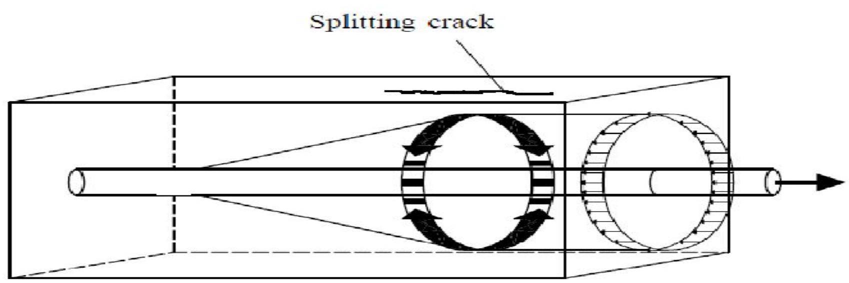

Fig. [2.11]: Tensile ring stresses in the anchorage zone, from Tepfers (1973).

The interaction between the reinforcement and the concrete is governed by the splitting stresses and by the friction between the reinforcement and the concrete. Corrosion causes volume expansion leading to splitting stresses that act on the surrounding concrete, thereby reducing the bond properties [13].

2.5.8 Mechanical behavior of corroded reinforced concrete Structures

The mechanical behavior of reinforced concrete structures, in terms of load-carrying capacity, as well as stiffness and force redistribution, is affected by the corrosion of reinforcement, see Fig. (2.12), both uniform and pitting corrosion reduce the reinforcement bar area and ductility, which causes volume expansion. Reduction of the reinforcement bar area leads to decreased shear and moment capacities as well as decreased stiffness of the structure. A change in rebar ductility directly influences the stiffness of the structure, the possibility for force and moment redistribution, and limits the load-carrying capacity of a statically indeterminate structure. Moreover, volume expansion of reinforcement bars may cause the surrounding concrete to crack and spall off, which decreases the concrete cross-section and concrete cover. On the compressive side of a concrete element, spalling of the cover decreases the internal lever arm, which in turn decreases the bending moment. Furthermore, reduced confinement influences the interaction between the reinforcement and the concrete, which affects the anchorage capacity [15].

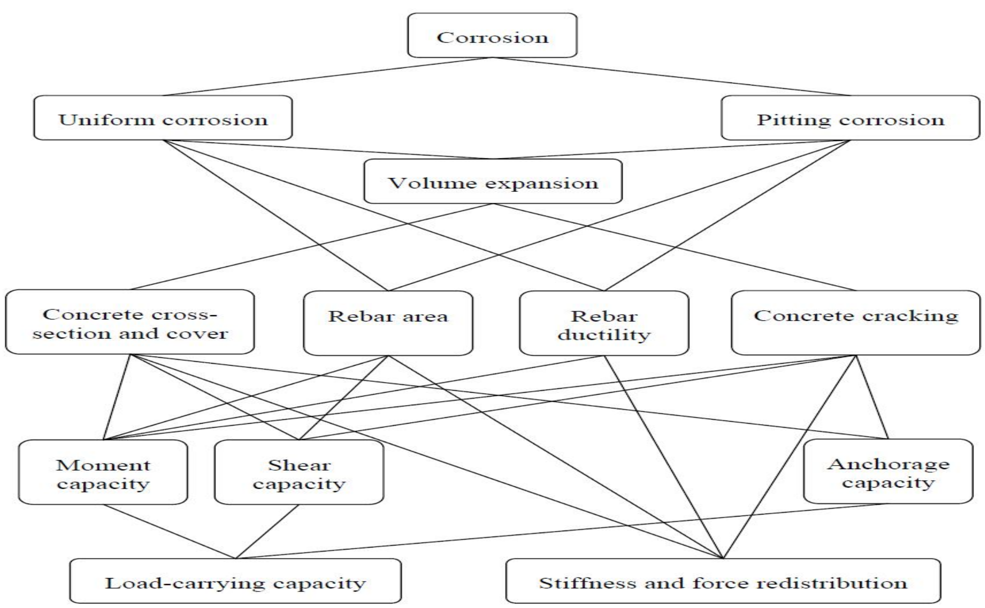

Fig. [2.12]: Effect of corrosion on load-carrying capacity, stiffness and force redistribution of a concrete element.

2.5.9 Modeling the effects of corrosion on the structural level

The methodology is based on the assumption that the usual method of structural analysis for concrete structures should be applied also to corroded reinforced concrete structures. It is assumed that the effect of corrosion can be modeled as a change in the geometry and material properties of the concrete, reinforcement and their interface through the following steps [11].

(1) If corrosion caused the concrete to spall off, the effect on both the concrete cross-section and the cover loss can be taken into account by modifying the geometry used in the analysis. In compression regions where corrosion leads to cracking of concrete, lower strength and stiffness than for the virgin concrete should be assigned to cracked concrete. The behaviour of concrete around corroded stirrups can be simulated by adapting lower tensile strength. The method of adjusting compressive and tensile strength of cracked concrete was suggested in [21]

(2) Reduction of the effective reinforcement area by both uniform and pitting corrosion is the most obvious effect to take into account. The actual area of a uniformly corroded bar can be calculated by assuming that corrosion has penetrated evenly around the bar. However, pitting corrosion affects the reinforcement locally; therefore, measurement or estimation of the pitting configuration is needed to be able to calculate the residual bar area. Finally, the ductility of corroded reinforcement can be calculated using practical models in which the residual ductility is confined to empirical correlations with area loss of the corroded reinforcement.

(3) Corrosion affects the interaction of reinforcement and concrete. Therefore, the bond-slip relationship should be modified accordingly. The modification could be done according to the method proposed in [20]. This

procedure can be applied to models at structural level where the bond-slip between the concrete and reinforcement is modeled by one-dimensional bond-slip relation, for example in plane-stress analysis. For simpler structural analysis models, such as beam-element analysis, where the bond slip is not directly accounted for in the model, it is proposed that the procedure described in Paper I can be applied to calculate anchorage length. Either the capacity of the reinforcement is then adjusted in the anchorage region, or the anchorage is checked manually.

2.6 Repair strategies

Numerous repair options are available and new technologies continue to make an impact in the field of concrete repairs. The suitability and cost effectiveness of repairs depends on the level of deterioration and specific conditions of the structure [9].

a) Patch repairs,

b) Coating systems,

c) Migrating corrosion inhibitors,

d) Electrochemical techniques,

e) Cathodic protection systems,

f) Demolition/reconstruction.

a) Patch repairs

Before patch repairs are considered it is important that the distinction between chloride- and carbonation-induced corrosion is appreciated. As a general rule chloride-induced corrosion is far more pernicious and difficult to treat than carbonation-induced corrosion. This often dictates a completely different approach to repairing damage due to the two types of corrosion. Carbonation-induced corrosion causes general corrosion with multiple pitting along the reinforcement. Carbonated concrete tends to have fairly high resistivity that discourages macro-cell formation and allows moderate corrosion rates. Steel exposed to corrosive conditions will therefore show signs of corrosion that can be easily identified (e.g. sur-face stains, cracking or spilling of concrete). Repairs are generally successful provided all of the corroded reinforcement is treated.

Chloride-induced corrosion is characterized by pitting corrosion with distinct anode and cathode sites. The presence of high salt concentrations in the cover concrete means that macro cell corrosion is possible with relatively large cathodic areas driving localized intense anodes. High corrosion rates can be sustained under such conditions resulting in severe pitting of the reinforcement and damage of the surrounding concrete. Much of the reinforcement may be exposed to corrosive conditions without showing any signs of corrosion; this is particularly noticeable when corroded structures are demolished.

Localized patch repairs of areas of corrosion damage are popular due to their low cost and temporary aesthetic relief. This form of repair has limited success against chloride-induced corrosion as the surrounding concrete may be chloride-contaminated and the reinforcement is therefore still susceptible to corrosion. The patched area of new repair material often causes the formation of incipient anodes adjacent to the repairs as shown in Fig. (2.13). These new corrosion sites not only affect the structure but often also undermine the repair leading to accelerated patch failures in as little as two years. Consequently, it is necessary to remove all chloride-contaminated concrete from the vicinity of the reinforcement.

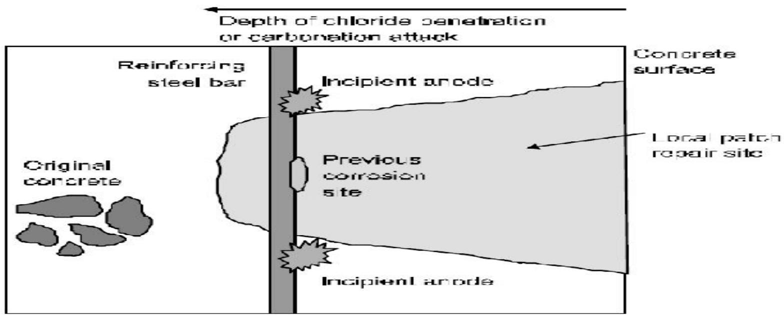

Fig. [2.13]: Formation of incipient anodes after patch repairs.

Complete removal of chloride-contaminated concrete, where it is possibleshould successfully halt corrosion by restoring passivating conditions to the reinforcement. Mechanical removal of cover concrete is usually done with pneumatic hammer, hydro jetting or milling machines. This form of repair is most successful when treating areas of localized low cover, before significant chloride penetration has occurred. If repairs are only considered once corrosion damage is fairly widespread it will be expensive to mechanically remove chloride-contaminated concrete from depths well beyond the reinforcement.

Patch repairs consist of the following activities that are briefly described below:-

• Removal of cracked and delaminated concrete to fully expose the corroded reinforcement

• Cleaning of corroded reinforcement and the application of a protective coating to the steel surface (e.g. anti-corrosion epoxy coating or zinc rich primer coat).

• Application of repair mortar or micro-concrete to replace the damaged concrete.

• Possible coating or sealant applied to the entire concrete surface to reduce moisture levels in the concrete.

b) Coating systems

A variety of coating and penetrant systems are available that are claimed to be beneficial in repairs of concrete structures. Barrier systems attempt to seal the surface thereby stifling corrosion by restricting oxygen flow to the cathode. In large concrete structures, corrosion control is theoretically unlikely due to the presence of oxygen already in the system. In practice barrier systems are generally ineffective due to the presence of defects in the new coating during application and further damage during service. Such an approach is more likely to promote the formation of differential aeration cells further exacerbating the potential for corrosion. The application of a hydrophobic coating (sometimes referred to as penetrate pore-liners) may be used to reduce the moisture content of concrete and thereby electrolytic ally stifle the corrosion reaction. The drying action works on the principle that surface capillaries become lined with a hydrophobic coating that repels water molecules during wetting but allows water vapour movement out of the concrete, to facilitate drying. Hydrophobic coatings using silanes and siloxanes are gen - erally most effective on uncontaminated concrete, free from cracks and surface defects. The feasibility of such an approach is questionable for marine structures where high ambient humidity, capillary suction effects and presence of high salt concentrations all interfere with drying. The long-term effectiveness of hydrophobic systems applied to new construction is not known but local studies suggest reasonable performance over 10-15 years service. The storms river bridge was coated with a saline system in 1985 and concrete cores were extracted from several parts of the structure in 1996 for analysis [16]. The effect of the hydrophobic coating on

Absorption was determined by sorptivity testing at increasing depth increments into the concrete. Sorptivity results are shown in Fig. (3.7) for arch and column concrete. The sharp increase in sorptivity at depths between 0.5 and 3 mm may be ascribed to the presence of the saline in the concrete near-surface zone.

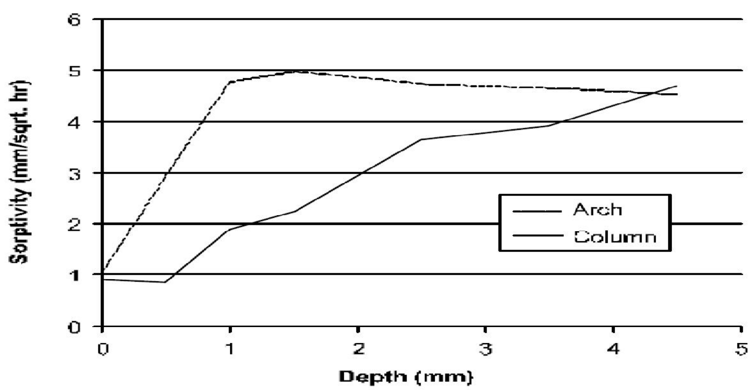

Fig. [2.14]: Sorptivity results from bridge cores.

c) Migrating corrosion inhibitors

A corrosion inhibitor is defined as a chemical substance that reduces the corrosion of metals without a reduction in the concentration of corrosive agents. Corrosion inhibitors work by reducing the rate of the anodic and/or cathodic reactions thereby suppressing the overall corrosion rate. The effectiveness of migrating corrosion inhibitors is generally controlled by environmental, material and structural factors, as shown in Table(2.1)[7].

Table (2.1): *Likely performance of migrating corrosion inhibitors in concrete [7].*

Likely inhibition	Corrosive conditions	Concrete conditions	Severity of corrosion
Good	Mildly corrosive, low chlorides or carbonation	Dense concrete with good cover depths (> 50 mm)	Limited corrosion with minor pitting of steel
Moderate	Moderate levels of chloride at rebar (i.e. <1%)	Moderate quality concrete, some cracking	Moderate corrosion with some pitting
Poor	High chloride levels at rebar (i.e. > 1%)	Cracked, damaged concrete, low cover to rebar	Entrenched corrosion with deep pitting

Migrating corrosion inhibitors are generally organic-based materials that move through unsaturated concrete by vapour diffusion. Organic corrosion inhibitors such as amino-alcohols are believed to suppress corrosion by primarily being adsorbed onto the steel surface thereby displacing corrosive ions such as chlorides. The adsorbed organic layer inhibits corrosion by interfering with anodic dissolution of iron while simultaneously disrupting the reduction of oxygen at the cathode.

When assessing the suitability of repairs with migrating corrosion inhibitors, two important issues must first be considered:

• The likely penetration of the material into the concrete needs to be determined.

• The severity of the corrosive environment at the reinforcement must be quantified.

Migrating corrosion inhibitors are designed to move fairly rapidly through partially saturated concretes that allow vapour diffusion. Penetration has however been found to be poor in near-saturated concretes typically found in partially submerged marine structures. This poor penetration performance may be ascribed to high moisture and salt levels that prevent significant vapour diffusion through the concrete. It is critical therefore that satisfactory penetration of corrosion inhibitors is checked before undertaking full-scale repairs.

The performance of migrating corrosion inhibitors in controlling chloride induced corrosion is largely dependent on chloride levels at the reinforcement. Work done by Rylands indicates that effective inhibition is not possible at chloride levels above 1.0% at the reinforcement [18]. This can be seen in Fig. (3.8) where ribbed steel bars embedded at 25 mm in a grade 40 Portland cement concrete were subjected to wetting and drying cycles with a salt solution for a period of 18 months.

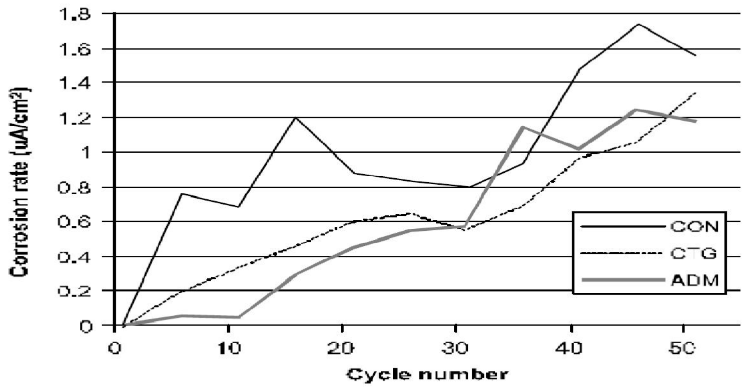

Fig. [2.15]: Corrosion rate measurements with time for grade 40 concrete.

Concrete blocks were either controls (CON) or contained organic corrosion inhibitor, either admixed during casting (ADM) or coated after 30 cycles (CTG). The chloride content at the level of the reinforcement was approaching 2% at the time of application of the migrating corrosion inhibitor and resulted in poor inhibition. Better inhibition is possible if treatment is done earlier when chloride contents are lower.

The effectiveness of migrating corrosion inhibitors appears to be enhanced when used in combination with hydrophobic coatings to reduce moisture levels in concrete. This has been noted in both laboratory trials and field monitoring of repairs. Such an approach has also been found to be effective in the repair of carbonation-induced corrosion damage.

2.7 Effect of Cracking on the Life or Durability of Structure

A good understanding of cracks in concrete will help us avoid failures of concrete on one hand and avoidable worries and expenditure on repairs on the other hand. Cracks in concrete are rarely symptoms of disease by itself. Cracks in concrete are more prevalent than imagination. But not all the cracks are dangerous. Cracks could lead to any of the following effects [10]:

- Reduce loading capacity of structure,
- Progressive failure (Cracks propagate at smaller stress than that required to initiate it),
- Loss of appearance,
- Leakages (effects serviceability),
- Apprehension of failure in mind (Psychological),
- All above reasons reduce durability of concrete. Some typical cases of cracks.

2.8 Design & Detailing For Durability

2.8.1: General

Design Engineers consider design of concrete structures to mean assessing:

(i) The size and strength of structural components and concrete strength grade to meet safety and serviceability limits.

(ii) The amount, size and distribution of reinforcements for strength and control cracks to an acceptable size.

It is anticipated that with good site control and good workmanship, the structure and its components shall last indefinitely. They have forgotten to take into account the environmental loads while designing the structures (Structures are designed adequately for DL & LL, and sometimes for erection loads). Environmental factors affect durability of structures. Hundreds of bridges and structures are collapsing or showing signs of deterioration with corroding reinforcement – all within 25 years of construction. It is, therefore, necessary for the designer to develop a feel for the problem and design the structures to satisfy safety, serviceability and durability requirements (structural and non-structural loads caused by environment) [3].

2.8.2 Environmental factors:

Critical environmental factors which affect Concrete are carbon dioxide, chlorides, water and temperature as shown in Fig. (2.16)[4].

2.8.3 Design for carbonation:

(1) The rate of carbonation depends upon the integration of concrete of the cover zone.

The penetration rate of carbonation in a good concrete structure remote from the seacoast and not subjected to de-icing salt is given by:

$$D = k\, t^{0.5} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \qquad (2.2)$$

k = 1 for M-35 & above (w/c= 0.40)

k = 2 for M-25 (w/c = 0.45)

k = 5 for M-15

D = Depth of carbonation in mm

k = Carbonation coefficient in mm/year depending upon quality of concrete

t = time of exposure in years.

M = Compressive strength of concrete

It is seen that by doubling the cover, quadruples the design life. Similarly with good quality concrete (M-35 concrete with w/c ratio 0.4) five times design life can be achieved as compared to poor quality concrete (M-15 concrete) [4].

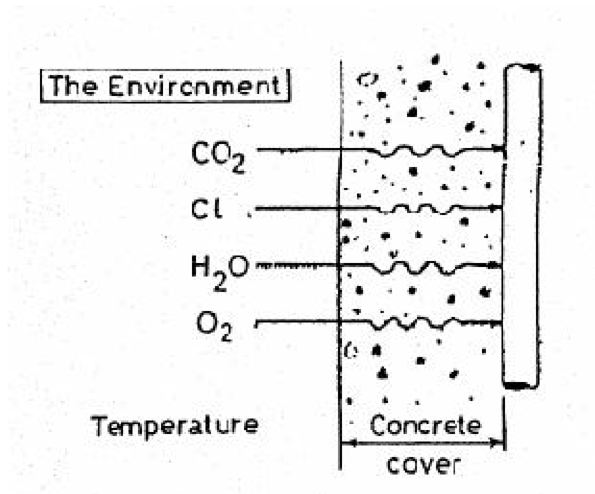

Fig. [2.16]: Critical environment factors for concrete.

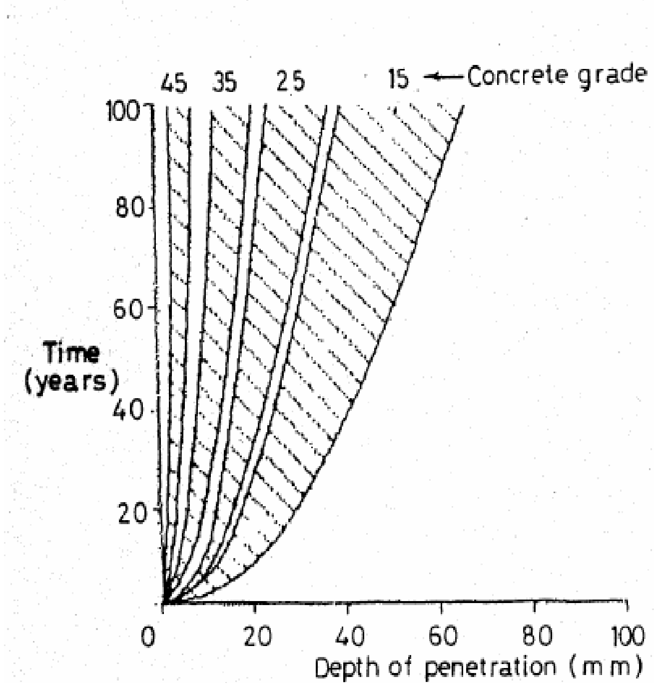

Fig. [2.17]: Carbonation penetration curve.

(2) The graph shown in Fig. (2.17) can be used to select the cover and grade to meet the required design life [3].

Example: For 100 years life, if the mix is M-35 or above the minimum cover requirement is 20 mm. i.e. if the mix is M-15 the cover requirement shall be between 40 and 60 (say 50 mm).

(3) From the graph in Fig. (2.17) it may be concluded that it is always better to go for richer mixes from durability angle even if it is not required from Strength criteria. In the interior of the buildings the rate of carbonation can be high and must be considered while designing. Carbonation effect can be taken care by ensuring adequate cover of good quality concrete. This is necessary for all the sites. It also presupposes that the integrity of cover concrete (Concrete in cover portion) i.e. both quality and extent, is assured. Thick cover is of no avail if the concrete is highly penetrable [3].

2.8.4 Design for chlorides induced corrosion:

For the marine structures, including buildings within 1 km of coast line, and for bridges exposed to deicing salt, the penetration of chloride is calculated from the following equation: [4].

(I) K.C Clear's Life Model:

$$\text{Life of corrosion on set (In years)} = \frac{129\times(cover^{-1.22})}{w/c \times CL^{-0.42}} \quad \ldots\ldots \quad (2.3)$$

Where;

Cover is in inches and w/c ratio and CL are in %.

(II) The Chloride penetration curves in Fig. (2.18) give a very simple method for design. Example: For 100 years. Design life and 60 mm cover, the minimum conc. Mix shall be M-50 or rich.

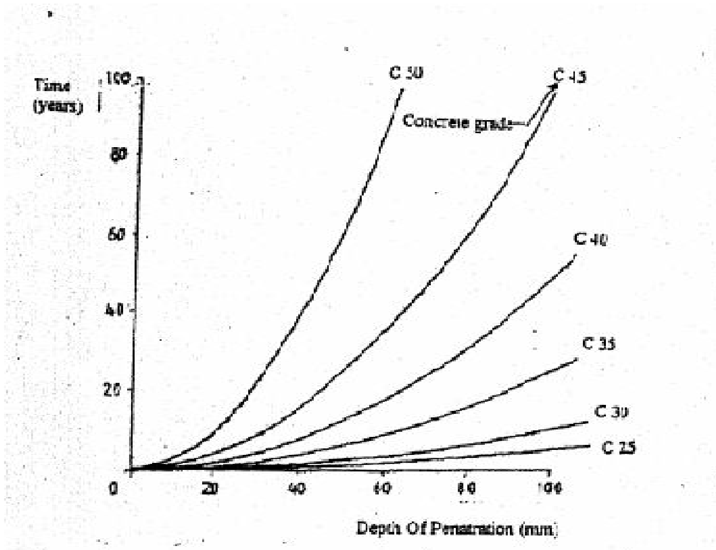

Fig. [2.18]: Chloride penetration curve.

2.8.5 Design for sulphate exposure:

This is dealt in detail under suitability of various types of cement [6].

2.8.6 Design for durability:

As durability depends upon permeability. It is important to control parameters affecting permeability. Therefore, it is necessary to lay down permissible limits for parameters like minimum cementations material content, maximum water cement ratio, maximum crack width and extent of cover to reinforcement etc. depending upon the exposure condition. Following values for this parameter are recommended [3].

I) Minimum cementations material content and maximum water cement ratio: Recommended values are given in table (2.2).

Table (2.2): *Minimum cementations material content and maximum w/c ratio.*

Exposure	PCC			RC			PSC		
	Min Grade Of mix	Min Cement Itious* Material content Kg/m3	Max w/c ratio	Min Grade Of mix	Min Cement Itious* Material content Kg/m3	Max w/c ratio	Min Grade Of mix	Min Cement Itious* Material content Kg/m3	Max w/c ratio
Mild	M-20	300	0.55	M-25	380	0.45	M-35**	400	0.40
Mod	M-25	350	0.50	M-30	400	0.40	M-35**	400	0.40
Severe	M-25	380	0.45	M-35	400	0.40	M-45	430	0.40
Very severe	M-30	400	0.45	M-40	430	0.38	M-50	440	0.35
Extreme	M-30	400	0.40	M-45	430	0.35	M-50	440	0.35

The areas covered under different exposure conditions shall be as under:

Mild**:** Concrete surfaces protected against weather or aggressive conditions.

Moderate**:** Concrete surfaces sheltered from severe rain or freezing while wet, concrete continuously under water.

Severe**:** Concrete surface exposed to severe rain, alternate wetting and drying or occasional freezing or severe condensation. Concrete exposed to aggressive sub soil / ground water or coastal environment.

Very severe**:** Concrete surface exposed to sea water spray, corrosive fumes, severe freezing.

Extreme**:** Concrete surface exposed to abrasive action. Surface of members in tidal zone.

* Min. cementations material content is for 20mm Max. Size aggregate (MSA)

Add extra cementations material

For 10 mm MSA = + 20 Kg/m^3

For 40 mm MSA = - 10 Kg/m^3

**Minimum grade of concrete mix should be M-40 for pretension PSC bridges.

Note: The maximum content of cementations material should be 500 Kg/m^3

ii) Minimum clear cover: Recommended values are given in table (2.3): The clear cover shall mean cover from the outer most metal /steel, binding wire or its end.

Table (2.3): *Minimum clear cover (mm).*

SR NO	Structures Rc / Psc	Extreme Environment	Very severe environment	severe environment	Mild and moderate environment
1	Slabs	50	50	25	25
2	Beams	60	50	40	35
3	columns	75	75	50	50
4	Wells, piles and footing	75	75	75	50
5	Psc girders	50	50	50	50
6	Psc girder of HTS cables	75	75	75	50

* While designing, it should be ensured that cover does not exceed 2.5 times dia. of reinforcing bar. If cover is more, chicken mesh may be provided in cover concrete to keep the concrete in position.

iii) Flexural Crack Width:

It is necessary to control the crack width to protect steel rebar against corrosion. The crack width is controlled by reducing shrinkage, and distributing the reinforcement over the zone of maximum concrete tension uniformly, and using smaller dia bars. Cement with low heat of hydration and not too fine should be used. Recommended value of maximum crack width to be considered during design is given in table (2.4). Design of crack width may be calculated as described in this chapter.

Table (2.4): *Maximum crack width to be considered during design.*

Type of structure	Aggressive environment- severe very, severe and extreme		Non - Aggressive environment - mild and moderate
	Exposed	unexposed	
bridges	0.1 mm	0.20 mm	0.20 mm
Rc beams, slabs	0.20 mm	0. 20 mm	0.30 mm

2.8.7 Admixtures:

Chloride free water-reducing super plasticizers may be used for RC & PSC works. However creep effect of plasticizers' need to be considered in design [5].

2.8.8 Temperature:

The concreting shall be done only when the temperature is between 5^0C & 32^0C. Preferably temperature variation of concrete during concreting should be within 5^0C. The special precautions to be taken shall be laid down by the designer [5].

2.8.9 Curing:

All properties of concrete improve with extended wet curing. This is particularly so for permeability, which strongly affects durability and service life. A minimum wet curing for 7 days by bonding water/continuous spraying; followed by 3 weeks curing by curing compounds is recommended [4].

2.8.10 Detailing**:**

Normally the aspect of congestion of reinforcement at the junction of beams & column is neglected by design engineer. The drawings should be prepared by showing the full size of bars, laps, bends, distribution bars, spacers, cover blocks etc. It should be possible to place & compact concrete by a 50 mm dia. needle vibrator. The corners & sharp edges should be chamfered. Circular sections are preferred. The design detailing should be such as to ensure effective drainage of water and to avoiding standing pool or rundown of water towards the critical components. The criteria of constructibility are of paramount importance. The layout and disposition of prestressing tendons should be designed for easy placement and vibration of concrete in the space between tendon ducts. When two or more rows of ducts are used, the horizontal space between the ducts should be vertically in line to facilitate proper flow of concrete as shown in Fig. (2.19)[3].

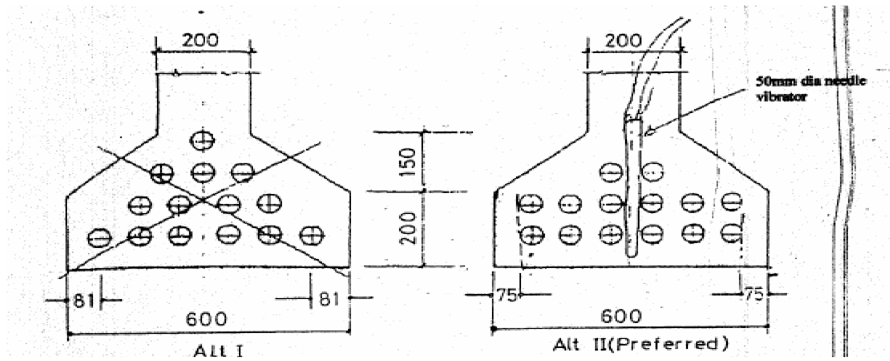

Fig. [2.19]: lay out of cables.

2.9 Design Crack Width Calculation

The design crack width may be calculated by the following formula: [6].

$$\text{Design surface crack width} = \frac{3a_{cr}\,\varepsilon_m}{1+2\left[\frac{a_{cr}-c_{min}}{h-x}\right]} \quad \ldots\ldots\ldots (2.4)$$

Where ;

c_{min} - is the minimum cover to the tension steel.

h - is the overall depth of the member.

x - is the depth of the neutral axis found from analysis to determine ε_m (see below)

a_{cr} - is the distance from the point considered to surface of the nearest longitudinal bar.

ε_m - is the average strain at the level where cracking is being considered, calculated allowing for the stiffening effect of the concrete in the tension zone, and is obtained from equation given below:

$$\varepsilon_m = \varepsilon_1 - [\frac{1.2b_t\, h(`a' - x)}{A_s(h-x)f_y} \times 10^{-3}]$$ where f_y in N/ mm^2 and

$$\varepsilon_m = \varepsilon_1 - [\frac{1.2b_t\, h(a'-x)}{A_s(h-x)f_y} \times 10^{-2}]$$ where f_y in kg/cm^2

……………. (2.5)

Where;

$$\varepsilon_1 = \frac{a'-x f_s}{d-x\ E_s}$$ …………… (2.6)

d ≡ Effective depth of tension reinforcement

f_s ≡ Actual stress in steel

E_s ≡ Modulus of elasticity of steel

ε_1 ≡ is the strain at the level considered, calculated ignoring the Stiffening effect of the concrete in the tension zone.

b_t ≡ is the width of the section of the centroid of tension steel.

a' ≡ is the distance from the compression face to the point at which crack width is being calculated, and

A_s ≡ is the area of tension reinforcement

f_y ≡ is the yield strength of steel in $N/\ mm^2$ (kg/cm^2)

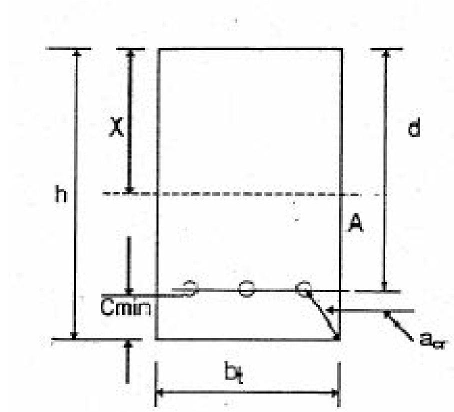

Fig. [2.20]: A negative value for ε_m indicates that the section is uncracked.

2.10 Previous studies

- In an experimental investigation carried out by Zhang (2005), the initiation and propagation phases of steel corrosion in a chloride environment were studied. The experimental measurements indicated that pitting corrosion cracks precede uniform corrosion cracks. At the crack initiation stage and the first stage of crack propagation, localized corrosion due to chloride ingression was the predominant corrosion pattern, and the pitting corrosion was the main factor that influenced the cracking process. With the propagation of corrosion cracks, uniform corrosion rapidly developed and gradually became predominant in the second stage of crack propagation [8].

- Several studies have investigated the effects of uniform corrosion on the mechanical properties of reinforcement. It has been shown that the yield and ultimate strength ratio and the elastic modulus of steel reinforcement are not significantly affected by corrosion; consequently, the corresponding values for uncorroded reinforcement are still reliable for corroded reinforcement, see Du *et al.* (2005a and

2005b). The level of reinforcement corrosion does not influence the tensile strength of reinforcement, calculated according to the actual area of cross-section; see Almusallam (2001), Cairns *et al.* (2005) and Du *et al.* (2005b). However, the ultimate strain is significantly reduced by uniform corrosion, as shown in Fig. (2.21). Although measured reductions in the ultimate strain and elongation of smaller diameter reinforcements were generally greater than those of larger bar diameter, the observed differences were not more than 5%. Hence, the reduction of the ductility of corroded reinforcement is primarily a function of the amount of corrosion, rather than the bar type and diameter, see Du *et al.* (2005a). In a reinforcement bar affected by pitting corrosion, the notch effect induces large and localized strain in the bar. Since the part of the bar affected by pitting corrosion is short, approximately twice the bar diameter, the average strain of the whole bar is less than the local strain at the pit, see Stewart and Al-Harthy (2008). Hence, the bar fails at an average strain lower than the ultimate strain of the uncorroded bar, and the ductility of the entire bar is impaired, see Coronelli and Gambarova (2004) and Du *et al.* (2005a). Very brittle behaviour is expected when 50% of the cross-section of the reinforcement is locally corroded, Palsson and Mirza (2002). The ultimate strain of locally corroded reinforcement is reduced much more significantly than the yield and ultimate strengths calculated according to the original bar area, see Darmawan and Stewart (2007) [13].

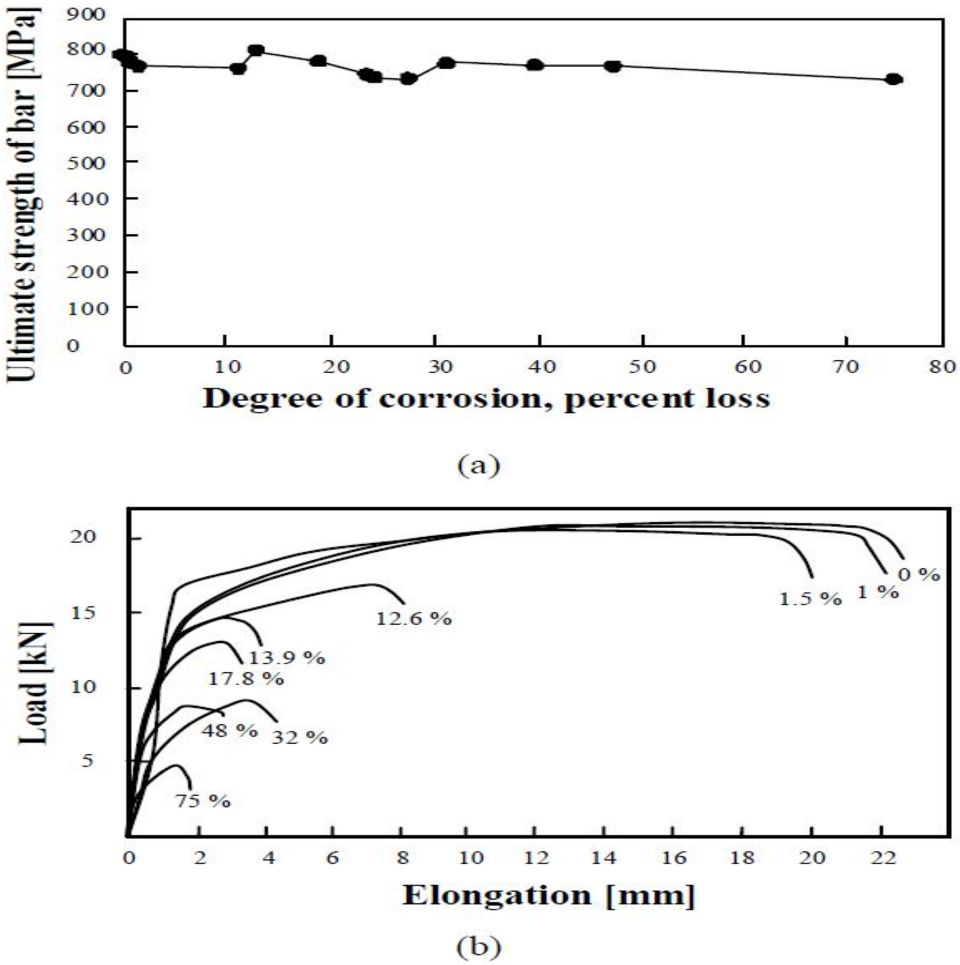

Fig. [2.21]:

*(**a**) Variation of tensile strength, based on the actual bar diameter, for selected degrees of uniform corrosion of a 6 mm diameter steel bar, from Almusallam (2001) ;(**b**) load – elongation curves for a 6 mm diameter steel bar for some degrees of uniform corrosion, from Almusallam (2001)*.

- The bond strength of corroded bars has been experimentally studied by several researchers; see Al-Sulaimani *et al.* (1990), Cabrera and Ghoddoussi (1992), Auyeung *et al.* (2000) and Rodriguez *et al.* (1995a). For a review of pull-out tests on corroded steel bars, see Sather (2009a). The main parameters affecting the relative bond strength of corroded steel bars were found to be corrosion penetration, bar position, confinement (concrete cover and transverse reinforcement) and the impressed current density, see Sather *et al.* (2007). In the overview presented by Lundgren (2007), the existence of transverse reinforcement and confinement due to the concrete and boundaries were considered to have the greatest influence on the bond. Various models exist to account for the bond behavior of corroded reinforcement, see Rodriguez *et al.* (1994), Bhargava *et al.* (2007) and Chernin *et al.* (2010). The friction between the reinforcement and the concrete is also influenced by corrosion. In the model by Lundgren (2005b), the volume increase of the corrosion product around a corroded bar has been modeled. Furthermore, it was assumed that corrosion affects the friction between the steel and the concrete.