CHAPTER – 85

# DESIGN FOR DURABILITY

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## 85 DESIGN FOR DURABILITY

Durability is the ability of the structure or structural element to assure no degradation in the required performance over a specified period of time resulting, materials deterioration that is harmful in the relevant environment.

### 85.1 General Criteria for design for durability

A durable structure shall be designed in such a way that it meets the requirements of serviceability, strength and stability throughout its design working life.
The level of deterioration over its design working life does not impair the performance of the structure without significant loss of utility or excessive anticipated level of maintenance.

Note: The degree of any deterioration may be estimated on the basis of calculations, experimental investigation, experience from earlier constructions, or a combination of these considerations.

The required protection of the structure shall be established by considering its intended use, design working life, maintenance program and actions.
The possible significance of direct and indirect actions, environmental exposure conditions and consequential effects shall be considered at the design stage.
Corrosion protection of steel reinforcement depends on type of SCM's used, density, quality and thickness of concrete cover and cracking. The cover density and quality are achieved by controlling the maximum water/cement ratio and minimum cement content and may be related to a minimum strength class of concrete.
Where metal fastenings are inspectable and replaceable, they may be used with protective coatings in exposed situations. Otherwise, they should be of corrosion resistant material.
Further requirements to those given in this section should be considered for special situations (e.g. for structures of temporary or monumental nature, structures subjected to extreme or unusual actions etc.).

### 85.2 Factors affecting durability of concrete

The various factors influencing durability of concrete shall consider the type of deterioration in context to the environmental exposure conditions. The main parameters that influences the durability of concrete are type of cement and permeability of concrete Permeability governs its resistance to the ingress of deleterious /aggressive agents like water, oxygen, carbon dioxide, chloride, sulphate and other potentially deleterious substances. Permeability of concrete is generally characterized on the basis of amount of the pores and pore size distribution and mainly governed by the constituents and workmanship used in making the concrete. With normal-weight aggregates a suitably low permeability is achieved by having an adequate cement content, sufficiently low free water/cement ratio, judicious utilization of mineral admixtures and good workmanship, including mixing, placement by ensuring complete compaction of the concrete, and by adequate curing.

The factors influencing durability include,

- The environment;
- The cover to embedded steel;
- The type and quality of constituent materials;
- The cement content and water/cement ratio of the concrete;
- Workmanship, to obtain full compaction and efficient curing; and
- The shape and size of the member.
- Design and detailing
- Maintenance

Complexity in structural form, as well as execution and use generally increase the sensitivity of the structure to an aggressive environment. The shape or design details of exposed structures should be such as to promote good drainage of water and to avoid standing pools and rundown of water. Care should also be taken to minimize any cracks that may collect or transmit water. Adequate curing is essential to avoid the harmful effects of early loss of moisture. Member profiles and their intersections with other members shall be designed and detailed in a way to ensure easy flow of concrete and proper compaction during concreting.

Concrete is more vulnerable to deterioration due to chemical or climatic attack when it is in thin sections, in sections under hydrostatic pressure from one side only, in partially immersed sections and at corners and edges of elements. Cyclic wetting and drying effects strongly accelerate the rate at which deleterious substances enter the concrete. The life of the structure can be lengthened by providing extra cover to steel, by chamfering the corners or by using circular cross sections or by using surface coatings which prevent or reduce the ingress of water, carbon dioxide or aggressive chemicals.

The degree of exposure anticipated for the concrete during its service life together with other relevant factors need to be considered. Maintenance plays an important role in sustained durability of concrete. The concrete mix to provide adequate durability under the conditions should be chosen taking account of the accuracy of current testing regimes for control and compliance as described in this section.

### 85.3 Concrete deterioration mechanism

Deterioration mechanisms are either chemical or physical in nature and may be originated from within the concrete, or may be result of the external environmental exposure conditions. Transport mechanism plays a significant role while deciding the rate and extent of deterioration. Some of the potential deterioration mechanism causing durability problems are classified below:

- Corrosion of reinforcement due to
    - Permeation of atmospheric carbon dioxide in the concrete (carbonation induced corrosion)
    - Presence of significant levels of chloride ion in concrete (chloride induced corrosion)
- Physical deterioration processes; Abrasion, freezing and thawing, weathering, shrinkage etc.
- Chemical deterioration processes; sulphate attack, alkali aggregate reaction etc.

Note 1: Physical damage in concrete structures can also arise due to overloading and accidental actions such as, fire, impacts and accidents; and also, earthquake/seismic events. Such physical damage is not considered to be a service life design or a durability issue, accordingly it is not addressed here.

Note 2: Chemical deterioration processes can arise from either external chemical attack or internal deleterious chemical reactions.

Note 3: In most of the situations one type of deterioration mechanism dominate. However, in some circumstances several deterioration mechanisms can act in combination. This is referred to as synergetic deterioration. In these circumstances' specialist literature may be referred and a risk-based approach might be adopted to guide these considerations.

### 85.4 Exposure Classes related to environmental factors

**General**

Exposure Environments are defined on the basis of the chemical and physical conditions to which a concrete structure shall be exposed or been exposed in additions to the mechanical actions. Different types of environmental conditions are classified by exposure classes as given in table 1 to 5 for different concrete deterioration mechanisms.

The type of environment is defined by the combination of:

General exposure classes related to the potential deterioration mechanism of corrosion of reinforcement as per ……..

The specific exposure classes relating to the other deterioration processes appropriate to each case as per ……… and ………

**Table 1- Exposure classes for concrete deterioration due to corrosion induced by carbonation (Where concrete containing reinforcement or other embedded metal is exposed to air and moisture)**

| Class designation | Description of the environment | Informative |
|---|---|---|
| C1 | Hot- dry | Mean monthly maximum temperature above 30 ºC with mean monthly relative humidity below 55% |
| C2 | Warm humid and Temperate | The temperatures usually vary between 25–35 ºC in summers with mean monthly relative humidity below 55%; while in winters, temperatures vary between 20–30 ºC with monthly relative humidity above 75% |
| C3 | Composite | For the whole year; it has a particular season for more than six months and may experience other seasons for the remaining period. |
| C4 | Cold/ mountainous | Mean monthly maximum temperature less than 25 ºC for all values of mean monthly relative humidity |
| *Note: Details of the climatic zone and precipitation zones are provided in Annex A* | | |

**Table 2- Exposure classes for concrete deterioration due to Corrosion induced by chlorides other than from sea water (Where concrete containing reinforcement or other embedded metal is subject to contact with water containing chlorides, including deicing salts from sources other than sea water)**

| Class designation | Description of the environment | Informative |
|---|---|---|
| CH1 | Wet, rarely dry | Swimming pools, Concrete exposed to waters containing chlorides |
| CH2 | Cyclic wet and dry | Parts of bridges exposed to spray containing chlorides. |

**Table 3- Exposure classes for concrete deterioration due to corrosion induced by chlorides from sea water (Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water)**

| Class designation | Description of the environment | Informative | |
|---|---|---|---|
| SW1 | Exposed to airborne salt but not in direct | a | Structures located up to 1km from the coast |
| | | b | Structures located beyond 1 km and up to 10 km from the coast |

| | | | |
|---|---|---|---|
| | contact with sea water | c | Structure located beyond 10 km and up to 50 km from the coast * |
| SW2 | Permanently submerged | Parts of marine structures | |
| SW3 | Tidal, splash and spray zones | Parts of marine structures | |

*to be checked for carbonation also.

**Table 4- Exposure classes for concrete deterioration due to sulphate attack**

| Class designation | Description of environment | Informative |
|---|---|---|
| S0 | No risk | $SO_3$<0.2% (soil), $SO_3$ <300 ppm (water) |
| S1 | Moderate | $SO_3$ =0.2% to 1.0% (soil), $SO_3$ =300 to 2500 ppm (water) |
| S2 | Severe | $SO_3$>1% (soil), $SO_3$>2500 ppm (water) |
| S3 | Very Severe | Magnesium sulphate $SO_3$>1% (soil), $SO_3$>2500 ppm (water) |

**Table 5- Exposure classes for concrete deterioration due to Freeze/thaw attack (Where concrete is exposed to significant attack from freeze-thaw cycles whilst wet)**

| **Class designation** | **Description of the environment** | **Informative** |
|---|---|---|
| F1 | Moderate | Concrete exposed to freezing and thawing cycles and occasional exposure to moisture |
| F2 | Severe | Concrete exposed to freezing and thawing cycles and in continuous contact with moisture |
| F3 | Very severe | Concrete exposed to freezing and thawing cycles and in continuous contact with moisture and exposed to deicing chemicals |

### 85.5 **Design for durability**

All structures will be designed for durability for a given design service life by selecting appropriate concrete grade, type of cement, SCM's, mix proportions and adequate cover to the reinforcement for the given exposure classes.

### 85.5..1 **Design service life**

The period for which the structure or structural element is to be used for its intended purpose with anticipated maintenance but major substantial repair being necessary.

### 85.5..1.1 Design service life table for different types of structures

Design service life $t_{sl}$ for different types of structures are defined in below table 6.

**Table 6- Indicative values for the design service life $t_{SL}$**

| **Design Service life $t_{SL}$ (years)** | **Examples** |
|---|---|
| 10 | Temporary structures (structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary) |
| 10 – 25 | Replaceable structural parts, e.g. gantry girders, bearings |
| 15 – 30 | Agricultural and similar structures |
| 50 | Building structures and other common structures |
| 50-75 | High rise structures, critical/important buildings such as hospitals |
| 100 or more | Monumental buildings structures, bridges, and other civil engineering structures |

These values or any other decided values may be taken up by the owner/design engineers for designing different type of structures on the basis of their utility as required design service life.

### 85.5..2 limit state of durability

Limit state of durability shall be decided such that deterioration produced by chemical and physical actions other than the loads and actions of the structural analysis which may deteriorate the characteristics of the concrete or reinforcements does not lead to an ultimate limit state, a serviceability limit state, restorability limit state or a fatigue limit state during its design life.

To check limit state of durability following condition shall be verified :

tl ≥ tsl

where:

tl: time needed for the aggressive agent to produce a significant degradation.

tsl: design value of the service life

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### 85.5..3 Carbonation induced corrosion

Carbonation is a physiochemical process involving chemical and physical changes. In concrete, the reaction between hydrated calcium compounds, primarily Ca $(OH)_2$, and atmospheric $CO_2$ causes a chemical change within the concrete that increases the risk of reinforcement corrosion. This process, known as carbonation of the concrete, causes a decrease in the alkalinity of the pore solution to below pH 9. The carbonation starts from the exposed concrete surface and penetrates slowly to the interior of the concrete.

To ensure adequate resistance to carbonation, basic requirements with regard to minimum cover to the reinforcement, concrete limiting values, e.g. maximum water/cement ratio or water/cementitious ratio, minimum grade of concrete, minimum cementitious content, minimum nominal cover and durability test requirements for 50 years design service life for normal structures is given in table 7. These values are applicable for Importance level 1. However, for other values of importance level or design service life more than 50 yrs and up to 120 year, detail design for durability is to be carried out as per ……...Limiting values given in table 7 need to be adhered to.

**Table 7 for Requirements for Normal structures subjected to carbonation for 50 years design service life for quality control level = 2 and 3 (med and high) and importance level 1**

| Class designation | Description of the environment | Type of member | Type of Cement | Nominal depth of cover (in mm) | maximum w/c or w/ccm | Minimum Grade of Concrete | minimum cementitious content (kg/m3) | Durability Test Requirement* |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Electrical Resistivity (kohm cm) at 28 days |
| C1 | Hot- dry | Internal and external members | OPC | 20 | 0.55 | M20 | 300 | > 10 ? |
| | | | PPC or OPC+ fly ash | 25 | 0.55 | M20 | 300 | > 20 ? |
| | | | PSC or OPC+ GGBS | 25 | 0.55 | M20 | 300 | > 40 ? |
| C2 | Warm humid and Temperate | For internal Members | OPC | 20 | 0.52 | M25/(M20 for cover>=25mm) | 320 for m25 /300 for m20 | >10 |
| | | | PPC or OPC+ fly ash | 25 | 0.5 | M25 | 320 | >20 |
| | | | PSC or OPC+ GGBS | 25 | 0.5 | M25 | 320 | >40 |
| | | For External Members | OPC | 25 | 0.50 | M25 | 320 | >10 |
| | | | PPC or OPC+ fly ash | 30 | 0.50 | M25 | 320 | >20 |
| | | | PSC or OPC+ GGBS | 30 | 0.50 | M25 | 320 | >40 |
| C3 | Composite | For internal Members | OPC | 20/ | 0.52 | M25 | 300 | >10 |
| | | | PPC or OPC+ fly ash | 25 | 0.50 | M25 | 320 | >20 |
| | | | PSC or OPC+ GGBS | 25 | 0.5 | M25 | 320 | >40 |
| | | For External Members | OPC | 25 | 0.50 | M25 | 320 | >10 |
| | | | PPC or OPC+ fly ash | 30 | 0.47 | M30 | 320 | > 20 |
| | | | PSC or OPC+ GGBS | 30 | 0.470 | M30 | 320 | > 40 |
| C4 | Cold/ mountainous | Internal members | OPC | 20 | 0.55 | M20 | 300 | > 10 |
| | | | PPC or OPC+ fly ash | 20 | 0.5 | M20 | 300 | >20 |
| | | | PSC or OPC+ GGBS | 20 | o.5/ | M20 | 300 | >40 |
| | | External members | OPC | 25 | 0.55 | M25 | 300 | >10 |
| | | | PPC or OPC+ fly ash | 25 | 0.50 | M25 | 300 | >20 |
| | | | PSC or OPC+ GGBS | 25 | 0.50 | M25 | 300 | >40 |
| | | | | | | | | |

Note:

Durability test required for class C2 and C3 , if grade of concrete is less than M30.

For QC level 1 and Where durability tests required but have not been conducted, then minimum grade of concrete will be increased to next level and maxim w/c ratio should be reduced by 0.05. However for important projects , durability tests are recommended for all classes.

**Commented [SB1]:** We can add a comment on changing the minimum cement content required when we use MSA other than 20 mm.

For internal members which may have easy access to water, such as members in and around toilets, kitchens, pantries and other water-handling areas, the val for external members shall be used.

For internal members of open type structures like parking garages etc exposed to moisture and temperature similar to external environment, the values given for external members will be used.

For QC tests values of durability tests obtained during mix design stage (which are likely to be stringent than values given above) will be used. Values given in table are values to be achieved at mix design stage. Acceptance criteria given in chapter…… On Quality control.

For QC purpose other durability test i.e. Oxygen permeability test as per IS 516 part ………. Can also be used, subject to conducting same test at mix design stage to fix the required values.

Values of minimum cement content given for OPC are for 43 gr OPc. For 33 gr OPC, values will be increased by 20 kg/cum and w/c ratio will be reduced by 0.02

"The minimum cement content, maximum free water cement ratio and minimum grade of concrete are individually related to explosion."

Adjustments to Minimum Cement Contents for Aggregates Other Than 20 mm Nominal Maximum Size

| Sl No. | Nominal Maximum Aggregate Size (mm ) | Adjustment to Minimum Cement Contents in Table 7 (kg/m3) |
|---|---|---|
| (1) | 10 | +40 |
| (2) ) | 20 | 0 |
| (3) | 40 | -30 |

### 85.5..3.1 limit state design for carbonation induced corrosion

Limit state for durability for carbonation induced corrosion will be the initiation of cracks due to reinforcement corrosion due to depassivation caused due to carbonation process. To ensure this limit state is not reached during the design service life, the calculated service life (t) as per procedure given below shall be less than design service life (tsl).

### 85.5..3.2 Actions and effects:

Weather/ environmental effects to be considered are:

- Atmospheric $CO_2$ levels
- Type of environment/weather

### 85.5..3.3 Characteristic value of $CO_2$ levels

Based upon geographical conditions, characteristic value of $CO_2$ ( at mean design service life), $CO_{2ck}$ is as follows

| Characteristic value of $CO_2$ levels, $CO_{2ck}$ | Description |
|---|---|
| 350 | For all areas under exposure class C4 and areas in exposure class C1 to C3 but away from urban areas, industrial areas |
| 400 | For internal members of air-conditioned buildings in exposure class C1 to C3 |
| 450 | External members of urban and Non-aggressive Industrial area in exposure class C1 to C3 |

Note: Industrial areas where concentration of $CO_2$ is high, shall be designed for the actual maximum expected value of $CO_2$ during the service life but not less than the values given above.

### 85.5..3.4 Effect of weather

Effect of weather will be considered in the assessment of service life of concrete structure. The weather coefficient, W takes into consideration meso-climatic conditions such as humidity and temperature

### 85.5..3.5 Resistance to carbonation

Resistance to carbonation of cover concrete is achieved by

Using concrete mixes that have appropriate resistance to carbonation, which is measured in terms of carbonation coefficient,.

Increasing cover to the rebar

Carbonation coefficient:

Equation to determine characteristic carbonation coefficient K1ck for different binder systems are as under

For OPC, $K1ck = 9.5(w/c \text{ or } w/ccm) - 2.75$

For PPC and PSC, $K1ck = 12.3(w/c \text{ or } w/ccm) - 1.6$

Where:

w/c= water-cement ratio;

w/ccm= water-cementitious ratio

Note

For concrete containing fly ash between 25 % to 35 %, K1ck value as proposed for PPC to be used.

For concrete containing fly ash up to 25%, for K1ck value linear interpretation to be done between OPC and PPC.

For concrete containing GGBS between up to 50%, for K1ck value linear interpretation to be done between OPC and PSC.

For concrete containing GGBS> 50 %, for K1ck value as proposed for PSC shall be used.

The values of K1m so determined are minimum values and need to be verified further at the time of mix design as per ………. below. Higher of the two values shall be adopted in durability design.

### 85.5..3.6 Characteristic values $K_{1ck}$ ( to be specified) and design values of carbonation coefficient of concrete , $K_{1d}$:

Design value of carbonation coefficient, K1d = K1ck * ϒm1

Where ϒm1 = 1.0 to 1.20 depending upon QC level and value of design service life Tsl.

| QC Level | ϒm for tsl<=75 yr | For tsl>75 yr |
|---|---|---|
| 1 (low) | 1.15 | 1.2 |
| 2 (med) | 1.07 | 1.12 |
| 3 (high) | 1.0 | 1.05 |

### 85.5..3.7 Nominal minimum cover to the reinforcement, C - To be chosen for design, in mm.

The Characteristic value of cover shall be same as chosen nominal value, as no negative tolerance is permitted on cover.

Design value of cover, Cd ( in mm)

Cd = C /ϒm2

Where ϒm2 depends upon QC level and is as under

| QC Level | ϒm |
|---|---|
| 1 (low) | 1.1 |
| 2 (med) | 1.05 |
| 3 (high) | 1.0 |

### 85.5..3.8 Verification of $K_1$

While conducting mix design, accelerated tests namely electrical resistivity test will be conducted and values of $K_{1ck}$ will be determined. For different value of $K_{1ck}$ and binder type, required electrical resistivity values are given in below table 8, 9 and 10.

**Table 8 For concrete made with OPC**

| K1 , carbonation coefficient | Electrical Resistivity ( in kohm-cm) at 28 days |
|---|---|
| 1.0 | 12 |
| 1.5 | 11 |
| 2.0 | 10 |
| 2.5 | 9 |
| 3.0 | 9 |

**Table 9 For concrete made with PPC or fly ash + OPC**

| K1 , carbonation coefficient | Electrical Resistivity ( in kohm-cm) 28 day |
|---|---|
| ≤2.5 | 30 |
| 3.0 | 26 |
| 3.5 | 24 |
| 4.0 | 22 |
| 4.5 | 20 |
| 5.0 | 18 |
| 5.5 | 17 |

**Table 10, For concrete made with PSC or GGBS + OPC**

| K1 , carbonation coefficient | Electrical Resistivity ( in kohm-cm) 28 days |
|---|---|
| ≤2.5 | 52 |
| 3.0 | 50 |
| 3.5 | 47 |
| 4.0 | 44 |
| 4.5 | 40 |
| 5.0 | 38 |
| >=5.5 | 35 |
| | |

For QC purpose other durability test i.e. Oxygen permeability test as per IS 516 part ……. Can also be used, subject to conducting same test at mix design stage to fix the required values.

For important structures, particularly for concrete structures that shall be designed for service life more than 50 year, $K_{1m}$ will be determined through accelerated carbonation test as per IS 516-part 2 section 4. This test method takes longer test duration as compared to i) above. However, test as mentioned at i) shall also be conducted for determining the values to be considered for acceptance during quality control. Procedure for determination $K_{1m}$ using IS 516 Part 2 section 4 is given in Annex B. Value of K1ck will be determined from k1m obtained from tests.

K1ck= k1m x 1.1

Minimum values of K1ck shall be as per clause ……… and above

### 85.5..3.9 Verification of performance (against carbonation )

For verification, it will be ensured that

tsl ≤ tl

The service life, tl is divided in two parts such that :

tl = ti + tp

where

Time for initiation of corrosion ( ti): It shall be taken as time for carbonation to reach reinforcement front, that is when Carbonation depth is equals nominal cover to rebar.

Propagation time ( tp) : Time from initiation of corrosion to initiation of crack. (refer cl.85.5.3.11

tsl ≤ ( ti + tp)

For t1 = tsl

Tsl = ti +tp

Or ti = tsl – tp

### 85.5..3.10 Determination of corrosion initiation period , ti :

**For determination of initiation period ti for the given design cover and material properties or for Determination of minimum design cover for required ti , following equation shall be used:**

Cd = W* K1d*(ti *ϒf *CO2ck/500)x

Where ti is corrosion initiation period in years

Cd is design value of nominal cover in mm

W is weather coefficient is given in table 11

K1d is design value of carbonation coefficient

CO2d is design value of CO2

x value depends upon type of binder, x value in case of

Concrete made with OPC =0.60
Concrete made with PPC =0.45
Concrete made with PSC =0.45

Note:

For concrete containing fly ash between 25 % to 35 %, x value as proposed for PPC to be used.

For concrete containing fly ash between 15 % to 25%, linear interpretation to be done between OPC and PPC.

For concrete containing fly ash < 15 %, x value will be 0.60.

For concrete containing GGBS between 25 % to 50%, linear interpretation to be done between OPC and PSC.

For concrete containing GGBS< 25 %, x value will be 0.60.

ϒ f , partial factors of safety on CO2ck– values of ϒf shall vary from 1.0 to 1.5 depending upon importance level selected as under:

| **Importance level** | **ϒf** |
|---|---|
| Normal | 1.0 |
| Important | 1.2 |
| Very important | 1.4 |

**Table 11 for weather coefficient**

| Class designation | Description of the environment | Type of member | Weather coefficient |
|---|---|---|---|
| C1 | Hot- dry | Internal and external members | 0.90 |
| C2 | Warm humid and Temperate | For internal Members | 1.00 |
| | | For External Members | 1.10 |
| C3 | Composite | For internal Members | 1.10 |
| | | For External Members | 1.25 |

| | | | |
|---|---|---|---|
| C4 | Cold/ mountainous | Internal | 0.9 |
| | | external members | 1.0 |

### 85.5.3.11 Determination of propagation period, tp :

| Grade ofconcrete | For concrete made with OPC | | | | | | | For concrete made with PPC/PSC or OPC+ fly ash / OPC+ GGBS | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Exposure classes | | | | | | | | | | | | | |
| | C | C2 | | C3 | | C4 | | C1 | C2 | | C3 | | C4 | |
| | | Internal member | External member | Internal member | External member | Internal member | External member | Int/ext | Internal member | External member | Internal member | External member | Internal member | External member |
| ≤ M35 | 13 | 10 | 5 | 10 | 8 | 8 | 6 | 13 | 10 | 8 | 12 | 10 | 10 | 7 |
| >M35 | 15 | 12 | 7 | 15 | 10 | 10 | 8 | 15 | 12 | 10 | 15 | 12 | 12 | 9 |

Considering initiation of corrosion crack as limit state, propagation period depends upon rate of corrosion, concrete cover and grade of concrete which is a complex phenomenon. For normal building design purposes values given in table 12 below may be used. For important structures, better assessment can be made using available models in literature. **Table 12, Propagation period ( tp) in years for nominal cover to bar 20 mm**

Commentary
For importance structures, models given in specialist literature may be referred for propagation period. The value given in table … are based on field experience and existing literature.

Note:For higher nominal cover , the values given in above table may be increased by 10 % for every increase of 10 mm in nominal cover.

For Structural members prone to ingress of moisture through seepage from roofs or pipes, propagation period should be taken as 5 years maximum.

### 85.5.4 Chloride induced Corrosion

Presence of chlorides in concrete in a quantity exceeding a certain limit, accelerates the corrosion of the steel in the concrete and results in early deterioration of the structure. The higher the chloride content, or if subsequently exposed to warm moist conditions, the greater the risk of corrosion. Such chlorides may enter the concrete from an external environment or supplied to the concrete from materials such as cement, aggregate, chemical admixtures, mixing water and any other concrete making constituents. In order to protect steel against corrosion, it is very important to control and keep the total quantity of chloride ions supplied to the concrete from harmful chemical salts. The levels of such harmful salts in concrete coming from concrete materials, that is, cement, aggregates, water and admixtures, as well as by diffusion from the environment should be within the range that does not cause or accelerates steel corrosion.

Even where structures are not exposed to chloride and is exposed to air and moisture, the total quantity of chloride ions be controlled in terms of the total acid soluble chloride ion content of concrete during mixing. The total acid soluble chloride ion content of concrete during mixing is the total of the quantities of chloride ions that are thought to be supplied from the materials used to the concrete calculated on the basis of mix proportions. Wherever possible, the total chloride content of the concrete should be determined.

For determination of total chloride content in concrete, total acid soluble chloride ion content for cementitious material and water-soluble chloride for all other constituents shall be considered. The chloride content so determined in concrete shall be within limits specified in table 13.The chlorides more than these prescribed limits can accelerate corrosion rate caused by carbonation or can initiate corrosion even in absence of carbonation, if chloride content is very high.

**Table 13 Limits of Chloride Content of Concrete**
(*Clause* 7.3.5)

| **Sl No.** | **Type or Use of Concrete** | **Maximum Total Acid Soluble Chloride Content Expressed as kg/m3 of Concrete** |
|---|---|---|
| **(1)** | **(2)** | **(3)** |
| | **Concrete containing metal and steam cured at elevated temperature and prestressed concrete** | **0.4** |
| | **Reinforced concrete or plain concrete containing embedded metal** | **0.6 (or 0.15 % by mass of cement, whichever is lower)** |
| | **Concrete not containing embedded metal or any material requiring protection from chloride** | **3.0** |

Note: For hardened concrete, whenever required total acid soluble chloride in Reinforced concrete shall be determined as per IS 14959 and the same shall not exceed 0.2 % by mass of cement. Where cement content is not known, the total acid soluble chloride shall not exceed the value given in table above.

#### 85.5.4.1 Concrete in Sea water

Seawater contains dissolved salts that are potentially aggressive to concrete. The most significant aggressive chemicals present in seawater include chloride, sulphates and magnesium. The mitigation of seawater exposure requires the use of mixture proportions that minimize the permeability. Concrete in sea-water or exposed directly along the sea-coast shall be at least M 20 Grade in the case of plain concrete and M 35 in case of reinforced concrete. The use of silica fume, slag or pozzolana cement is advantageous under such conditions. Lower permeability of concrete made with appropriate amounts of micro silica, slag or pozzolana cement and with their combination can be achieved.

Special attention shall be given to the design of the mix to obtain the densest possible concrete; slag, broken brick, soft limestone, soft sandstone, or other porous or weak aggregates shall not be used.

As far as possible, preference shall be given to precast members unreinforced, well-cured and hardened, without sharp corners, and having trowel-smooth finished surfaces free from crazing, cracks or other defects; plastering should be avoided.

No construction joints shall be allowed within 600 mm below low water-level or within 600 mm of the upper and lower planes of wave action. Where unusually severe conditions or abrasion are anticipated, such parts of the work shall be protected by bituminous or silico-fluoride coatings or stone facing bedded with bitumen.

In reinforced concrete structures, care shall be taken to protect the reinforcement from exposure to saline atmosphere during storage, fabrication and use. It may be achieved by treating the surface of reinforcement with cement wash or by suitable methods

To ensure adequate resistance to carbonation, basic requirements with regard to minimum cover to the reinforcement, concrete limiting values, e.g. maximum water/cement ratio or maximum water/cementitious ratio, minimum grade of concrete, minimum cementitious content, minimum nominal cover and durability test requirements for 50 years design service life for normal structures is given in table 14 . However, for other values of design service life up to 120 years, proper design for durability can be carried out as per………..

**Table14 for Requirements for Normal structures subjected to chloride induced corrosion**
**for 50 years design service life for** ϒr =1.05 and ϒm =1.05

| Class designation | Description of the environment | | Type of Cement | Nominal depth of cover (in mm) | maximum w/c or $w/c_{cm}$ | Minimum Grade of Concrete | minimum cementitious content $(kg/m^3)$ | Durability Requirement | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Electrical Resistivity (kohm cm) at 28 days | Charged passed ( in coulombs) |
| SW1 | a | Near shoreline ( up to 0.1 km ) | PPC or OPC+ fly ash | 50* | 0.35 | M40 | 400 | ≥28 | <600 |
| | | | PSC or OPC+ GGBS | 45 | 0.40 | M35 | 380 | ≥~~45~~ 48 | <500 |
| | | 0.1 km to 0.5 km | PPC or OPC+ fly ash | 45* | 0.40 | M35 | 380 | ≥24 | 601-800 |
| | | | PSC or OPC+ GGBS | 40 | 0.42 | M35 | 360 | ≥~~45~~ 46 | <500 |
| | | 0.5 km to 1.0 km | OPC | 45 | 0.42 | M35 | 360 | ≥11 | 2001-2500 |
| | | | PPC or OPC+ fly ash | 30 | 0.5 | M25 | 340 | ≥21 | 601-800 |
| | | | PSC or OPC+ GGBS | 30 | 0.50 | M25 | 340 | ≥40 | 501-700 |
| | b | 1.0 km to 10 km | OPC | 35 | 0.50 | M25 | 360 | ≥10 | 2001-2500 |
| | | | PPC or OPC+ fly ash | 30 | 0.50 | M25 | 340 | ≥21 | 601-800 |
| | | | PSC or OPC+ GGBS | 30 | 0.50 | M25 | 340 | ≥40 | 501-700 |
| | c | 10 km to 50 km | OPC | 30 | 0.50 | M25 | 320 | ≥10 | 2501-3000 |
| | | | PPC or OPC+ fly ash | 25 | 0.50 | M25 | 320 | ≥21 | 801-1000 |
| | | | PSC or OPC+ GGBS | 25 | 0.50 | M25 | 320 | ≥40 | 501-700 |
| SW2, CH1 | | | PPC or OPC+ fly ash | 50* | 0.35 | M40 | 400 | ≥28 | 601-800 |
| | | | PSC or OPC+ GGBS | 45 | 0.40 | M35 | 380 | ≥48 | <500 |
| SW3, CH2 | | | PPC or OPC+ fly ash | 55* | 0.35 | M40 | 400 | ≥28 | <600 |
| | | | PSC or OPC+ GGBS | 50 | 0.41 | M40 | 400 | ≥48 | <500 |

* Note

i. In these specific 4 cases, cover can be reduced by 5mm by reducing w/c ratio by .05 and increasing min cementitious content by 40 kg content. For other cases, cover mentioned is minimum cover and can not be reduced further.
ii. In exposure class, SW1 b and c, corrosion due to carbonation may also be a critical. Therefore, while designing RCC structures in this exposure class, requirements for corrosion due to carbonation shall also be checked.
iii. For exposure classes where OPC is not mentioned, OPC is not to be used . (Because It is difficult to obtain required resistance to chloride ingress with reasonable concrete cover and w/c.)

iv. Composite cement conforming to IS:……… can be used in exposure classes SW1a and SW2 CH1 in lieu of PPC. Resistivity values with use of composite cement, however need to be established for the specific cement proposed to be used.

v. Durability test required shall be done for expsoure class SW1 a , SW2 , SW3 and CH2

vi. For classes SW1 a, SW2 ,SW3 and CH2 , if durability tests have not been done, then minimum grade of concrete will be increased to next level and maximum w/c or $w/c_{cm}$ will be reduced by 0.05. However for important projects, durability tests are recommended for all classes

**85.5.4.2 Limit state of chloride induced corrosion (**Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water)

Limit state for durability for deterioration due to chlorides will be when a threshold value of chlorides induced corrosion is reached at the surface of steel reinforcement.

### 85.5.4.3 **Actions and Effects**

#### 85.5.4.3.1 Surface chloride content Cs

Design value of surface chloride content, Csd will be taken as under for different exposure classes

| Sr No | Exposure Class | | | Characteristics surface chloride content Csck (by weight of concrete , kg/m3) for service life design of concrete structure |
|---|---|---|---|---|
| 1. | SW1 | a | Near shoreline (up to 0.1 km) | 9 |
| | | | 0.1 km to 0.50 km | 5.0 |
| | | | 0.50 km to 1 km | 2.0 |
| | | | 1 km to 10 km | 1.75 |
| | | b | 10 Km to 50 km | 1 |
| 2. | SW2, CH1 | | | 9 |
| 3. | SW3, CH2 | | | 13 |

Design value of surface chloride content, Csd = Csck* ϒr

Here, ϒr =1.05

ϒr , partial factor of safety ( for action /effects) .

Note :Higher value may be adopted for importance structures

### 85.5.4.4 **Threshold Chloride content, Cth**:

Design value of threshold chloride content/ critical chloride content will be as under:

- For concrete made with OPC = 0.4% by mass of cement
- For concrete made with PPC or concrete containing fly ash ≥ 25% by mass of cement =0.3% by mass of cement
- For concrete made with PSC or concrete containing GGBS ≥ 50% by mass of cement =0.3% by mass of cement

### 85.5.4.5 **Resistance to chloride ingress:**

Rate of ingress indicate the resistance of the concrete against the chloride ion penetration. Chloride Diffusion coefficient Da is considered the rate determining parameter of chloride ingress in concrete. Da is a time dependent function and decreases with time.

Initial Diffusion coefficient Da(t0), For normal structures, can be determined using following equation based on w/c

For OPC, Da(t0) ( mm2/yr) = 524(w/c)+68
For PPC, Da(t0) ( mm2/yr) = 361(w/c)-18.7
For PSC, Da(t0) ( mm2/yr) = 270(w/c)-51

Note

For concrete containing fly ash between 30 % to 35 %, Da value as proposed for PPC to be used.

For concrete containing fly ash between 15 % to 30%, for Da value linear interpretation to be done between OPC and PPC.

For concrete containing fly ash < 15 %, for Da value as proposed for OPC shall be used.

For concrete containing GGBS between 25 % to 50%, for Da value linear interpretation to be done between OPC and PSC.

For concrete containing GGBS< 25 %, for Da value as proposed for OPC shall be used.

### 85.5.4.6 **Characteristic values ( to be specified) ,** Da(t0) **and design values of chloride diffusion coefficient of concrete ,** Dd(t)**:**

The values determined on the basis of w/c ratio will be considered as characteristic values.

Design value of chloride diffusion coefficients for service life(exposure period) t will be calculted as under:

Dd(t) = Da(t0) *γm*At

Dd(t) = Design value of chloride diffusion coefficient after exposure period t

γm= Partial factor of safety of material

ϒm– values of ϒm shall vary from 1.0 to 1.15 depending upon quality control level selected and design service life as under

| Quality control level | Γm for Tsl <= 75 yr | For Tsl >75 yr |
|---|---|---|
| 1 -low | 1.11 | 1.15 |
| 2 - med | 1.05 | 1.1 |
| 3 - high | 1.0 | 1.05 |

At is time dependent multiplying factor

At = $\left(\frac{t_0}{teff}\right)^{\alpha}$

$t_0$ = initial duration of hydration at the time of exposure in years, to be taken as 0.2466 yrs (90 days)

$\alpha$ is the ageing factor which will be taken as under:

For Ordinary Portland cement concrete, $\alpha = 0.30$

For Portland fly ash cement concrete, $\alpha = 0.50$

For Blast furnace slag cement concrete, $\alpha = 0.45$

$t_{eff}$ ( duration in years through which reduction in Da will be there) = 0.8 x Tsl, subject to maximum value of 60 yr

### 85.5.4.7 **Verification of chloride diffusion coefficient, $D_{a(t0)}$**

At design mix stage for a known w/c or w/cm ratio, performance based accelerated test such as RCPT/electrical Resistivity shall be conducted as applicable and value of Da(t0) shall be determined based upon relationship refer to table 15 .The relationship is valid for concrete made with Ordinary Portland cement, for concrete made with PPC or concrete containing fly ash between 30 % to 35 % by mass of cement and for concrete made with PSC or concrete containing GGBS 50 % by mass of cement. For other percentages of SCM's , relationship should be drawn based upon the actual test values.

For important structures, Da(t0) shall be determined through unidirectional diffusion test as per IS516 part…, which is generally a long duration test as comparted to 7.3.5.3c i. values determined using above test will be considered as mean value Da(mean). Characteristic value Da(to) will be:

$$Da(t_0) = Da(mean) * 1.1$$

Note: durability tests as mentioned in ………... shall also be conducted for determining the values to be considered for acceptance during quality control.

**Table 15, Relationship between electrical resistivity and charge passed (RCPT)**

| Chloride diffusion coefficient Da (t0), $mm^2/yr$ | Electrical Resistivity ( kohm-cm) at 28 days | Charge passed ( in coulombs) at 28 days | Remarks |
|---|---|---|---|
| 50 | 50 | < 500 | 1.With OPC, it may not be possible to obtain Da(t0) <200. For lower values, blended cements need to be used.<br>2. With PPC, values of Da(to) <100 may not be possible. For values, less than 100, PSC need to be used. |
| 75 | 42 | 501-700 | |
| 100 | 29 | < 600 | |
| 125 | 24 | 601-800 | |
| 150 | 21 | 801-1000 | |
| 175 | 18 | < 1500 | |
| 225 | 12 | 1501-2000 | |
| 250 | 12 | 2001-2500 | |
| 300 | 11 | 2501-3000 | |

### 85.5.4.8 **Verification of performance against chloride ingress**

Verification of performance against chloride ingress in designed exposure class shall be done by determining service life on the basis of critical surface chloride content and threshold chloride content value using modified Fick’s second law of diffusion.

$$c(x,t) = Cs – (Cs – Ci) * erf(z)$$

where

Cx,t is the chloride content at cover depth x and exposure time t, % by mass of concrete;

Cs is the chloride content at the exposed surface, % by mass of concrete for different exposure classes as given in table ..;

Ci is the initial chloride content, % by mass of concrete ( to be determined as per IS 14959 Part 2 and it should not be more than as specified in table .. clause 7.3.5);

Dd(t) is the Design non-steady-state chloride diffusion coefficient, in m2/sec for time t;

erf(z) is the error function (refer table in appendix .. for values of erf(z) against diff values of z

$$Z = \frac{x}{(2* sqrt(Dd(t)*t ) )}$$

x is the nominal cover in mm

t is the design service life in yrs,

Dd(t) in $mm^2/yr$

It will be ensured that

Cx,t ≤ Cth ( threshold chloride content as given in clause …………..) .

#### 85.5.4.9 Effect of crack width on service life due to corrosion of rebar (due to chloride ingress or carbonation

Crack width can impact the durability, however effect of crack width at service loads on durability has not been established. The crack width control from serviceability requirements obtained through proper reinforcement detailing has been found to be sufficient in the absence of any established model for calculating its effect on durability. However for structures exposed to aggressive environment lower crack widths are recommended. As a guide following criteria can be used.

| **Sl.** | **Exposure type** | **Permissible crack width at service loads** |
|---|---|---|
| 1 | Normal structures except as under | 0.3 mm |
| 2 | Structures or Parts of structure exposed to high moisture continuously, alternate wetting drying , parts buried under soil and structures within 0.1 km of coastline | 0.2 mm |
| 3 | Structures or part of structures under sea water or in splash zone (exposure class, SW2 and SW3 | 0.15 mm |
| 4 | Water retaining structures | Refer IS 3370 |

### 85.5.5 Sulphate Attack

Sulphate attack most often occurs in concrete exposed to external sources although internal sources of sulphate can also result in damage, particularly when concrete is exposed to excessive temperature at early ages. Sulphate ions attack the hydrates in the cement paste, leading to the formation of ettringite and gypsum. The formation of these compounds can cause expansion, cracking and eventual disintegration of the concrete.

Sulphate attack can either be 'external' or 'internal'. Internal sulphate attack generally occurs when a source of sulphate is incorporated into the concrete at the time of mixing. Internal source of sulphates mostly consists of cements, aggregates and water. The excessive amounts of water-soluble sulphate from these or other mix constituents can cause expansion and disruption of concrete. 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.

To prevent this, the total water-soluble sulphate content of the concrete mix, expressed as $SO_3$ (where, $SO_3 = 0.833\ SO_4$), should not exceed 4 percent by mass of the cement in the mix. The sulphate content should be calculated as the total from the various constituents of the mix.

The 4 percent limit does not apply to concrete made with supersulphated cement complying with IS 6909.

Note1. Delayed ettringite formation (DEF) is a special case of internal sulfate attack. It occurs in concrete which has been cured at elevated temperatures, for example, where steam curing has been used. It can also occur in large concrete pours where the heat of hydration has resulted in high temperatures within the concrete.

### 85.5.6.1 **Design table against sulphate attack :**

Requirements for concrete to have adequate resistance to sulphate attack are given in table 16 below.

**Table 16: Requirements for concrete against sulphate attack**

| **Exposure classification** | **Min. grade of concrete** | **Min. Cementitious Content (kg/m$^3$)** | **Max. w/cm** | **Min. clear cover (mm)** | **Type of cement** |
|---|---|---|---|---|---|
| S0 | M 25 | 300 | 0.50 | 30 | Any cement allowed for RC |
| S1 | M 35 | 340 | 0.45 | 40 | SRC, PPC, PSC, OPC with slag or silica fume. |
| S2 | M 40 | 400 | 0.40 | 50 | SRC, PPC, PSC, OPC with slag or silica fume. |
| S3 | M 50 | 400 | 0.40 | 50 | SRC, PPC, |

**NOTES:**

i. Cement content given in this table is irrespective of grades of cement. The cement contents given in column 3 of this table are the minimum recommended. For $SO_3$ contents near the upper limit of any class, cement contents above these minimums are advised. For the purpose of calculation of $SO_3$ content (if the test results are in the form of $SO_4$), the following relation shall be used: $SO_3 = 0.833\ SO_4$.

ii. For severe conditions, such as thin sections under hydrostatic pressure on one side only and sections partly immersed, considerations should be given to a further reduction of water - cement ratio.

iii. Where chlorides encountered along with the sulphates in soil or ground water, ordinary Portland cement with content from 5 to 8 percent shall be desirable to be used in concrete, instead of sulphate resisting cement. alternatively, Portland slag cement conforming to IS 455 having more than 50 percent slag or a blend of ordinary Portland cement and slag may be used provided sufficient information is available on performance of such blended cements in these conditions.

iv. Supersulphated cement is not readily available, and has issues of strength reduction when subjected to temperatures above 40 oC. Thus, it has not been included in the above table.

### 85.5.6.2 Freezing and thawing

Deterioration of concrete exposed to freezing generally occurs when there is sufficient amount of internal moisture. The transition of water to ice results an increase in volume. When the saturated pores in the concrete freezes, it can cause severe cracking and disruption.

Some of the factors that can affect the freezing and thawing behavior of concrete are:

1. Moisture content
2. Curing regime and procedure
3. Severity of exposure condition (i.e. number of freeze cycle)
4. Compressive strength

Where freezing and thawing actions under wet conditions exist, enhanced durability can be obtained by the use of suitable air entraining admixtures. When concrete lower than grade M 50 is used under these conditions, the mean total air content by volume of the fresh concrete at the time of delivery into the construction should be as per table 17:

**Table 17 . Mean total air content ( by volume )**

| Nominal Maximum Size Aggregate mm | Entrained Air Percentage |
|---|---|
| 10 | 7 ± 1 |
| 20 | 5 ± 1 |
| 40 | 4 ± 1 |

Since air entrainment reduces the strength, suitable adjustments may be made in the mix design for achieving required strength. It shall be noted that air entraining admixtures complying the requirements of IS 9103 to be used.

### 7.3.8.3 Design table against freezing and thawing attack

Requirements for concrete for adequate resistance against freezing and thawing are given in table 18 below:

**Table18 : Requirements for concrete against freezing and thawing attack**

| **Class designation** | **Type of Cement** | **Maximum w/c** | **Minimum grade of concrete** | **Minimum cement content ( kg/m$^3$ )** | **Additional requirements** |
|---|---|---|---|---|---|
| F1 | OPC or PPC or PSC | 0.50 | M25 | 300 | As given in table 17 Mean Total air content (by volume ) |
| F2 | OPC or PPC or PSC | 0.45 | M30 | 320 | |
| F3 | PPC or PSC | 0.45 | M35 | 340 | |

Note: In case of exposure class F3, additional use of silica flume is advantages within the permissible limits of 35 % for fly ash and 50 % for GGBS.

### 85.5.7 Alkali Aggregate Reactivity

Alkali aggregate reactivity have been recognized as Alkali silica reactivity (ASR) and Alkali carbonate reactivity (ACR). Alkali carbonate reaction is associated with certain dolomitic limestone and it involves a reaction between an alkali source and certain dolomites. Alkali silica reaction (ASR) results from the reaction between alkali hydroxide in the pore solution and certain forms of reactive silica present in some types of siliceous aggregates. ASR is far more widespread than ACR.

Some aggregates containing particular varieties of silica may be susceptible to attack by alkalis ($Na_2O$ and $K_2O$) originating from cement or other sources, producing an expansive reaction which can cause cracking and disruption of concrete. Damage to concrete from this reaction will normally only occur when all the following are present together:

A high moisture level, within the concrete;

A cement with high alkali content, or another source of alkali; and

Aggregate containing an alkali reactive constituent.

#### 85.5.7.1.1 Tests for alkali Silica Reactivity:

To determine the manifestation of Alkali silica reactivity, Accelerated Mortar Bar test as per IS 383 and/or Mortar Bar test as per IS 2836 Part VII shall be conducted. In case of calcite limestone or dolomitic limestone aggregates are encountered, concrete prism bar test shall be preferred over mortar bar test. Till this test is included in IS 2386 part VII, specialist literature may be referred for the test and applicable requirement.

Where the service records of particular cement/aggregate combination are well established, and do not include any instances of cracking due to alkali-aggregate reaction, no further precautions should be necessary. When the materials are unfamiliar, precautions should take one or more of the following forms:

- Use of non-reactive aggregate from alternate sources.
- Use of low alkali ordinary 'Portland cement having total alkali content not more
  - than 0.6 percent (as $Na_2O$ equivalent).
  - Further advantage can be obtained by use of fly ash conforming to IS 3812 or granulated blast furnace slag conforming to IS 12089 as part replacement of ordinary Portland cement (having total alkali content as $Na_2O$ equivalent not more than 0.6 percent), provided fly ash content is at least 25 percent or slag content is at least 50 percent.
- Alternatively Portland Pozzolana Cement (PPC) or Portland Slag Cement (PSC) shall be used after determining the potential alkali-silica reactivity of combination of PPC or PSC proposed to be used at site for making concrete and aggregate using accelerated mortar bar method as per IS: 383 and the expansion of aggregate shall be less than 0.10% at 16 days.
- Measures to reduce the degree of saturation of the concrete during service such as use of impermeable membranes.
- Limiting the cement content in the concrete mix and thereby limiting total alkali content in the concrete mix. For more guidance, specialist literatures may be referred.

### 85.5.8 Abrasion

Abrasion of concrete surfaces results from friction, which may cause a grinding action; or by repetitive impact and overloading, which causes local crushing. Abrasion of concrete is broadly classified into following areas

- Wear on floor and slab construction
- Wear on concrete road surfaces
- Erosion of hydraulic structure due to abrasive action of material carried by flowing liquid
- Cavitation action on concrete in hydraulic structures due to high flow velocities and negative pressure

Abrasion resistant surfaces primarily involve consideration of surface finishing technique curing regime, aggregate characteristics, compressive strength and cement content. The use of controlled permeability formwork can help to reduce abrasion. Compressive strength of concrete may provide a relative indication of resistance.

Specialist literatures may be referred to for durability requirements of concrete surfaces exposed to abrasive action, for example, in case of machinery and metal tyres.

Abrasion resistance of concrete for industrial floors shall be determined as per IS516 part 9 through revolving disc method for the surfaces exposed to attrition, rubbing and friction. Abrasive resistance of concrete for hydraulic structures shall be determined as per IS 519 part 9, under water abrasion test.

### 85.5.9 Protection Measures

Special protection systems may be adopted in particularly aggressive environments, when normal protection measures are deemed insufficient, such as:

- Applying surface coverings, using specific products for concrete protection (paints or coatings),.
- Cathodic protection of reinforcements using sacrificial anodes or applied current,
- Active corrosion inhibitors.

Additional protections may have a useful life shorter than that of the structural element. The design shall therefore include schedules for the appropriate maintenance of these protection systems.

At sites where alkali concentrations are high or may become very high, the ground water should be lowered by drainage so that it will not come into direct contact with the concrete. Additional protection may be obtained by the use of suitable impermeable barriers.

## Annex A

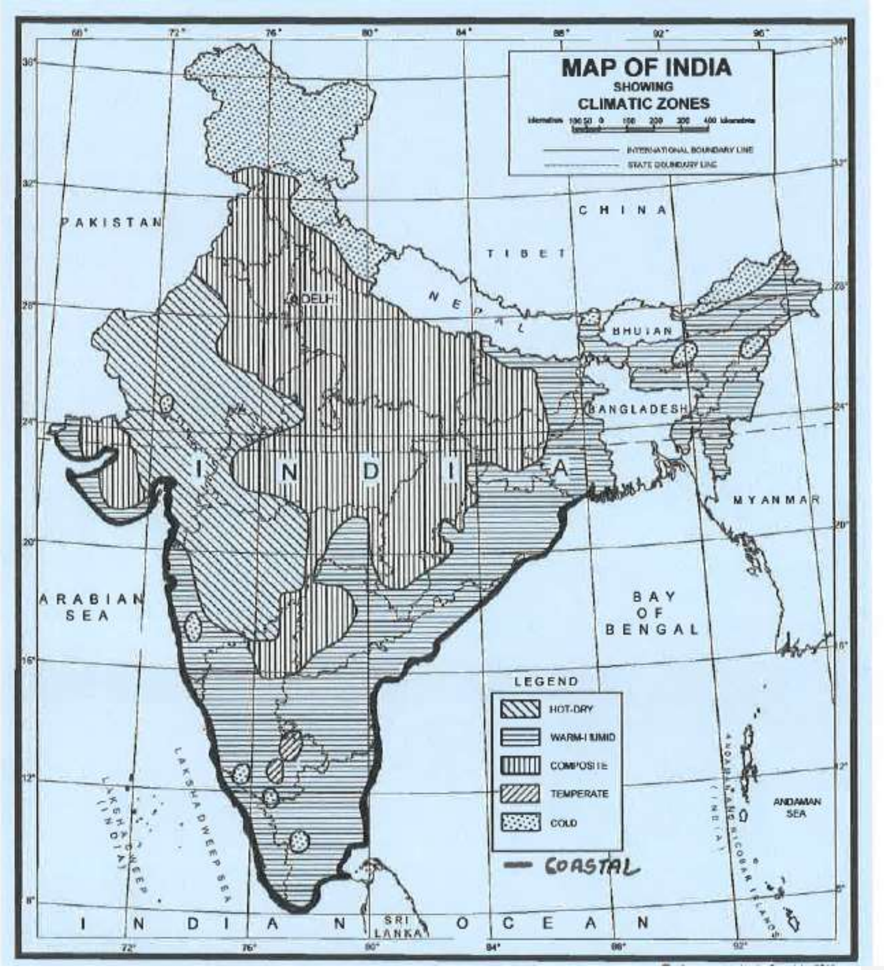

**Fig A. 1: Climatic zones as classified in India**

# Annex B

## Determination of mean carbonation coefficient $K_{1m}$ from accelerated carbonation resistance test as per IS 516-part 2 section 4.

IS 516 part (Part 2/Section 4) covers the procedure for the measurement of carbonation depth by accelerated carbonation test. For the determination of mean carbonation coefficient $K_{1m}$ , test specimens shall be exposed to carbon dioxide levels between 3.0 to 3.5 % ( by volume ) for 70 days after 28 days water curing prior to conditioning period of 14 days. The testing will take 112 days period.

Measure mean depth of carbonation, Cm after testing the concrete specimens as per the procedure IS 516-part 2 section 4. The mean carbonation coefficient, Km1 will be determined through following equation:

K1m = (Cm/t) 1/x

Where:

Cm= mean depth of carbonation in mm measured at 70 days exposure to CO2

x = constant , this value depends upon type of binder, x value in case of

Concrete made with OPC =0.60

Concrete made with PPC =0.45

Concrete made with PSC =0.45

t = Euivalent time in years represented by 70 day accelerated exposure.

For determining t in years from accelerated exposure period of 70 days following formula to be adopted

t = (70*x*10000)/(500*365)

[ x =% by volume of CO2 ].

## Annex C

## Determination of surface electrical resistivity of concrete using four point wenner probe method

The four-point electrode method is currently the most widely used method /technique for the measurement of concrete resistivity. During testing, a low frequency AC current is applied between the two outer electrodes while the voltage drop is measured in the two inner electrodes (refer to figure C.1).

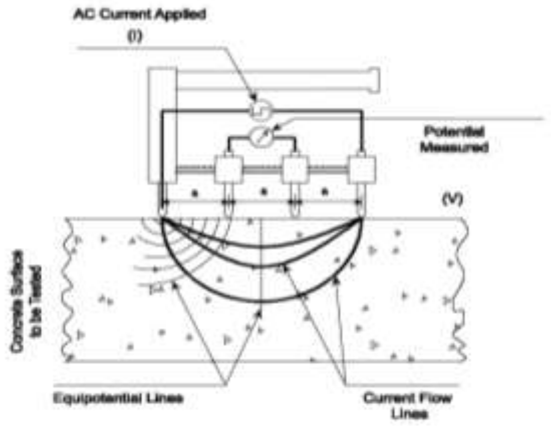

**Figure C.1: Four-Point Wenner Array Probe Test Setup**

The concrete resistivity for a semi-infinite region can be calculated as

$\rho = 2\pi aV/I$

where

$\rho$ is the electrical resistivity (Kohm-cm),

a is electrode spacing (m),

V is voltage drop (V), and

I is the current (A).

**Apparatus**: Wenner array probe capable of adjustment of the probe tip spacing up to 50 mm, a measuring range from 0 to 999±1 Kohm -cm , with a resolution of 0.1 Kohm-cm and an accuracy of +/- 2% of reading should be used for testing.

Test specimen: concrete slabs ( 300 mmx300 mmx100 mm) ( 3 nos )

Test procedure

The specimens should be at least 28 days water cured at the time of testing. The testing should be done at saturated surface dry condition. The test surface preferably shutter surface should be thoroughly cleaned to remove all laitance. At least 4 readings at different positions on the test surface of each specimen as shown in figure C.2 should be taken. The average value of the readings taken from three specimens represents the surface electrical resistivity of the concrete.

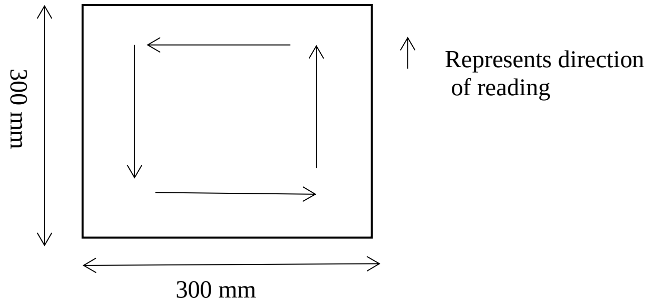

**Figure C.2 : Concrete Slab ( 300mmx300mmx100 mm)**