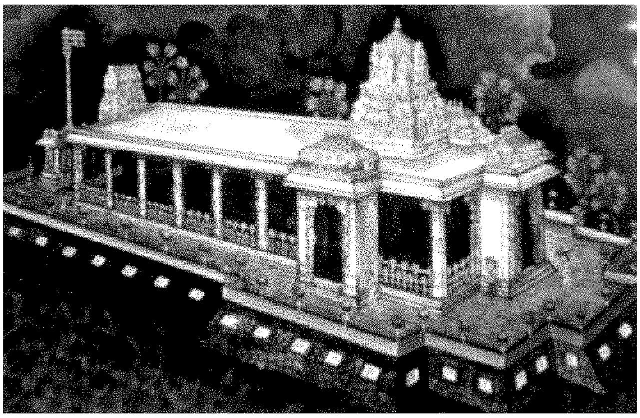


An architect's rendering of the Hindu Temple built at Kauai Island, Hawaii. A massive concrete foundation was laid to last for at least one thousand years. They have used high volume fly ash concrete replacing OPC by 57%.

Courtesy : P.K. Mehta

9

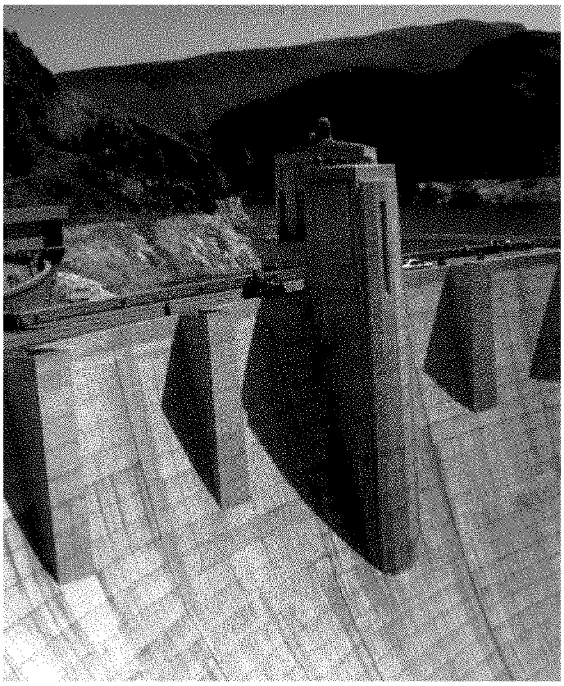
CHAPTER

Durability of Concrete

General

For a long time, concrete was considered to be very durable material requiring a little or no maintenance. The assumption is largely true, except when it is subjected to highly aggressive environments. We build concrete structures in highly polluted urban and industrial areas, aggressive marine environments, harmful sub-soil water in coastal area and many other hostile conditions where other materials of construction are found to be non-durable. Since the use of concrete in recent years, have spread to highly harsh and hostile conditions, the earlier impression that concrete is a very durable material is being threatened, particularly on account of premature failures of number of structures in the recent past.

In the past, only strength of concrete was considered in the concrete mix design procedure assuming strength of concrete is an all pervading factor for all other desirable properties of concrete including durability. For the first time, this pious opinion was proved wrong in late 1930's when



Hoover Dam, USA (1931-36).
A symbolic structure for Sustainable Development with ever 1000 years of predicted life

they found that series of failures of concrete pavements have taken place due to frost attack. Although compressive strength is a measure of durability to a great extent it is not entirely true that the strong concrete is always a durable concrete. For example, while it is structurally possible to build a jetty pier in marine conditions with 20 MPa concrete, environmental condition can lead this structure to a disastrous consequences. In addition to strength of concrete another factor, environmental condition or what we generally call exposure condition has become an important consideration for durability.

Concrete durability is a subject of major concern in many countries. Number of international seminars are held on concrete durability and numerous papers written on failures of concrete structures are discussed and state-of-the-art reports are written and disseminated , regularly.

In the recent revision of IS 456 of 2000, one of the major points discussed, deliberated and revised is the durability aspects of concrete, in line with codes of practices of other countries, who have better experiences in dealing with durability of concrete structures.

One of the main reasons for deterioration of concrete in the past, is that too much emphasis is placed on concrete compressive strength. As a matter of fact, advancement in concrete technology has been generally on the strength of concrete. It is now recognised that strength of concrete alone is not sufficient, the degree of harshness of the environmental condition to which concrete is exposed over its entire life is equally important. Therefore, both strength and durability have to be considered explicitly at the design stage. It is interesting to consider yet another view point regarding strength and durability relationship.

Strength and Durability Relationship

In the previous paragraphs, we have been discussing all the time that although the strength of concrete has direct relationship with durability it does not hold gold in all situations.

This aspect needs little more discussions.

Generally, construction industry needs faster development of strength in concrete so that the projects can be completed in time or before time. This demand is catered by high early strength cement, use of very low W/C ratio through the use of increased cement content and reduced water content. The above steps result in higher thermal shrinkage, drying shrinkage, modulus of elasticity and lower creep coefficients. With higher quantity of cement content, the concrete exhibits greater cracking tendencies because of increased thermal and drying shrinkage. As the creep coefficient is low in such concrete, there will not be much scope for relaxation of stresses. Therefore, high early strength concretes are more prone to cracking than moderate or low strength concrete. Of course, the structural cracks in high strength concrete can be controlled by use of sufficient steel reinforcements. But this practice does not help the concrete durability, as provision of more steel reinforcement, will only results in conversion of the bigger cracks into smaller cracks. All the same even this smaller cracks are sufficient to allow oxygen, carbon dioxide, and moisture get into the concrete to affect the long term durability of concrete.

Field experience have also corroborated that high early strength concrete are more cracks-prone. According to a recent report, the cracks in pier caps have been attributed to the use of high cement content in concrete. Contractors apparently thought that a higher than the desired strength would speed up the construction time, and therefore used high cement content.

Similarly, report submitted by National Cooperative Highway Research Programme (NCHRP) of USA during 1995, based on their survey, showed that more than, 100000 concrete bridge decks in USA showed full depth transverse cracks even before structures were less than one month old. The reasons given are that combination of thermal shrinkage and drying shrinkage caused most of the cracks. It is to be noted that deck concrete is made of high strength concrete. These concretes have a high elastic modulus at an early age. Therefore, they develop high stresses for a given temperature change or amount of drying shrinkage. The most important point is that such concrete creeps little to relieve the stresses.

A point for consideration is that, the high early strength concrete made with modern Portland cement which are finer in nature, containing higher sulphates and alkalis, when used 400 kg/m^3 or more, are prone to cracking. Therefore if long-term service life is the goal, a proper balance between a too high and a too low cement content must be considered. This is where the use of mineral admixtures comes in handy.

We discussed in the above paragraphs, that the present day practice is to use high early strength concrete for early completion of projects. We have also seen that high early strength concrete made by using very low W/C ratio of the order of 0.30 or less by using low water content and high cement content is prone to micro cracking which affects the long term durability.

It is interesting to see that the above point of view is not fully convincing when seen from many other considerations.

Firstly, the high early strength concrete has high cement content and low water content. On account of low water content, only surface hydration of cement particle would have taken place leaving considerable amount of unhydrated core of cement grains. This unhydrated core of cement grains has strength in reserve. When micro cracks have developed, the unhydrated core gets hydrated, getting moisture through micro cracks. The hydration products so generated seal the cracks and restore the integrity of concrete for long term durability.

Secondly, as per Aiticin, the quality of products of hydration (gel) formed in the case of low W/C ratio is superior to the quality of gel formed in the case of high W/C ratio. [9.1]

Again as per Aiticin, in low W/C ratio concrete (high early strength concrete) the weak transition zone between aggregate and hydrated cement paste does not exist at all. Unhydrated cement particles are also available in such low W/C ratio concrete for any eventual healing of micro cracks.

Thirdly, the micro structure of concrete with very low W/C ratio, is much stronger and less permeable. The interconnected network of capillaries are so fine that water cannot flow any more through them. It is reported that when tested for chloride ion permeability, it showed 10-50 times slower penetration than low strength concrete.

It is difficult to conclude whether the micro cracks developed in high early strength concrete reduces the long term durability or the delayed hydration of unhydrated core of cement grains would heal up the micro cracks and thereby improve long term durability along with the better quality of product of hydration, higher strength, reduced permeability, in case of low W/C ratio concrete. It is a subject for research.

Volume Change in Concrete

It will not be wrong to attribute the lack of durability to the reason of volume change in concrete. Volume change in concrete is caused by many factors. As a matter of fact, probing into the factors causing volume change in concrete will lead to an interesting study of concrete technology. The various causes that are responsible for volume change, fully expose the various factors affecting durability which encompasses wide spectrum of concrete technology.

If one takes a close look, one comes to know that, the entire hydration process is nothing but an internal volume change, the effect of heat of hydration, the pozzolamic action, the sulphate action, the carbonation, moisture movement, all types of shrinkages, the effect of chlorides, rusting of steel reinforcement and host of other aspects come under the preview of volume change in concrete.

It can also be viewed that it is the permeability that leads to volume change. The volume change results in cracks. It is the cracks that promotes more permeability and thus it becomes a cyclic action, till such time that concrete undergoes deterioration, degradation, disruption and eventual failure.

Definition of Durability

The durability of cement concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment.

Significance of Durability

When designing a concrete mix or designing a concrete structure, the exposure condition at which the concrete is supposed to withstand is to be assessed in the beginning with good judgement. In case of foundations, the soil characteristics are also required to be investigated. The environmental pollution is increasing day by day particularly in urban areas and industrial atmospheres. It is reported that in industrially developed countries over 40 per cent of total resources of the building industries are spent on repairs and maintenance. In India, the money that is spent on repair of buildings is also considerable. Every government department and municipal bodies have their own "Repair Boards" to deal with repairs of buildings. It is a sad state of affairs that we do not give enough attention to durability aspects even when we carry out **repairs**.

We carry out repairs job in a casual manner using only ordinary cement mortar practised decades back. Today, special repair materials and techniques are available. The use of such materials make the repair job more effective and durable. This aspects have been covered in chapter 5.

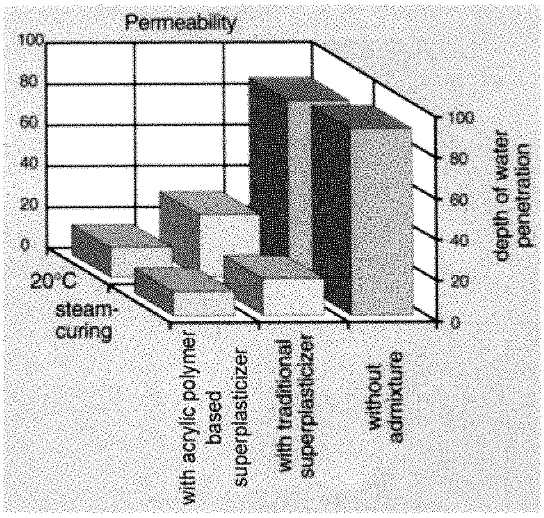
Another point for consideration is that, presently, the use of concrete has been extended to more hostile environments, having already used up all good, favourable sites. Even the good materials such as aggregate, sand are becoming short supply. No doubt that the cement production is modernised, but sometimes the second grade raw materials such as limestones containing excess of chloride is being used for pressing economical reasons. Earlier specifications of portland cement permitted a maximum chloride content of 0.05 per cent. Recently, maximum permissible chloride content in cement has been increased to 0.1 per cent. This high permissible chloride content in cement demands much stricter durability considerations in other aspects of concrete making practices to keep the total chloride content in concrete within the permissible limits. In other words, considerations for durability of modern concrete constructions assume much more importance, than hitherto practised.

Impact of W/C Ratio on Durability

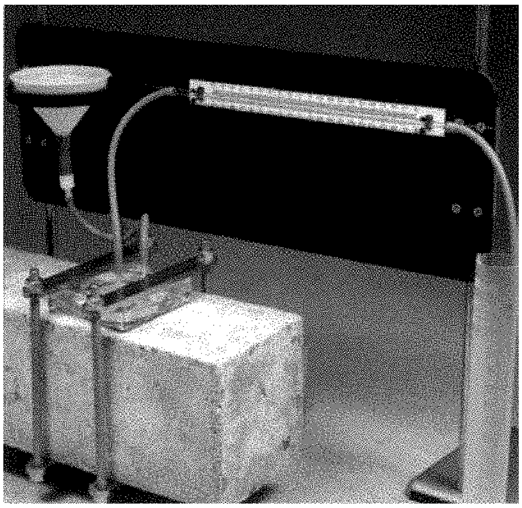
In the preceding pages we have discussed that volume change results in cracks and cracks are responsible for disintegration of concrete. We may add now that permeability is the contributory factor for volume change and higher W/C ratio is the fundamental cause of higher permeability. Therefore, use of higher W/C ratio — permeability — volume change — cracks — disintegration — failure of concrete is a cyclic process in concrete. Therefore, for a durable concrete, use of lowest possible W/C ratio is the fundamental requirement to produce dense and impermeable concrete.

There is a tremendous change in the micro structure of concrete made with high W/C ratio and low W/C ratio. With low W/C ratio the permeability decreases to such a level that these concretes are impervious to water. This does not mean that they do not contain interconnected network of capillaries, but these capillaries are so fine that water cannot flow any more through them. When such concretes are tested for chloride ions permeability test, it is found that chloride ions diffuse such concretes at a rate 10 — 50 times slower than that of high W/C ratio concrete.

It has been proved beyond doubt that low W/C ratio concrete are less sensitive to carbonation, external chemical attack and other detrimental effects that causes lack of durability of concrete. It has been reported that it become impossible to corrode unprotected steel reinforcement in accelerated corrosion test of a concrete with very low W/C ratio. From this it could be inferred that the best way to protect reinforcing steel against corrosion is to use low W/C and adequate cover, rather than using higher W/C ratio and then protecting the steel by epoxy coating.



Degree of Permeability



Cap and Initial Surface Absorption Test (ISAT)

It is easy to preach on paper the virtues of using low W/C ratio for all-round durability of concrete. But in actual practice for many years it has been found almost impossible to reduce the W/C ratio below 0.4. This situation has changed for the last fifteen years in India with the practice of using superplasticizers. The advent and use of superplasticizers have revolutionised the art and science of making durable concrete by drastically reducing the W/C ratio of concrete. The modern superplasticizers are so efficient that it is now possible to make flowing concrete with a W/C as low as 0.25 or even as low as 0.20. This technological breakthrough, in conjunction with the use of silica fume and other secondary cementitious materials, has made it possible to develop a new family of high-strength concrete which is generally referred as high-performance concrete—a concrete which is very durable.

In most of these new low W/C ratio concretes, as explained earlier, there is not enough water available to fully hydrate all the cement particles. The water available can only hydrate the surface of cement particles and there exist plenty of unhydrated particles which can play an important role as they constitute strength in reserve. If for any reasons, structural or environmental, concrete gets cracked, the unhydrated cement particles begin hydrating as soon as water or moisture starts penetrating through cracks. This is to say that unhydrated cement particles offer self healing potential to improve durability of concrete.

Permeability

We have discussed that W/C ratio is the fundamental point for concrete durability. Another important point for consideration is the permeability of concrete. When we talk about durability of concrete, generally we start discussion from the permeability of concrete as it has much wider and direct repercussion on durability than that of W/C ratio. For example, micro-cracks at transition zone is a consideration for permeability whereas W/C ratio may not get involved directly. It may be mentioned that microcracks in the initial stage are so small that they may not increase the permeability. But propagation of microcracks with time due to drying shrinkage, thermal shrinkage and externally applied load will increase the permeability of the system.

Permeability of Cement Paste

The cement paste consists of C-S-H gel, $Ca(OH)_2$ and water filled or empty capillary cavities. Although gel is porous to the extent of 28 per cent, the gel pores are so small that hardly any water can pass through under normal conditions. The permeability of gel pores is estimated to be about 7×10^{-16} m/s. That is approximately about 1/100 of that of paste.[9.2] Therefore, the gel pores do not contribute to the permeability of cement paste.

The extent and size of capillary cavities depend on the W/C ratio. It is one of the main factors contributing to the permeability of paste. At lower W/C ratio, not only the extent of capillary cavities is less but the diameter is also small. The capillary cavities resulting at low W/C ratio, will get filled up within a few days by the hydration products of cement. Only unduly large cavities resulting from higher W/C ratio (say more than 0.7) will not get filled up by the products of hydration, and will remain as unsegmented cavities, which is responsible for the permeability of paste.

Table 9.1 shows the permeability of cement paste at various ages

Table 9.1. Reduction in Permeability of Cement Paste (W/C Ratio = 0.7) with Progress of Hydration. [9.3]

Age days	*Coefficient of Permeability Km/s*
fresh	2×10^{-6}
5 days	4×10^{-10}
6 days	1×10^{-10}
8 days	4×10^{-11}
13 days	5×10^{-12}
24 days	1×10^{-12}
ultimate	6×10^{-13} (calculated)

It is very interesting to see that the permeability of cement paste with very low W/C ratio can be compared to the permeability of dense rocks. Table 9.2 shows the comparison between permeabilities of rocks and cement paste.

Table 9.2. Comparison Between Permeabilities of Rocks and Cement Pastes [9.2]

Type of Rocks	*Coefficient of Permeability m/s*	*Water/cement ratio of mature paste of the same permeability*
1. Dense trap	2.47×10^{-14}	0.38
2. Quartz ciorite	8.24×10^{-14}	0.42
3. Marble	2.39×10^{-13}	0.48
4. Marble	5.77×10^{-12}	0.66
5. Granite	5.35×10^{-11}	0.70
6. Sandstone	1.23×10^{-10}	0.71
7. Granite	1.56×10^{-10}	0.71

From Table 9.2 it is seen that the cement paste even with high W/C ratio of 0.70 is quite impervious as that of granite with coefficient of permeability of 5.35×10^{-11} m/s. This value of coefficient of permeability is so small, that physically no water will permeate through in any perceptible manner. However in actual practice, it is noticed that mortar and concrete exhibit appreciable permeability much higher than the values shown in the table 9.2. This is definitely not because of the permeability of aggregates in mortar or concrete. The aggregate used in mortar or concrete is as impermeable as that of paste as can be seen in Table 9.2. The higher permeability of mortar or concrete in actual structures is due to the following reasons.

(*a*) Formation of microcracks developed due to long term drying shrinkage and thermal stresses.

(*b*) The large microcracks generated with time in the transition zones.

(*c*) Cracks generated through higher structural stresses.

(*d*) Due to volume change and cracks produced on account of various minor reasons.

(*e*) Existence of entrapped air due to insufficient compaction.

Fig. 9.1 shows the relation between permeability and capillary porosity of cement paste

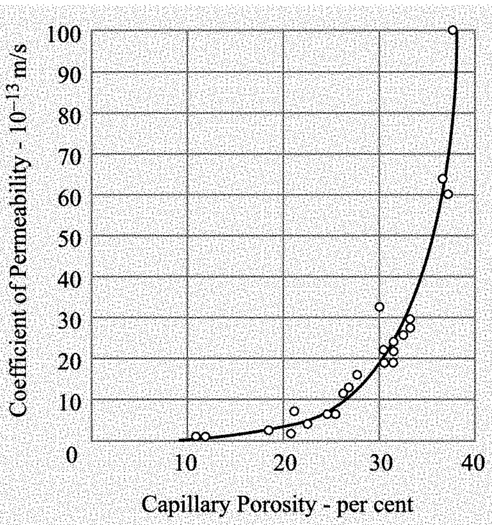


Fig. 9.1. Relation between permeability and capillary porosity of cement paste[9.2]

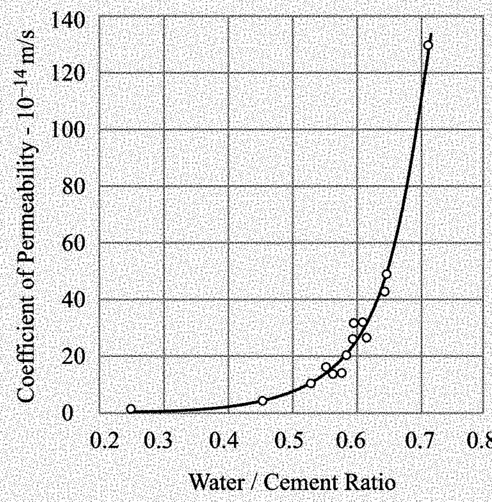


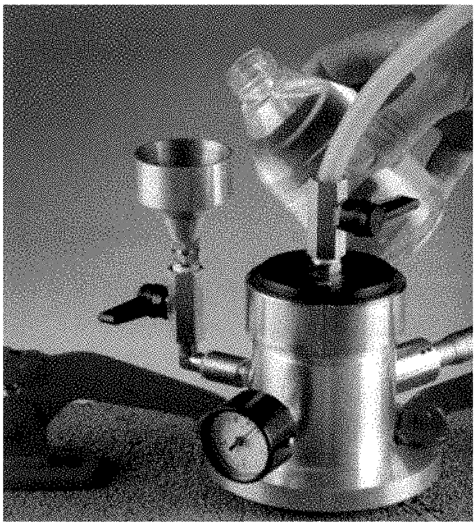
Fig. 9.2. Relation between permeability and water/cement ratio for mature cement pastes (93 per cent of cement hydrated)[9.3]

Fig. 9.2 shows the relation between permeability and water/cement ratio for mature cement paste (93 per cent of cement hydrated)

From Fig. 9.2 it can be seen that coefficient of permeability increases more than 100 times from W/C ratio of 0.4 to 0.7. Therefore, many code of practices fix the maximum W/C ratio at 0.4 so that the ingress of aggressive chemicals is restricted. The restriction of W/C ratio is also imposed in liquid retaining structures.

Permeability of Concrete

Theoretically, the introduction of aggregate of low permeability into cement paste, it is expected to reduce the permeability of the system because the aggregate particles intercept the channels of flows and make it take a circuitous route. Compared to neat cement paste, concrete with the same W/C ratio and degree of maturity, should give a lower coefficient of



In-situ Water Permeability of Concrete

permeability. But in practice, it is seen from test data it is not the case. The introduction of aggregate, particularly larger size of aggregates increase the permeability considerably.

The explanation lies in the development of microcracks that are produced in the transition zone. Opinion differs in this regard about the size of microcracks that are generated at the transition zone. However, the drying shrinkage, thermal shrinkage and externally applied load may cause cracks in weak transition zone at the young age. It is reported that the size of the cracks in transition zone is much bigger than most of the capillary cavities present in cement paste.

Table 9.3 shows the typical observed values of permeability of concrete used in some of the dams in the United States.

The use of pozzolanic materials in optimum proportion reduces the permeability of concrete. This is evidently due to the conversion of calcium hydroxide, otherwise soluble and leachable, into cementitious product.

Though air-entrainment, makes the concrete porous, when used up to 6%, makes the concrete more impervious, contrary to general belief.

Table 9.3. Typical values of Permeability of concrete used in Dams

Cement Content kg/m^3	*Water/Cement Ratio*	*Permeability* 10^{-12} *m/s*
156	0.69	8
151	0.74	24
138	0.75	35
223	0.46	28

High pressure steam cured concrete in conjunction with crushed silica decreases the permeability. This is due to the formation of coarser C-S-H gel, lower drying shrinkage and accelerated conversion of $Ca(OH)_2$ into cementitious products.

Interaction Between Permeability, Volume Change and Cracking

In the preceding pages we have discussed about permeability, volume change and cracking of concrete are responsible for lack of durability of concrete and concrete structures. It is difficult to pin point which of these are primarily responsible for affecting durability. Permeability of concrete is often referred as the root cause for lack of durability. But it can be seen that volume change that takes place in an otherwise impervious concrete due to heat of hydration or internal manifestation can crack the concrete affecting durability. Microcracks in transition zone even in initially impermeable concrete, can start the cycle of deterioration process in concrete. Therefore, these three factors, one follows the other two, like day follows the night, are responsible for affecting durability of concrete and concrete structures.

Table 9.4. Types and Causes of Concrete Cracks[9.4]

	Prehardening				
1. Primary Classification	Constructional Movement		Settlement Shrinkage	Setting Shrinkage	
2. Secondary Classification	Subgrade	Formwork	Reinforcement or formwork obstructions	Plastic shrinkage	Drying shrinkage
3. Cause	Settlement of subgrade	Movement of formwork	Settlement of concrete during setting	Chemical reactions	Rapid drying while setting occurs
4. Example or Contributing Condition	Moisture changes in Subgrade or lack of compaction of subgrade	Swelling of wood or pressure of wet concrete	Settlement of around obstruction mix too fluid	Cracks occur soon after placing and under moist conditions	Cracking of exposed surfaces due to high wind, low humidity or temperature differential
5. Remedy	Control of subgrading	Construction of adequate forms	Dense mixes with low water content & adequate compaction of low lifts	Remedy not clear but refloating eliminates cracks	Proper protection

Table 9.5. Other Properties, Types and Causes of Concrete Cracking[9.4]

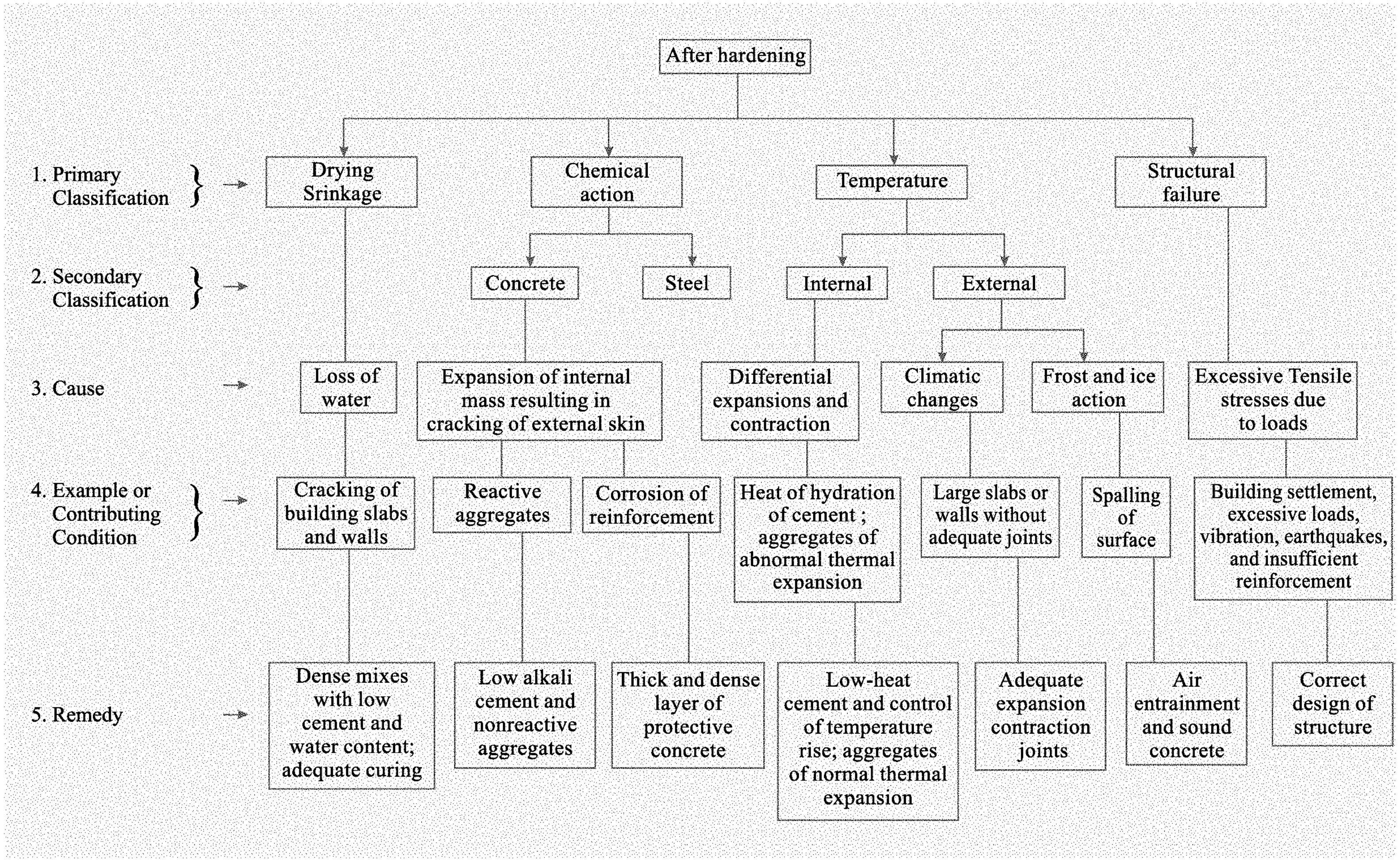
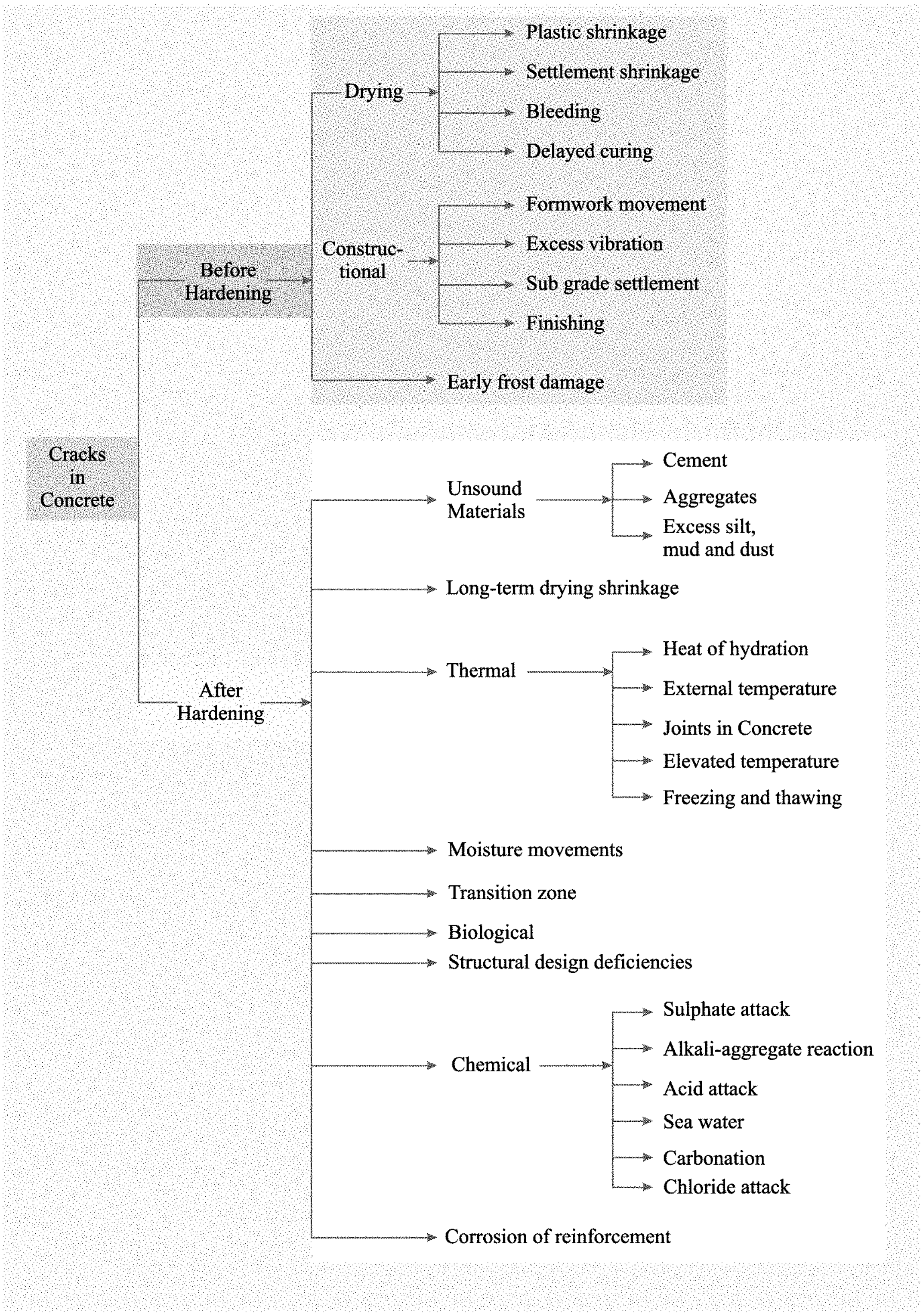


Table 9.6. Various Types and Causes of Cracks in Concrete



In discussing durability of concrete, one can go through permeability route, or volume change route or cracks route. By following any one of the above routes, it is possible to discuss the entire aspects of durability of concrete. Therefore, we shall follow the "cracks in concrete" route to discuss and describe the various factors that affect the durability of concrete.

Factors Contributing to Cracks in Concrete

Mercer L.B,[9.4] in his paper on classification of concrete cracks has given two tables to explain the types and causes of concrete cracks Refer Table 9.4 and Table 9.5.

But in our discussion, we shall follow the Table 9.6 step by step and describe the cracks in concrete which affect durability of concrete.

Plastic Shrinkage Cracks

Water from fresh concrete can be lost by evaporation, absorption by subgrade, formwork and in hydration process. When the loss of water from surface of concrete is faster than the migration of water from interior to the surface, the surface dries up. This creates moisture gradient which results in surface cracking while concrete is still in plastic condition. The magnitude of plastic shrinkage and plastic shrinkage cracks are depending upon ambient temperature, relative humidity and wind velocity. In other words, it depends upon the rate of evaporation of water from the surface of concrete.

Rate of evaporation in excess of 1 kg/m^2 per hour is considered critical (refer Fig. 5.33). In such a situation, the following measures could be taken to reduce or eliminate plastic shrinkage cracks.

- Moisten the subgrade and formworks.
- Erect temporary wind breakers to reduce the wind velocity over concrete.
- Erect temporary roof to protect green concrete from hot sun.
- Reduce the time between placing and finishing. If there is delay cover the concrete with polythylene sheets.
- Minimise evaporation by covering concrete with burlap, fog spray and curing compound.

Fig. 9.3 shows the typical plastic shrinkage cracks. It is seen that cracks are parallel to one another and are spaced 0.3 to 1.0 meter apart. They can be deep and the width varying from 0.1 to 3.0 mm.

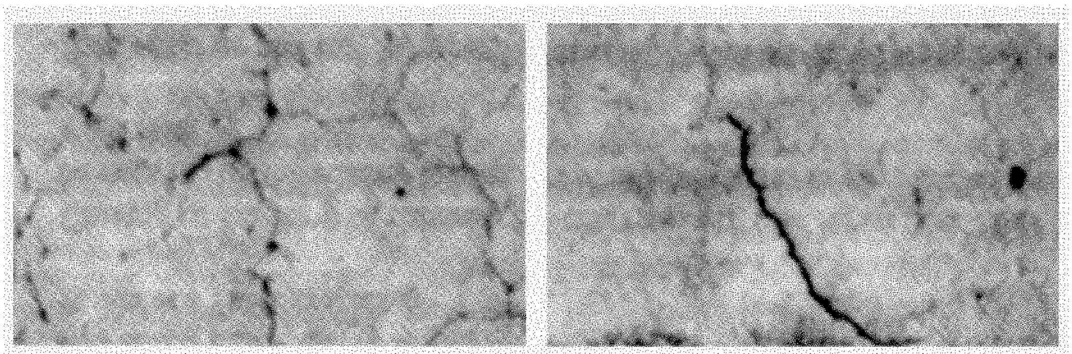


Fig. 9.3. Typical plastic shrinkage cracks.[9.5]

Plastic shrinkage cracks are very common in hot weather conditions in pavements floor and roof slab concrete.

Once they are formed it is difficult to rectify. In case of prefabricated units, they can be healed by controlled revibration, if the concrete is in plastic condition. In roof and floor slab it is difficult to repair. However, sometimes, a thick slurry is poured over the cracks and well worked by trowel after striking each side of the cracks to seal the same. The best way is to take all precautions to prevent evaporation of water from the wet concrete, finish it fast, and cure it as early as feasible.

In Mumbai - Pune express highway, the fresh concrete is protected by 100 meter long low tent erected on wheel to brake the wind and also to protect the green concrete from hot sun. In addition, curing compound is sprayed immediately after finishing operations.

Plastic shrinkage cracks, if care is not taken, will affect the durability of concrete in many ways.

Settlement Cracks

Plastic concrete when vibrated or otherwise settles. If the concrete is free to settle uniformly, then there is no cracks. If there is any obstruction to uniform settlement by way of reinforcement or larger piece of aggregate, then it creates some voids or cracks. This is called settlement cracks. This generally happens in a deep beam.

Concrete should be poured in layers and each layer should be properly compacted. Building up of large quantity of concrete over a beam should be avoided.

Sometimes, the settlement cracks and voids are so severe it needs grouting operators to seal them off. Revibration, if possible is an effective step. Otherwise, they effect the structural integrity of the beam or any other member and badly affects, the durability.

Bleeding

Water being the lightest ingredient of all the other materials in concrete, bleeding, *i.e.*, the upward movement of water when concrete settle downwards, is natural in concrete.

The bleeding water, in certain situations emerge at the surface and in some other situations may not come up to the surface. But bleeding does take place.

The bleeding water gets trapped by flat or flaky pieces of aggregates and also by reinforcement and gets accumulated below such aggregates and reinforcement. This is known as internal bleeding. In addition to internal bleeding, the water may further emerge out and accumulate on the top surface of concrete.

Firstly the internal bleeding water trapped below flat pieces of aggregate and reinforcement affect the bond between hardened cement paste, (hcp) and aggregate or reinforcement on account of local higher W/C ratio. The interface is easily prone to microcracking due to shrinkage stresses caused on dissipation of heat of hydration and drying shrinkage. The interface becomes a weak link in concrete. On loading, the micro cracks propagate further, making the concrete susceptible to degradation by environmental agencies.

The bleeding water, emerged at the top surface of concrete, when evaporates make the top surface porous, having very little abrasion resistances. Often, masons float the concrete when bleeding water is still standing on the surface. Too much working of the top surface presses the coarse aggregate down and brings up fine particles of cement and water. Such top surface made up of too fine materials with excess water develops cracks and craziness, affecting durability of concrete.

Delayed Curing

Fundamental requirement for good concrete is to maintain uninterrupted hydration, especially at early age, when the hydration process is faster. If young concrete dries up fast due to hot sun, drying winds and lower relative humidity, the top surface of concrete is devoid of enough water for continuous hydration process. This results in, as described earlier, formation of plastic shrinkage cracks, poorly formed hydration products and all other deformities in the structures of hydrated cement paste.

Modern high grade cements, being finely ground, with higher C_3S content needs early curing particularly in hot weather conditions. Structural members which are thin with large surface to volume ratio, such as sunbreakers, chajja etc., needs early curing .

The common builders in India have wrong notion that curing is to be done only on the following day of concreting. They are insensitive to the cry of drying and thirsty concrete for water or keeping it in required state of wetness. Delayed curing or interruption in continuous curing or not curing for required period are common bad practices followed in most of the construction site in India. Delayed or interrupted curing or not curing for required period can be compared to an ill fed babies or poorly fed human beings during growing years. Such persons are vulnerable to all kinds of diseases and sure to die prematurely. Similarly, insufficient curing is one of the major causes for lack of strength and durability of concrete structures.

Constructional Effects

In many construction sites, properly designed standard formworks are not used. Formworks are made in an adhoc manner. Such formworks may fail to maintain their rigidity and firmness when wet concrete is placed and vibrated. Sinking, bending, settlement or lack of rigidity of formwork may cause cracks or deformation in plastic concrete, after compaction, which may go unnoticed.

It is well known that excess vibration causes segregation which affects the uniformity of concrete mix. These days high consistency concrete is used either for pumping requirements or on account of using superplasticizers. Care must be taken to vibrate such high slump concrete, otherwise, segregation is sure to take place. Segregated concrete matrix, devoid of coarse aggregate, shrinks more than homogeneous concrete and exhibits high shrinkage cracks.

Recently, in one of the major construction sites in Mumbai-Pune express highway, in a road over bridge prestressed concrete girder, 2 to 3 cm thick matrix emerged next to the shuttering plate on account of careless over vibration. On removal of formwork, due to delayed or inefficient curing visible cracks and craziness appeared in such places only. The contractor, much against advice, chipped off such cracked mortar and replastered. This action is sure to reduce the durability of such important prestressed concrete girder.

Finishing becomes an important operation in situations where abrasion resistance is an important factor, such as roads and airfield pavements, factory floor, dock yard, warehouse floor etc. Ideally, cement paste must be contained by fine aggregate and matrix must be contained by coarse aggregates. Such a uniform mixture, devoid of excess paste on the surface will suffer from almost no shrinkage and exhibit good abrasion resistance. The stiffness of concrete at the time of trowelling, extent of trowelling and method of trowelling will all become important to improve the abrasion resistance and durability of concrete surface.

Early Frost Damage

At low temperature, the rate of hydration is slow. The hydration process stops at about −10°C. Till such low temperature hydration process though slow, continues. Freshly mixed concrete must not be exposed to freezing condition to protect the same from disruptive action of ice lens. Ice lens will assume 9 per cent more volume than the equivalent water volume. The cumulative effect of increased volume disrupts the integrity of fresh concrete. Once frozen, it is difficult to bring back the integrity of concrete subsequently. It is reported that significant ultimate strength reductions up to about 50 per cent, can occur if concrete is frozen within a few hours after placement or before it attains a compressive strength of 3.5 MPa.

Pure water freezes at 0°C. The water in fresh concrete is not pure, but it is a solution of various salts and as such it does not freeze at 0°C, but at lower than 0°C. It must also be understood that, as long as the temperature is more than –10°C, the hydration process continues and concrete gets heated due to heat of hydration. The temperature inside concrete is also influenced by formwork material, reinforcements exposed to outside weather, and thickness of member. Therefore, it is difficult to forecast whether the concrete has undergone freezing or not.

When concrete has attained a strength of 3.5 MPa, some quantity of water may have been consumed in the hydration process as bound water and certain amount of water may have been imbibed in gel pores. The gel pores are so fine that no ice could be formed in it. Partially filled capillary water, even if it is frozen no appreciable damage will have taken place to seriously disrupt the concrete. It is only saturated and fully filled capillary water, when no hydration has taken place, if frozen, will cause disruption of concrete and affect the longterm durability.

Unsound Materials

Cement or aggregate is considered unsound when they cause unacceptable extent of volume change in hardened concrete or mortar which causes cracks and affects durability.

In cement, if the raw materials contain more lime that can combine with other acidic oxides, or if the raw materials are not properly burnt to the required temperature for the lime to get fully combined with other oxides, cement becomes unsound. Similarly, the presence of MgO which reacts with water in the similar manner as CaO, can also cause unsoundness.

We all know that gypsum is added in appropriate quantity depending upon the C_3A content to prevent flash setting. By chance if gypsum is added in excess quantity, it can cause unsoundness in cement by way of slow expansion in hardened concrete.

Aggregates containing certain materials such as shale, clay lumps, coal, iron pyrites etc. show unsoundness later when concrete undergoes wetting and drying or freezing and thawing. Moisture absorption is often used as a rough index for unsoundness. But there are standard tests for unsoundness of coarse aggregates.

Now a days, crushed sand is being used more often in large works and this practice will grow. Unless proper care is taken crushed sand is likely to contain considerable amount of dust. The excess dust (very fine particle less than 75 micron) is harmful from many points of view and more important being that it causes cracks in concrete.

In many parts of our country, good natural fine aggregate is not available. Often they contain unacceptable amount of organic and inorganic fine particles referred as silt. Excess silt in sand interfere with setting time, shrinkage and bond strength. The ultimate effect is the reduction in tensile strength and shrinkage cracks. One of the contributory causes of cracks and craziness in plaster is the presence of excessive silt and mud in natural sand.

Shrinkage

Shrinkage of concrete is one of the important factors contributing to lack of durability of concrete. Shrinkage is mainly responsible for causing cracks of larger magnitude or minor microcracks. The aspect of cracking in concrete is very complex, involving many factors such as magnitude of shrinkage, degree of retraint, extensibility of concrete, extent of stress relaxation by creep and at what age the shrinkage is appearing etc. Fig. 9.4 shows the influence of shrinkage and creep relaxation on concrete cracking. From Fig. 9.4 it can be seen that the cracking does not take place at the predicted point, but the cracking is delayed because of stress relaxation due to creep. Cracks can be avoided only if the stress induced by shrinkage strain, after relaxation by creep, is at *all* time less than the tensile strength of concrete. The above situation is not happening in most of the cases and as such generally shrinkage causes cracks in concrete.

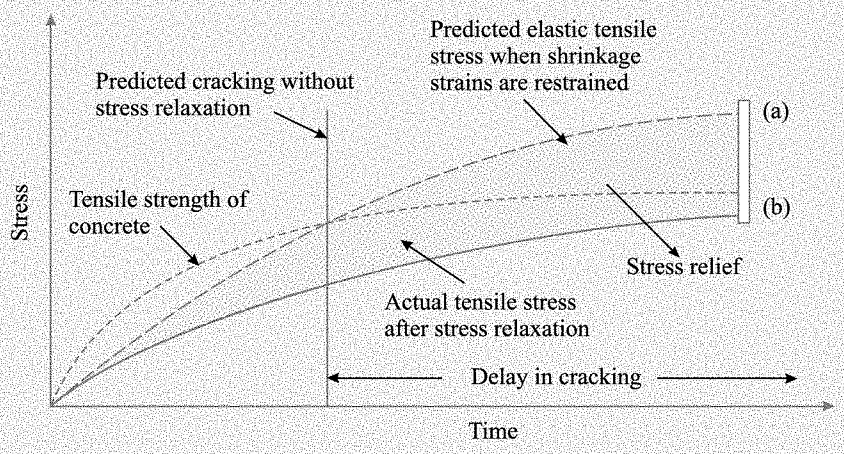


Fig. 9.4. Influence of shrinkage and creep on concrete cracking$^{9.6}$

When we discuss about the shrinkage of concrete, there are mainly three aspects of shrinkage which are required to be considered.

Firstly, the drying shrinkage, secondly the thermal shrinkage related to heat of hydration and subsequent cooling, and thirdly thermal shrinkage in connection with concrete subjected to variation of ambient temperature. First let us consider the long-term drying shrinkage, which is referred as drying shrinkage.

Drying Shrinkage

Drying shrinkage in concrete is an inherent property of concrete. This aspect has been dealt in greater detail in chapter 8. The shrinkage is one of the fundamental reasons for initial induction of micro cracks in concrete. The mechanism involved are too complex. Generally the pattern of shrinkage is a function of cement content, water content and W/C ratio. In practical terms at a constant W/C ratio, shrinkage increases with an increase in cement content. But at a given, workability, which approximately means a constant water content, shrinkage is unaffected by the increase in cement content, or shrinkage even decreases because of lower W/C ratio. At lower W/C ratio concrete is stronger to resist shrinkage. But it should not be forgotten that stronger concrete, creeps less, there is less stress relaxation and therefore more microcracks. Fig. 9.5 shows the pattern of shrinkage connecting cement content, W/C ratio and water content.

The drying shrinkage takes place over long period if concrete is subjected to lower relative humidity. Fig. 9.6 shows the range of shrinkage vs time. It is seen that shrinkage increases with time but at a decreasing rate.

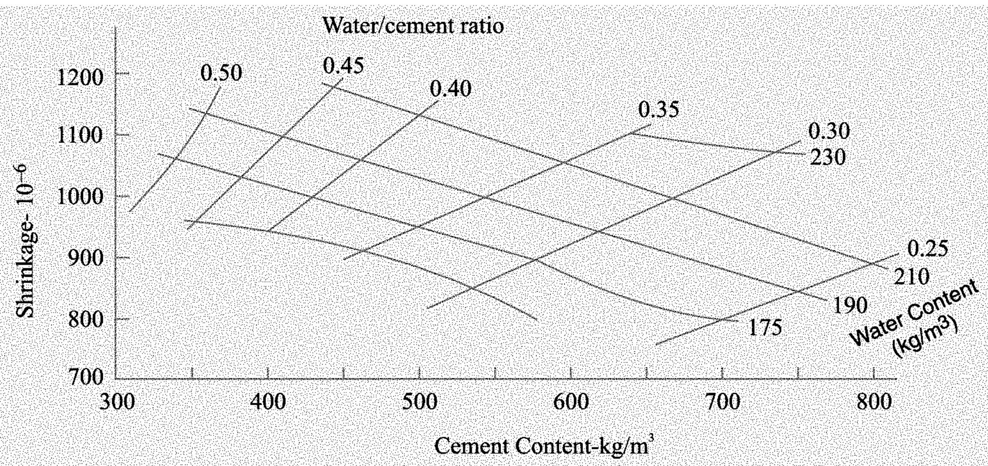


Fig. 9.5. The pattern of shrinkage as a function of cement content, water content, and water/cement ratio; concrete moist-cured for 28 days, thereafter dried for 450 days [9.7]

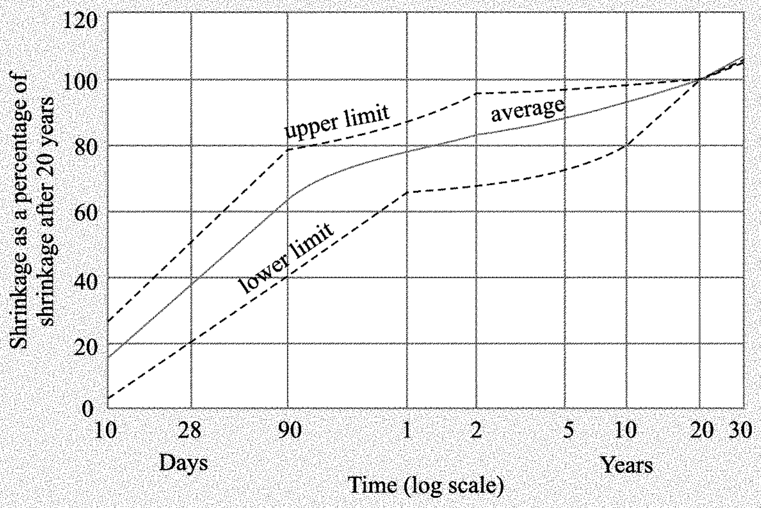


Fig. 9.6. Range of shrinkage-time-curves for different concretes stored at relative humidities of 50 and 70 per cent [9.8]

According to ACI 209 R–92 the development of shrinkage with time is given by the following equation :

$$S_t = \frac{t}{35+t} \times S_{ult.}$$

when S_t = shrinkage after t days since the end of 7 days moist curing

$S_{ult.}$ = ultimate shrinkage, and

t = time in days since the end of moist curing.

Although the above equation is subject to considerable variability, this equation can be used to estimate ultimate shrinkage of a wide range of moist cured concrete. It can be seen that 50 per cent of ultimate shrinkage is expected to occur after 35 days of drying. For steam cured concrete, the value of 35 in the denominator is replaced by 55, and time t is reckoned from the end of steam curing, generally 1 to 3 days.

The IS 456 of 2000, makes the following statement: In the absence of test data, the approximate value of the total shrinkage strain for design may be taken as 0.0003.

In addition to drying shrinkage, which takes place throughout the mass, may be in different magnitude, across the cross-section, there is another type of shrinkage known as carbonation shrinkage which occurs near surface zone of concrete where CO_2 can react with $Ca(OH)_2$ to form $CaCO_3$. Carbonation of concrete has serious repercussion on concrete durability which we shall deal separately. For the time being we shall deal with the total shrinkage due to drying and subsequent carbonation. Fig. 9.7 shows the drying shrinkage and carbonation shrinkage of mortar at different relative humidities. It can be noticed that carbonation shrinkage is nearly nil at less than 25 percent relative humidity because of lack of water in the pores to produce carbonic acid. The carbonation shrinkage is also nearly nil at 100 per cent relative humidity because the diffusion of CO_2 is not taking place on account of pores filled with water. It can also be noted that at 100 per cent relative humidity even drying shrinkage is also not there. Possibly there is a slight swelling.

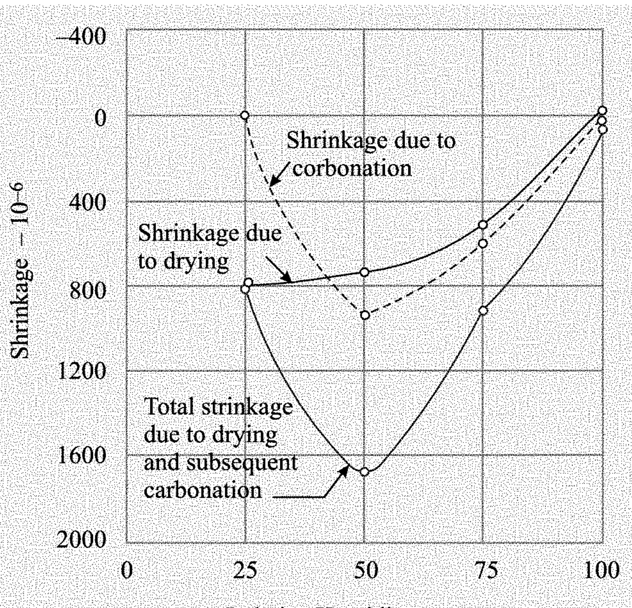


Fig. 9.7. Drying shrinkage and carbonation shrinkages of mortar at different relative humidities[9.9]

It is mentioned earlier that 50 per cent of shrinkage will have taken place in about 35 days or considerable amount of shrinkage will have taken place near the surface even earlier. At such an early age, the concrete will not have attained good strength to resist shrinkage stress and therefore the concrete is much more vulnerable to cracking inspite of stress relaxation by high creep at low strength.

Thermal Shrinkage

We shall now discuss the aspects of shrinkage associated with heat of hydration. Before we go into the aspect of thermal shrinkage it is necessary at this stage to go into the thermal properties of concrete to understand the behaviour of concrete to heating and cooling. The study of thermal properties of concrete is an important aspect while dealing with the durability of concrete.

Concrete is a material used in all climatic regions for all kinds of structures. Knowledge of thermal expansion is required in long span bridge girders, high rise buildings subjected to variation of temperatures, in calculating thermal strains in chimneys, blast furnace and pressure vessels, in dealing with pavements and construction joints, in dealing with design of concrete dams and in host of other structures where concrete will be subjected to higher temperatures such as fire, subsequent cooling, resulting in cracks, loss of serviceability and durability.

The important properties that will be discussed are:

- Thermal conductivity
- Thermal diffusivity
- Specific heat
- Coefficient of thermal expansion

Thermal conductivity: This measures the ability of material to conduct heat. Thermal conductivity is measured in joules per second per square metre of area of body when the temperature deference is 1□C per metre thickness of the body.

The conductivity of concrete depends on type of aggregate, moisture content, density, and temperature of concrete. When the concrete is saturated, the conductivity ranges generally between about 1.4 and 3.4 j/m^{2}s □c/m. Typical values of conductivity of concrete made with different aggregates are listed in Table 9.7.

Table 9.7. Typical Values of Thermal Conductivity of Concrete Made with Different Aggregates.[9.11]

Type of Aggregate	Wet density of concrete kg/m^3	Conductivity J/m^2 S□C/m
Quartzite	2440	3.5
Dolomite	2500	3.3
Limestone	2450	3.2
Sandstone	2400	2.9
Granite	2420	2.6
Basalt	2520	2.0
Baryte	3040	2.0
Expanded shale	1590	0.80

Table 9.8 shows the values of conductivity recommended by Loudon and Stacey [9.10]

Thermal diffusivity: Diffusivity represents the rate at which temperature changes within the concrete mass. Diffusivity is simply related to the conductivity by the following equation.

Diffusivity =

where C is the specific heat, and P is the density of concrete.

The range of diffusivity of concrete is between 0.002 to 0.006 m^2/h

Table 9.8

Values of Conductivity Recommended by Loudon and Stacey.[9.10]

Unit weight	Conductivity, $Jm/m^2s^oC/m$							
	For concrete protected from weather				For concrete exposed to weather			
kg/m^3	Aerated concrete	Light weight concrete with foamed slag	Light weight concrete with expanded clay or sintered fly ash	Normal weight aggregate concrete	Aerated concrete	Light weight concrete with foamed slag	Light weight concrete with expanded clay or sintered fly ash	Normal weight aggregate concrete
320	0.109	0.087	0.130		0.123	0.100	0.145	
480	0.145	0.116	0.173		0.166	0.130	0.187	
640	0.203	0.159	0.230		0.223	0.173	0.260	
800	0.260	0.203	0.303		0.273	0.230	0.332	
960	0.315	0.260	0.376		0.360	0.289	0.433	
1120	0.389	0.315	0.462		0.433	0.360	0.519	
1280	0.476	0.389	0.562		0.533	0.433	0.635	
1440		0.462	0.678					
1600		0.549	0.794	0.706				0.808
1760		0.649	0.952	0.838				0.952
1920				1.056				1.194
2080				1.315				1.488
2240				1.696				1.904
2400				2.267				2.561

***Specific heat*:** It is defined as the quantity of heat required to raise the temperature of a unit mass of a material by one degree centigrade. The common range of values for concrete is between 840 and 1170 j/kg per °C

Coefficient of thermal expansion

Coefficient of thermal expansion is defined as the change in unit length per degree change of temperature. In concrete it depends upon the mix proportions. The coefficient of thermal expansion of hydrated cement paste varies between 11 x 10^{-6} and 20 x 10^{-6} per °C. Coefficient of thermal expansion of aggregates vary between 5 x 10^{-6} and 12 x 10^{-6} per °C. Limestones and Gabbros will have low values and Gravel and Quartzite will have high values of coefficient of thermal expansion. Therefore the kind of aggregate and content of aggregate influences the coefficient of thermal expansion of concrete.

It has been discussed earlier while dealing with the properties of aggregate in chapter 3, that too much of thermal incompatibility between aggregate and paste, causes differential expansion and contraction resulting in rupture of bond at the interface of paste and aggregate. Coefficient of thermal expansion of 1:6 concrete made with different aggregates is given in Table 9.9

Table 9.9. Coefficient of Expansion of 1:6 Concretes Made with Different Aggregates[9.12]

	Linear coefficient of thermal expansion		
Type of aggregate	*Air-cured concrete 10^{-6} per °C*	*Water-cured concrete 10^{-6} per °C*	*Air cured and wetted concrete 10^{-6} per °C*
Gravel	13.1	12.2	11.7
Granite	9.5	8.6	7.7
Quartzite	12.8	12.2	11.7
Dolerite	9.5	8.5	7.9
Sandstone	11.7	10.1	8.6
Limestone	7.4	6.1	5.9
Portland stone	7.4	6.1	6.5
Blast furnace slag	10.6	9.2	8.8
Foamed slag	12.1	9.2	8.5

The values of the coefficient of thermal expansion of concrete, so far discussed applies to concrete subjected to a temperature less than about 65°C. It has been seen that the concrete subjected to higher temperatures show somewhat different values, presumably because of the lower moisture content in the concrete. The importance of the values of coefficient of thermal expansion becomes necessary at higher temperature when dealing with concrete subjected fire or higher temperatures. Table 9.10 shows the values of the coefficient of thermal expansion at conditions of higher temperatures.

Having seen a few aspects of properties of concrete which have bearing on expansion and contraction on heating and cooling, let us revert back to thermal shrinkage associated with heat of hydration.

A large quantity of heat, up to about 500 j/g (120 cal/g) could be liberated in the hydration of cement. Since the thermal conductivity of concrete is low, a very high

Table 9.10

Coefficient of Thermal Expansion of Concrete at High Temperature[9.13]

Curing condition	Water/ cement ratio	Cement content Kg/m^3	Aggregate	Linear coefficient of thermal expansion at the age of			
				28 days		90 days	
				below 260°C 10^{-6} per °C	Above 430°C 10^{-6} per °C	below 260°C 10^{-6} per °C	above 430°C 10^{-6} per °C
	0.4	435		7.6	20.3	6.5	11.2
Moist	0.6	310	Calcareous	12.8	20.5	8.4	22.5
	0.8	245	Gravel	11.0	21.1	16.7	32.8
Air	0.4	435		7.7	18.9	12.2	20.7
50 per cent	0.6	310	Calcareous	7.7	21.1	8.8	20.2
relative humidity	0.8	245	Gravel	9.6	20.7	11.7	21.6
Moist air	0.68	355	Expanded	6.1	7.5	—	—
	0.68	3.55	Shale	4.7	9.7	5.0	8.8

temperature could be generated in the interior of a large mass of concrete. At the same time, the exterior of the concrete mass loses heat with the result a steep temperature gradient may get established. During subsequent cooling of the interior, serious cracking may take place.

The rate of evolution of heat as well as total heat generated depend on the compound composition of cement. C_3S and C_3A produces large amount of heat in a short time. The fineness of cement also influences the rate of heat development. The faster rate of heat development is more harmful than the total heat of hydration which develops slowly. Therefore for mass concrete and hydraulic structures they use cement with low C_3S and C_3A. It is also advantages to use low cement content and blended cement. Blended cement with high pozzolanic material content gives out the heat rather slowly because of slow pozzolanic reaction, during which time certain quantity of heat gets dissipated, virtually reducing temperature difference between interior and exterior.

Mass Concrete

Mass concrete is a concrete having considerable dimensions that may get affected by thermal behaviour of concrete. Concrete dam is one such example of mass concrete. In the design of dam, strength of concrete is of less importance. The primary considerations are given to the aspect of how to reduce the heat of hydration, or if certain amount of heat is generated, how to absorb such heat so that the heat inside the body of concrete is minimised so that it does not cause any detrimental effect by way of cracks in concrete.

Now a days, there are many structural elements which are of sizeable dimensions, such as bridge piers, deep beams, massive columns, thick foundation concrete etc. The pouring of concrete in such massive sections need understanding of thermal behaviour mainly with respect to heat of hydration of concrete. Fig. 9.8 and Fig. 9.9 show the typical pattern of temperature change which causes external and internal cracking of large concrete mass. Fitz Gibbon from his research work has shown that temperature deference of more than 20□C between surface and interior causes cracks, assuming the coefficient of thermal expansion of

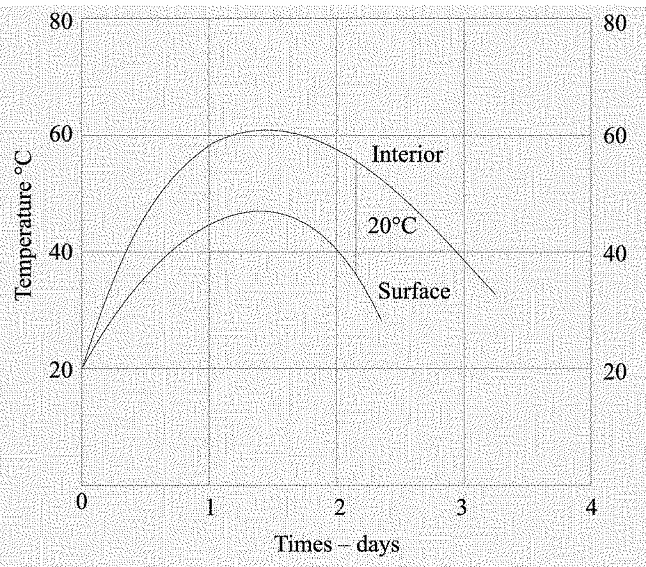


Fig. 9.8. An example of the pattern of temperature change which causes external cracking of large concrete mass. The critical 20âC temperature difference occurs during cooling. [9.14]

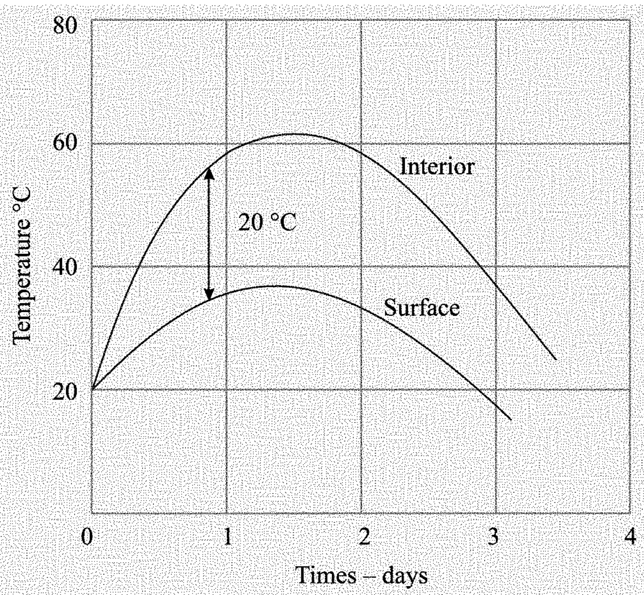


Fig. 9.9. An example of the pattern of temperature change which causes internal cracking of a large concrete mass. The critical 20°C temperature difference occurs during heating but the cracks open only when the interior has cooled through a greater temperature range than the exterior[9.14]

concrete as 10×10^{-6} per °C. A deference of 20□C, the differential strain would come to 200×10^{-6}. This amount of strain is considered as a realistic tensile strain at cracking.

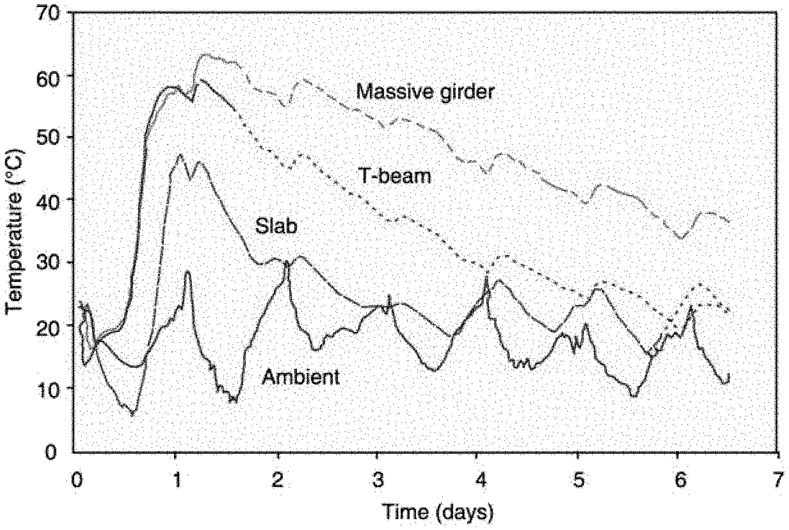
Aitcin and Raid cites a case where a 1.1 m square column made of reinforced concrete with ASTM type I cement content of 500 kg/m^3 and a silica fume content of 30 kg/m^3, showed a rise in temperature of 45□C above the ambient temperature, after 30 hours of placing.[9.15] Therefore, there is a need for controlling the heat of hydration in concrete and also not allowing the surface of the concrete to cool rapidly. If the surface is insulated, the difference in temperature between interior and exterior is reduced which improves the cracking behaviour. The retention of the formwork and its insulating properties may be made use of to reduce the difference in temperatures between interior and surface,.

In reinforced mass concrete structures also cracking will develop, if the difference in temperature between the interior and the exterior is large. However, appropriate detailing of the reinforcement could be made to control the width and spacing of cracks. Fitz Gibbon estimated that the temperature rise under adiabatic condition is 12□C per 100 kg of cement per cubic metre of concrete, regardless of the type of cement used, for cement contents between 300 to 600 kg/m^3. The above fact shows the importance of using blended cement in mass concrete and use of high volume fly ash in concrete constructions for crack free durable concrete.

Thermal Expansion and Shrinkage

Now let us see the effect of expansion and contraction of concrete subjected to ambient increase or decrease of temperature and their effect on concrete cracking.

Earlier we have discussed about the increase and decrease of temperature in concrete due to heat of hydration. The thermal changes due to heat of hydration will only be important for first few days in normal structures but may last for longer time in large mass concrete.



Temperature variations observed at the centre of the three structural elements

Under the above sub heading we are going to discuss about the concrete members, subjected to external temperatures, such as roof slabs, road or airfields pavements, bridge decks and other members. They are subjected to diurnal or seasonal changes of temperature. In India, in certain states or cities, the change of ambient temperature in day and night could be as high as 30°C and the change in temperature of actual concrete surface is much higher than 30°C. The seasonal variation could be as large as 40°C. In other countries, in the middle east it is of still higher ranges.

The change in diurnal or seasonal temperatures mentioned above makes the concrete expand and contract. Since the structures are not free to expand and contract on account of restraint at support in case of roof slabs and sub grade reaction in case of pavements, a considerable tensile stress more than the tensile strength is generated resulting in cracks in concrete.

It is usual that the diurnal variation of temperature in a place like Pune is 20°C or more, (In many other cities, in India too the variation could be as high as 30°C or more). We can have an idea of the tensile strain or the tensile stress that could develop in concrete, from the following calculations.

Assume the characteristic compressive strength of concrete = 25 MPa

Modulus of Elasticity = 5000 x N/mm^2

= 2.5 x 10^4 N/mm^2

Flexural strength = 0.7 x $\sqrt{f_{ck}}$ N/mm^2

= 3.5 N/mm^2

Assume the coefficient of thermal expansion of concrete

= 10 x 10^{-6} per °C

Assume the diurnal variation of temperature is 20°C

∴ The thermal shrinkage strain = 20 x 10 x 10^{-6}

= 200 x 10^{-6}

It has been seen from experimental work by Lowe that concrete cracks at a differential strain of 200 x 10^{-6}. A strain which is more than 200 x 10^{-6} will cause a very high degree of microcracking in concrete. Further,

$$\text{Modulus of elasticity} = \frac{\text{stress}}{\text{strain}}$$

$$2.5 \times 10^4 = \frac{\text{stress}}{200 \times 10^{-6}}$$

$$\text{Tensile stress} = 2.5 \times 10^4 \times 200 \times 10^{-6}$$

$$= 5.0 \text{ N/mm}^2$$

Tensile strength of concrete = 3.5 N/mm^2. Therefore a tensile stress of 5.0 N/mm^2 is sure to cause microcracks in concrete. The tensile stress will be much higher in case of stronger concrete with higher modulus of elasticity and higher degree of variation of temperature. No doubt concrete of higher compressive strength will have higher tensile strength to withstand the higher tensile stress. But due to lower value of creep of such concrete, a smaller extent of stress relaxation takes place and as such, the stronger concrete will crack more than weaker concrete, from this consideration. However, as written earlier, cracking of concrete is a complex matter.

Extensibility

From the above examples and explanation the magnitude of the shrinkage strain is only one of the factors contributing to the cracking of concrete. The following are the other factors influencing the cracking of concrete.

- *Modulus of elasticity:* The lower the modulus of elasticity, the lower will be the amount of induced elastic tensile stress for a given magnitude of shrinkage.
- *Creep:* The higher the creep, the higher is the extent of stress relaxation and hence lower is the net tensile stress.
- *Tensile strength:* Higher the compressive strength, the higher will be the tensile strength, and therefore the lower is the risk that tensile stress will exceed the tensile strength.

The combination of factors that are desirable to reduce the advent of cracking in concrete can be described by a single term called extensibility. Concrete is said to have a higher degree of extensibility when it can be subjected to large deformations without cracking. Obviously, for a minimum cracking, the concrete should undergo not only less shrinkage but also should have high degree of extensibility (*i.e.*, low elastic modulus, high creep, and high tensile strength). In general, as said earlier, high strength concretes may be more prone to cracking on account of greater thermal shrinkage (higher cement content) and lower stress relaxation (lower creep). On the other hand, low-strength concretes tend to crack less, because of lower thermal shrinkage (lower cement content) and higher stress relaxation (higher creep).

Incidentally, lime mortar has 5 – 7 times more extensibility than cement mortar. Therefore, lime mortar used as plaster, cracks less than cement mortar plaster.

In the foregoing paragraphs we have described about the cracks produced in concrete by virtue of variation in the ambient temperature. Sometimes it is possible that combined effect of shrinkage caused by heat of hydration, long-term drying shrinkage and shrinkage on account of variation in ambient temperature, may cause such a high total shrinkage, which may virtually cause considerable cracking of concrete members, particularly in roof slabs and concrete pavements.

It is seen that roof slab may not leak for a few years. But due to long-term drying shrinkage increasing with age and cyclic expansion and contraction due to variation of external temperatures, the roof slab is likely to leak, after some years. The leakage causes corrosion of reinforcements and affect the durability. Therefore however good is the quality of concrete in roof slab water-proofing treatment is necessary to stop leakage and to increase the durability of structures.

Similarly, to improve the functional efficiency and to increase the durability of concrete pavements well-planned and designed joints must be provided in the construction of road, airfield pavements and industrial floors.

Joints in Concrete

Industrial floors and concrete pavements are constructed generally in alternate bays to allow for the incidental early shrinkage of concrete. A time interval as much as practicable is given between the adjacent bays to provide scope for the maximum possible shrinkage. In case of roof slab of large dimension and in other special cases, expansion joints are provided to cater for the expansion and contraction. In pavements proper joints are provided to direct the possible cracks arising out of expansion and out of thermal expansion and contraction, due to variation in temperature and also due to long-term drying shrinkage.

Joints can be broadly classified into four categories:

- Construction joints
- Expansion joints
- Contraction joints
- Isolation joints

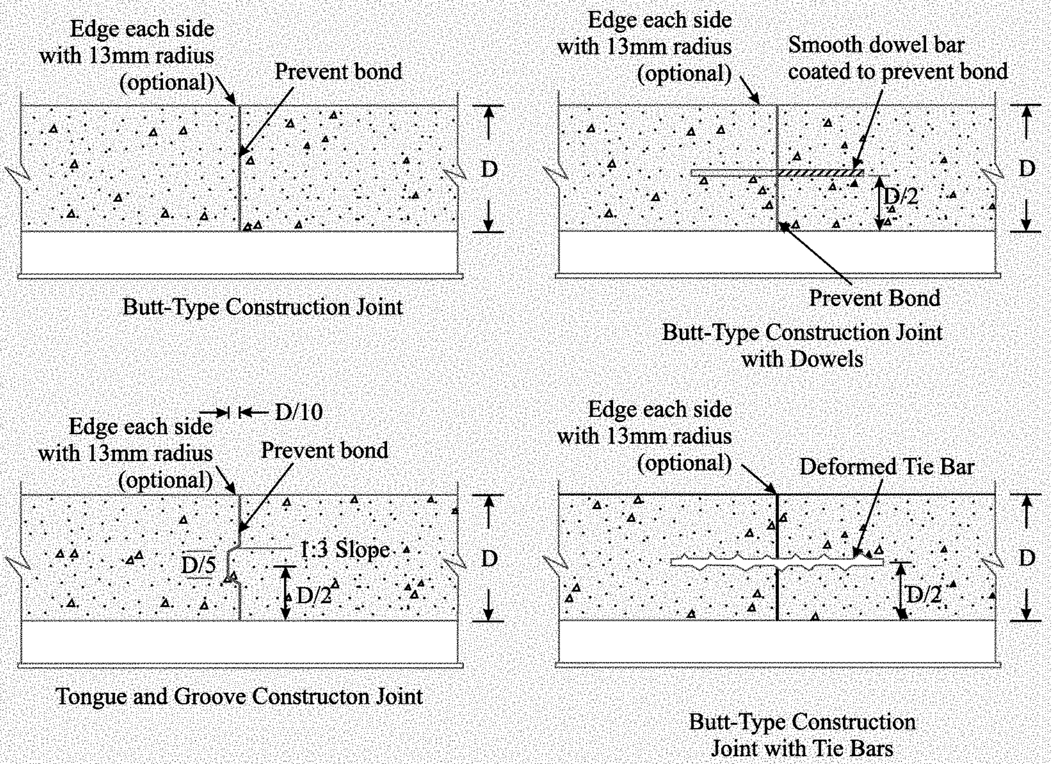
Construction Joints

Construction joints are the temporary joints left between subsequent concreting operations. The position of the construction joints should be pre-planned before concreting is started. Till such position and location, concrete must be poured in one operation. The joints must be made at such places that the concrete is less vulnerable to maximum bending moment and maximum shear force. In walls and columns construction, joints should be horizontal and arranged at such a level to coincide with the general architectural features. In columns, the concrete should be filled to the level, preferably, a few inches below the junction of beams. Joints in beams and slabs should be formed at the point of minimum shear. It is also not desirable to have the construction joints at the point of maximum bending moment, therefore the joints may be made at the extreme position of the middle third.

The procedure for joining the new concrete to the old concrete at the place of construction joint has been described under placing of concrete. Construction joint should be properly masked when finishing the structure. Badly made and unmasked construction joint will give an ugly appearance to the concrete construction. The groove may be incorporated at the joint to make a feature and to hide the joint. Refer Fig. 9.10.

Expansion Joints

Concrete is subjected to volume change due to many reasons. Provision must be made to cater for the volume change by way of joint to relieve the stresses produced. Expansion is a function of length. In case of a small building, the magnitude of expansion is not much and



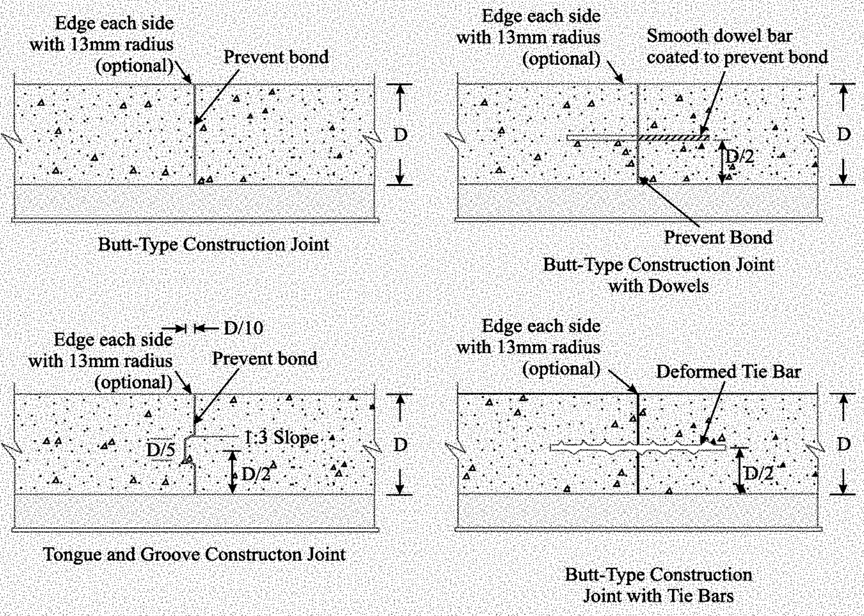
Butt-Type Construction Joint

Butt-Type Construction Joint with Dowels

Tongue and Groove Constructon Joint

Butt-Type Construction Joint with Tie Bars

Fig. 9.10. Construction Joints



Butt-Type Construction Joint

Butt-Type Construction Joint with Dowels

Tongue and Groove Constructon Joint

Butt-Type Construction Joint with Tie Bars

Fig. 9.10. Construction Joins

therefore no joint is necessary either in the floor or in the roof. A long building experiences large expansion. It is estimated that for the worst conditions, a long building may undergo an expansion as much as 2.5 cm. Therefore, buildings longer than 45 metres are generally provided with one or more expansion joints.

The roof is one of the building elements subjected to maximum temperature differences. The roof is subjected to expansion and contraction during day and night or from season to season and causes pushing or pulling to the supporting load bearing walls. Serious cracks have been experienced in the masonry walls supporting the slab. Attempts are made to create a condition for slab to slide over the wall to prevent pushing and pulling.

In the past, expansion joints were provided at closer intervals in the floors and pavements. These days from experience, it is seen that concrete does not actually expand to the extent indicated by the simple analytical calculations, because of the frictional resistance offered by the subgrade. It is therefore, possible to provide expansion joints at a much farther interval than in the past. I.S. 456-2000 recommends as under:

In view of the large number of factors involved in deciding the location, spacing and nature of expansion joints, the provision of expansion joint in reinforced cement concrete structures should be left to the discretion of the reinforced concrete designer. For purposes of general guidance, however, it is recommended that structures exceeding 45 m in length shall be divided by one or more expansion joints.

The details as to the length of a structure where expansion joints have to be provided can be determined after taking into consideration various factors, such as temperature, exposure to weather, the time and season of the laying of the concrete, etc. Under no circumstances shall a structure of length 45 m or more be without an expansion joints. Refer Fig. 9.11.

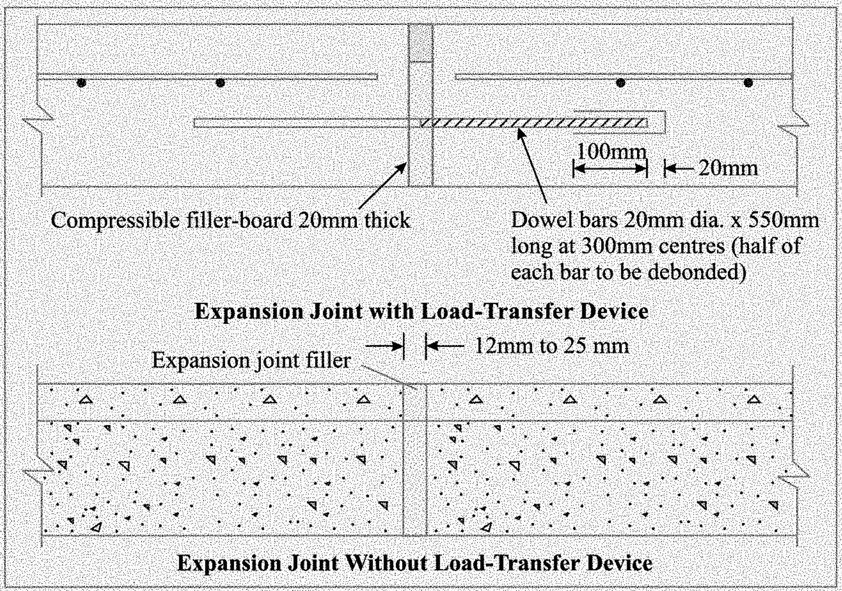
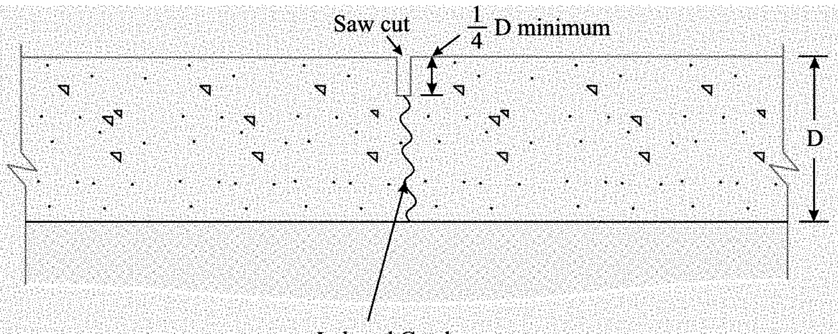


Fig. 9.11. Expansion Joints

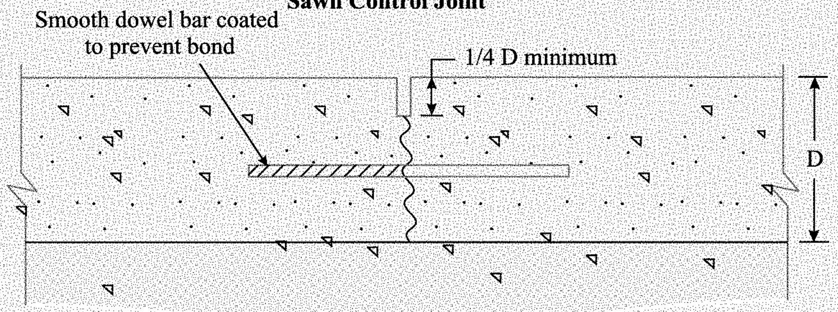
Contraction Joints

Concrete undergoes plastic shrinkage and drying shrinkage as a result of which concrete shrinks. When shrinkage is restrained, stresses are developed which result in the formation of cracks. To avoid these cracks, contraction joints are provided. Normally, the interval at which these joints are provided will vary from 5 to 10 metres. Contraction joints are sometimes called dummy joints or control joints. Contraction joints will not be necessary if sufficient reinforcement is provided to take up the shrinkage stresses. Contraction joints are generally provided in unreinforced floors and pavements.

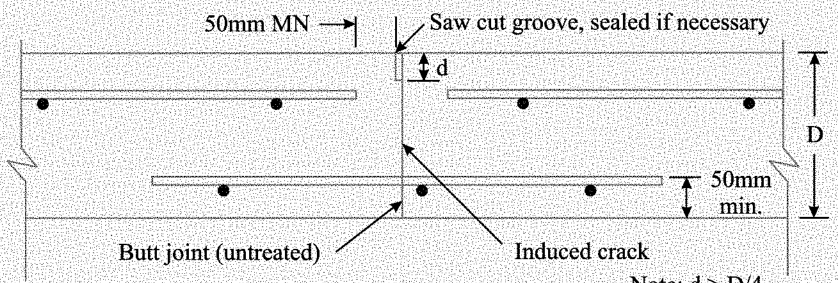
Contraction joints are made at the time of laying concrete by embedding a timber batton or plank of sufficient depth and required thickness. This is subsequently removed when the concrete is hardened. Sometimes, steel plates of sufficient thickness and width are beaten



Sawn Control Joint



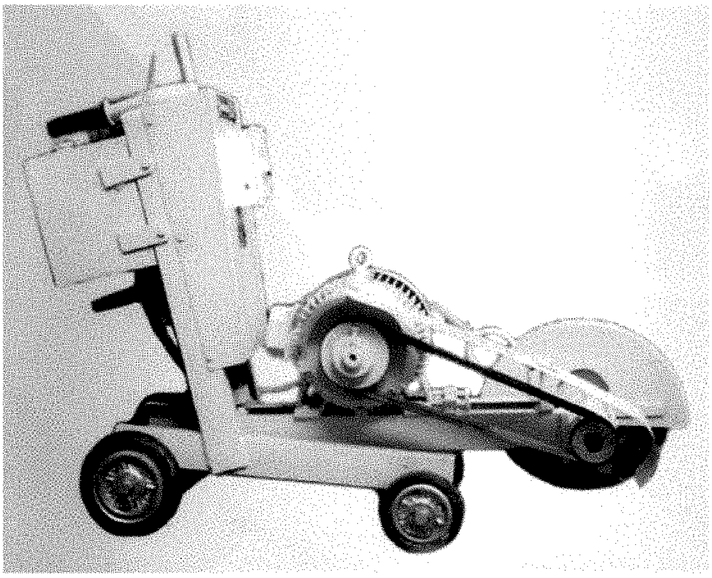
Sawn Control Joint with Dowels



Control Joints (Alternate Detail)

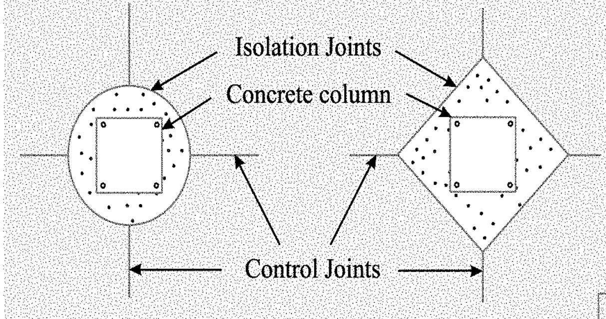
Fig. 9.12. Control Joints (or Shrinkage Joints or Dummy Joints or Contraction Joints)

down into the fresh concrete and then removed when the concrete is hardened. Thirdly, the contraction joint of stipulated width and depth is cut by employing a joint sawing machine. Sawing the joint is a very common practice in the recent time. Sawing is done as soon as the concrete is strong enough to resist tearing or other damage by the blade. Normally sawing is performed within about 24 hrs of finishing. Saw cut can also be done after seven days or more in which case depth of cut could be 1/3 the thickness of slab. Due to the standard width of diamond blades manufactured world wide, a minimum width of 3 mm to 4 mm would be sufficient. Wider width cuts are unnecessary and will lead to higher cutting and sealing costs. It is necessary that the groove made or cut should be filled up with joint sealing compound to improve the riding quality, to protect the edges of the concrete and also to prevent water from being held and subsequent ingress of moisture. The depth of the joint should be about $\frac{1}{4}$ the thickness of slab., Refer Fig. 9.12.



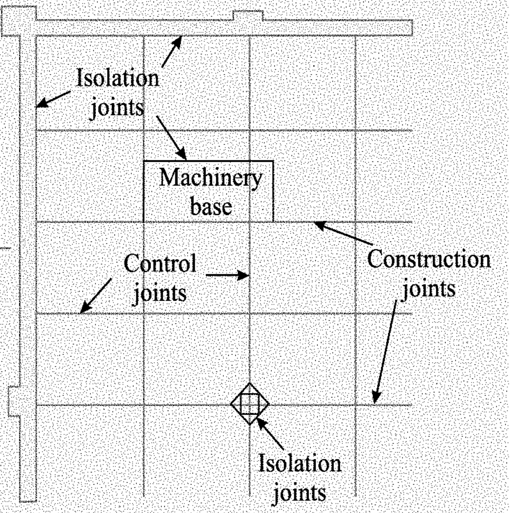
Concrete Joints Cutter

Joints are sawn within 6 to 18 hours before the contraction phase of the concrete commences.



Isolation Joints Around Concrete Columns

Fig. 9.13.



Typical Joint Layout for Concrete Floor on Ground

Fig. 9.14.

Spacing of Contraction Joints

Contraction joints are generally spaced 4.5 to 5.0 m intervals in unreinforced slabs. The spacing could be increased approximately upto 15 m in reinforced slabs depending upon the amount of reinforcement. The IRC stipulations on contraction joint spacing, with and without reinforcement, are given in table below:

Slab thickness (cm)	*Air-cured concrete Joints spacing (m)*	*Weight of reinforcement for reinforced slabs (kg/m²)*
a. Unreinforced Slabs		
10	4.5	-
15	4.5	-
20	4.5	-
b. Reinforced slabs		
10	7.5	2.2
15	13.0	2.7
20	14.0	3.8
25 and above	17.0	5.5

Spacing of Contraction Joints

The spacing of expansion joints, has been a matter of discussion because of varied practices and ranges from 20 meters to a few hundred meters. Recommended expansion joint spacing for Indian temperature conditions are given in table below:

Period of construction	*Degree of roughness of sub grade / sub base*	*Maximum expansion joint spacing (m) Slab thickness*		
		20 cm	*30 cm*	*40 cm*
Winter	Smooth	50	50	60
(Oct – March)	Rough	140	140	140
Summer	Smooth	90	90	120
(April – Sept)	Rough	140	140	140

In residential flooring the conventional contraction joint is omitted by casting the slab in alternate bays, to allow for the complete plastic shrinkage and also for maximum extent of drying shrinkage. It is usual to place glass-strip or aluminium strip in between the bays to create discontinuity between adjacent bays to prevent the development of continuous cracks.

Isolation Joints: This joint, as the name indicates is provided where the concrete floor meets the permanent structural elements such as walls, columns, foundation blocks, machine foundations etc. Since the movements associated with these structural elements are different from those of the concrete floor, an isolation joints are provided between them. It is provided to full depth of the concrete floor. The width (the gap) of such joint is kept about 10 to 12 mm. To avoid ingress of moisture or other undesirable materials, these joints are filled with a resilient materials and topped with joint fillings compounds. Refer fig. 9.13. Typical joint layout for concrete floor on ground is shown in fig. 9.14

Concrete Subjected to High Temperature

Fire Resistance

Concrete, though not a refractory material, is incombustible and has good fire-resistant properties. Fire resistance of concrete structure is determined by three main factors—the capacity of the concrete itself to withstand heat and the subsequent action of water without losing strength unduly, without cracking or spalling; the conductivity of the concrete to heat; and coefficient of thermal expansion of concrete. In the case of reinforced concrete, the fire resistance is not only dependent upon the type of concrete but also on the thickness of cover to reinforcement, The fire introduces high temperature gradients and as a result of it, the surface layers tend to separate and spall off from the cooler interior. The heating of reinforcement aggravates the expansion both laterally and longitudinally of the reinforcement bars resulting in loss of bond and loss of strength of reinforcement.

The effect of increase in temperature on the strength of concrete is not much upto a temperature of about 250°C but above 300°C, definite loss of strength takes place. Hydrated hardened concrete contains a considerable proportion of free calcium hydroxide which loses its water above 400°C leaving calcium oxide. If this calcium oxide gets wetted or is exposed to moist air, rehydrates to calcium hydroxide accompanied by an expansion in volume. This expansion disrupts the concrete. Portland blast furnace slag cement is found to be more resistant to the action of fire in this regard.

In mortar and concrete, the aggregates undergo a progressive expansion on heating while the hydrated products of the set cement, beyond the point of maximum expansion, shrinks. These two opposing actions progressively weaken and crack the concrete. The various aggregates used differ considerably in their behaviour on heating. Quartz, the principal mineral in sand, granites and gravels expands steadily upto about 573°C. At this temperature it undergoes a sudden expansion of 0.85%. This expansion has a disruptive action on the stability of concrete. The fire resisting properties of concrete is least, if quartz is the predominant mineral in the aggregate.

The best fire resistant aggregates, amongst the igneous rocks are, the basalts and dolerites. Limestone expands steadily until temperature of about 900°C and then begins to

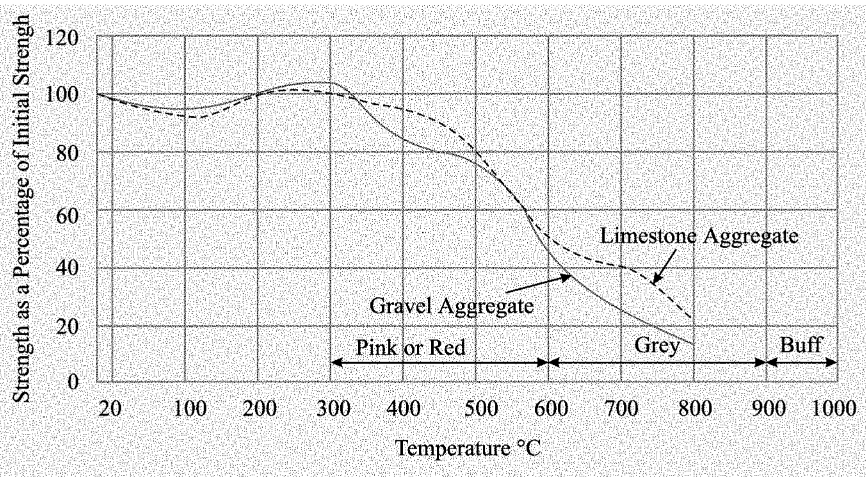


Fig. 9.15. Compressive strength of concrete after heating to high temperatures

contract owing to decomposition with liberation of carbon dioxide. Since the decomposition takes place only at a very high temperature of 900□C, it has been found that dense limestone is considered as a good fire resistant aggregate. Perhaps the best fire resistant aggregate is blast furnace slag aggregate. Broken bricks also form a good aggregate in respect of fire resistance. The long series of tests indicated that even the best fire resistant concretes have been found to fail if concrete is exposed for a considerable period to a temperature exceeding 900□C, while serious reduction in strength occurs at a temperature of about 600□C. Concrete does not show appreciable loss of strength upto a temperature of about 300□C. The loss of strength may be about 50% or more at about 500□C. Figures 9.15 and 9.16 show the effect of different temperatures on the strength of concrete and Fig. 9.17 shows the influence of temperature on the relative modulus of elasticity.

Freezing and Thawing

The lack of durability of concrete on account of freezing and thawing action of frost is not of great importance to Indian conditions. But it is of greatest considerations in most part of the world. However, certain regions in India, experience sub-zero temperatures in winter.

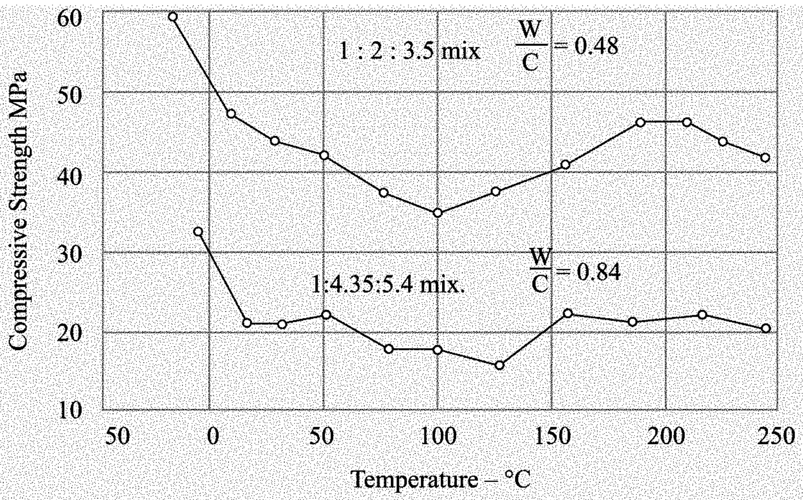


Fig. 9.16. Compressive strength of concrete after heating to different temperatures

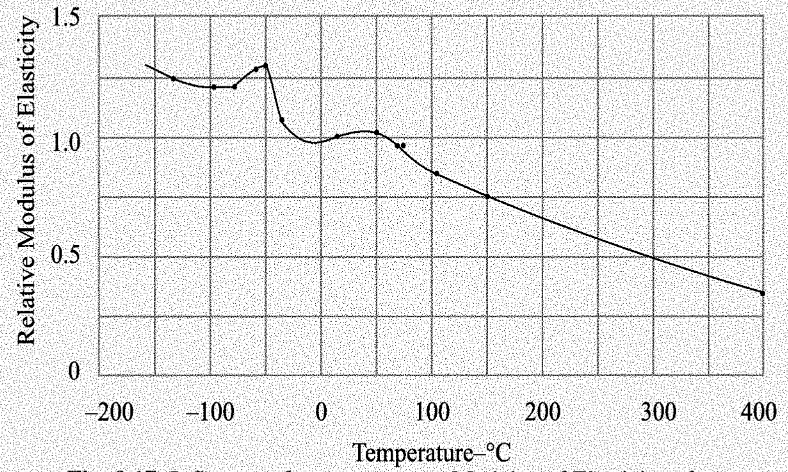


Fig. 9.17. Influence of temperature on Modulus of Elasticity of concrete

The concrete structures particularly, the one which are exposed to atmosphere are subjected to cycles of freezing and thawing and as such suffer from the damaging action of frost. The frost action is one of the most powerful weathering action on the durability of concrete. In the extreme conditions, the life span of concrete can be reduced to just a couple of years. The damage from freezing and thawing is most common and as such it is one of the extensively studied field on weathering of concrete in the United States of America, Russia and Northern European countries.

Though the durability of concrete is affected by alternative wetting and drying, heating and cooling, penetration and deposition of salt and other aggressive chemicals, leaching of calcium hydroxide, action of certain acids, alkali-aggregate reaction, mechanical wear and tear, abrasion and cavitation, one of the very important factors affecting the durability of concrete in the cold countries, is the action of frost. Therefore the aspect of frost resistance is of much importance and has been studied for more than 70 years.

It is very well known that fresh concrete should not be subjected to freezing temperature. Fresh concrete contains a considerable quantity of free water; if this free water is subjected to freezing temperature discrete ice lenses are formed. Water expands about 9% in volume during freezing. The formation of ice lenses formed in the body of fresh concrete disrupt the fresh concrete causing nearly permanent damage to concrete. The fresh concrete once subjected to forst action, will not recover the structural integrity, if later on allowed to harden at a temperature higher than the freezing temperature. Therefore, the fundamental point to note in dealing with cold weather concreting is that the temperature of the fresh concrete should be maintained above 0°C. The hardening concrete also should not be subjected to an extremely low temperature. It has been estimated that the freezing of water in the hardened concrete may exert a pressure of about 14 MPa. The strength of concrete should be more than the stress to which it is subjected at any point of time to withstand the damaging action.

Concrete structures subjected to alternate cycles of freezing and thawing undergoes considerable loss of durability.

The fully hardened concrete is also vulnerable to forst damage, particularly to the effect of alternate cycles of freezing and thawing. The severest conditions for frost action arise when concrete has more than one face exposed to the weather and is in such a position that it remains wet for a long period. Examples are road kerbs, parapets, concrete members in hydraulic structures just above water level etc.

There are various explanations for frost damage. One of the theories attributes the damage directly to the empty space available being insufficient to accommodate the additional solid produced when the free water held in concrete freezes. The damage is related to the degree of saturation.

Another theory attributes the failure to the production of pressure due to the growth of ice lenses parallel to the surface of the concrete owing to the migration of water from capillaries where the freezing point is depressed. This is similar to the theory of forst heaving in soils. Yet another theory explains the failure to generation of water pressure within the capillary cavities as the ice crystals grow. This hydraulic pressure can only be delivered by flow of water in other spaces, since the ice formed on the surface seals the exterior and the pressure generated forces the water through the fine capillaries. The local pressure so generated eventually exceeds the tensile strength of the concrete and causes breakdown. In all these theories, the permeability, rate of absorption and degree of saturation of the concrete are all important factors. Freezing starts at the surface in the largest cavities and gradually extends to smaller cavities. Water contained in the gel pores are too small to get frozen till the temperature goes below—78□C. In practice no ice is formed in the gel pores. The resistance of concrete to frost action depends on the strength of the paste, water/cement ratio, type of aggregate used, age of concrete, duration and extent to which the concrete is subjected to freezing action. More than all these one of the main factors is the degree of saturation of concrete.

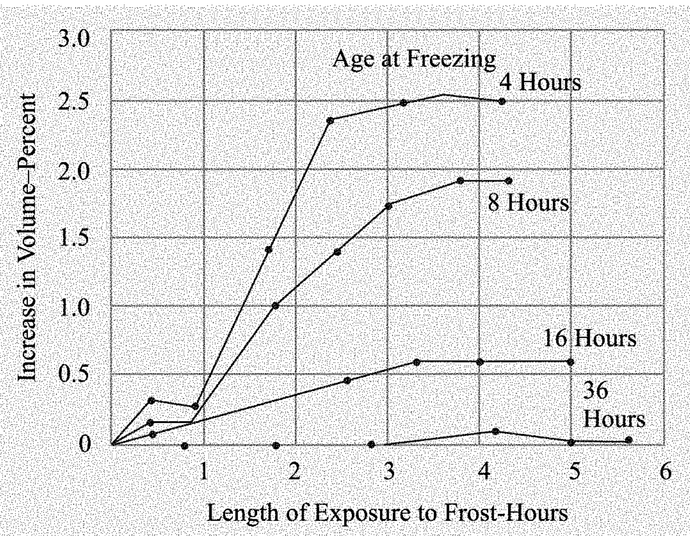


Fig. 9.18. Increase in volume of concrete during prolonged freezing as a function of age when freezing starts[9.16]

Figure 9.18 indicates the increase in volume to the length of exposure at the different ages and figure 9.19 shows the increase in volume with the number of cycles of freezing at different ages. The fully dry concrete is totally unaffected by frost action, but this is a theoretical statement; because when freezing takes place, naturally, the concrete becomes wet subsequently and loses durability. Figure 9.20 illustrates the influence of water/cement ratio on the frost resistance of concrete. The fine air bubbles entrained in the body of the concrete will act as a buffer to relieve the pressure created while freezing. The part of the water while getting frozen, runs into neighbouring air voids which are partially or fully empty. This relieves the pressure. Figure 9.20 shows the superiority of air entrained concrete with respect to freezing action.

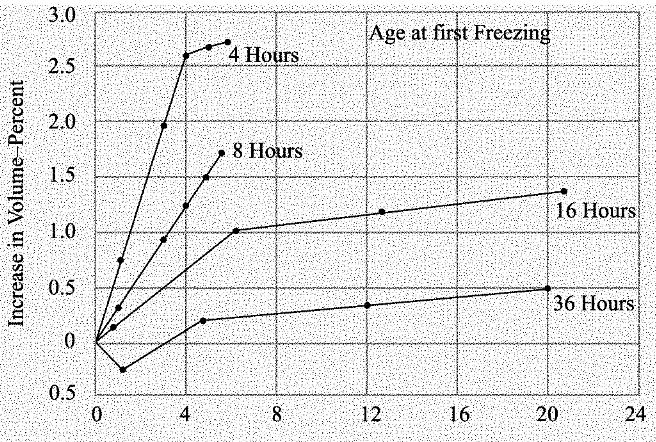


Fig. 9.19. Increase in volume of concrete subjected to freezing and thawing as a function of age at age when first freezing starts

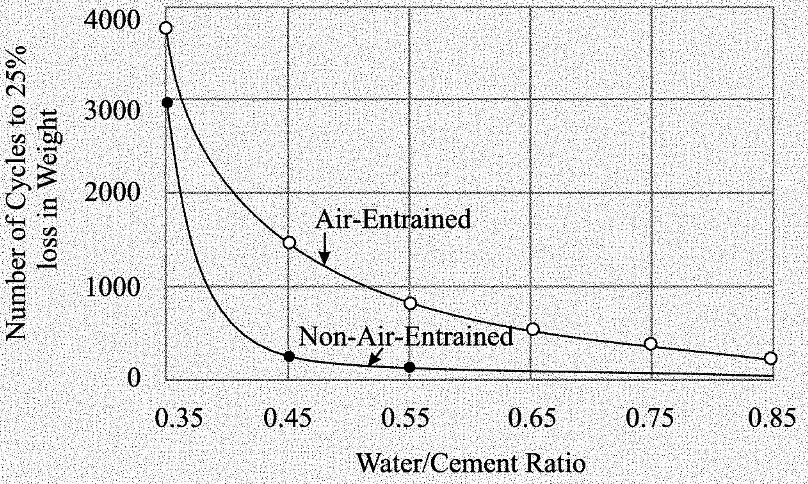


Fig. 9.20. Incluence of water/cement ratio on the resistance of concrete moist-cured for 28 days[9.16]

When the concrete is young, it contains more water and if such concrete is subjected to a low temperature greater quantity of water gets frozen and the total disruptive force is of a high order; whereas concrete at later ages contains less moisture and the freezing of such concrete will exert less total pressure. This is shown in Fig. 9.18

The frost damage can be assessed in several ways. Assessment of loss of weight of a sample of concrete subjected to a certain number of cycles of freezing and thawing is one of the methods. Measuring the change in the ultrasonic pulse-velocity or the change in the dynamic modulus of elasticity of specimen is another method. The resistance of the concrete to freezing and thawing is also measured by the durability factor. Blanks defined the durability factor as the "Number of cycles of freezing and thawing to produce failure divided by one hundred". ASTM method of calculating the durability factor is to continue freezing and thawing for 300 cycles or until the dynamic modulus of elasticity is reduced to 60% of its original value, whichever occurs first.

The durability factor =

There are no established criteria for acceptance or rejection of concrete in terms of the durability factor. Its value is primarily for comparison of different concretes, preferably when only one variable is changed. However, some guidance in interpretation can be obtained from the following.

A factor smaller than 40 indicates that the concrete is unsatisfactory with respect to resistance to freezing and thawing. 40 to 60 is the range for doubtful performance. Above 60, the concrete is probably satisfactory and value around 100 is considered satisfactory.

Deicing Effect of Salts

Deicing chemicals used for snow and ice clearance can cause and aggravate surface scaling. Studies have shown that formation of salt crystals in concrete may contribute to concrete scaling and deterioration layer by layer.

In cold regions in the winter, sodium chloride or calcium chloride is used for de-icing snow clearance on concrete roads. The repeated use of salts causes surface scaling of concrete roads. This has been attributed to the physical action of salt and not the chemical action. The use of air-entrainment makes the concrete road more resistance to surface scaling on account of salt action.

Moisture Movement

We discussed the topic moisture movement in chapter 8. We may recall that concrete shrinks when allowed to dry in air at low relative humidity and it swells when placed in water. Concrete members in outdoor conditions such as pavements, bridge decks, transmission poles, water tanks, swimming pools etc are subjected to alternate wetting and drying conditions and therefore undergoes expansion and shrinkage.

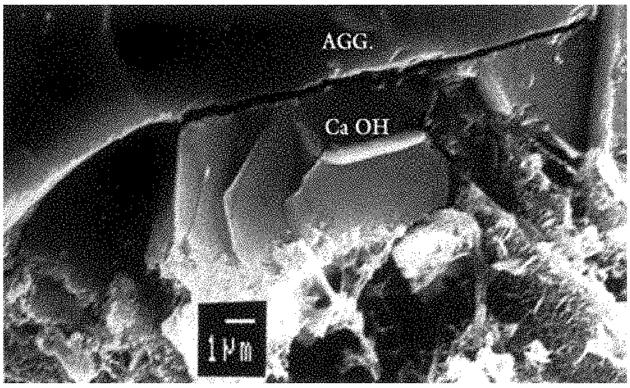
$$\frac{\text{Number of cycles at the end of test} \times \text{Percentage of original modulus}}{300}$$

The exposure of concrete to repetitive expansion and shrinkage or repetitive compressive stress and tensile stress which may cause fatigue in concrete and affect the durability of concrete.

It is a common experience that swimming pools which are kept dry for some times for repairs or such other reasons, develops cracks and leaks.

Transition Zone

We have dealt with the topic transition zone in some detail under the topic of strength in chapter 7. Now, when we are dealing with cracks and durability of concrete, it is necessary to touch upon this as it is of fundamental nature in inducing micro cracks due to very many reasons. Micro cracks in transition zone is a strength limiting factor. Concrete is a brittle material which develops microcracks even before any load is applied.



Transition zone between aggregate and hydrated cement paste.

On account of the dissimilar material, lack of bond, higher W/C ratio, and bleeding water, the transition zone becomes the weakest link in concrete mass. Under load, microcracks propagate further starting from largest microcracks. At a stress level of 70 per cent of the ultimate strength, the mortar matrix develops small cracks. With increasing stress level, the matrix cracks gradually and spreads throughout the mass. The microcracks in the transition zone at the interface with steel reinforcement becomes more permeable and admits air and water to promote corrosion of steel reinforcement. Incidentally these microcracks increases the depth of carbonation also .

Generally speaking, the microcracking at the transition zone is a general feature of concrete which is fundamentally responsible for reducing the long term durability of concrete.

Biological Process

It is a common site that in many buildings plants grow and the roots slowly penetrate into concrete or small cracks in concrete and converts it into bigger cracks with further growth. Even small plants such as lichen, algae and mass growing on concrete surface attract moisture and encourage physical and chemical process to deteriorate the concrete. Besides, humic acid produced by micro-growth reacts with cement.

In tropical countries, the concrete sewers carrying sewage, produce hydrogen sulphide (H_2S) due to anerobic decomposition of sulphur compounds. Hydrogen sulphide gets oxidised by aerobic bacteria producing sulphuric acid. The sulphuric acid attack the concrete above the liquid level on the crown portion of concrete sewer. In this way progressive deterioration of concrete takes place.

Marine borers and marine plants also contribute to the deterioration of concrete. Sometimes in tropical conditions algae, fungi and bacteria use atmospheric nitrogen to form nitric acid which attack concrete.

Structural Design Difficiencies

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

Innumerable examples can be cited such as conjestion of reinforcement and difficulties in proper compacting concrete, particularly at the column and beam junctions, deep beams, the negative reinforcement over T and L beams, should be taken care. In the absence of such care concrete is sure to crack.

In certain structures the ultimate creep deformation must be considered, otherwise more than the permissible deflection due to excess creep and unacceptable width of cracks will affect durability.

The permissible width of crack depends upon the functions of the structural members and on the exposure conditions of the concrete. Reis et al suggest the following permissible crack widths.[9.17]

Interior members	0.35 mm
Exterior members under normal exposure conditions	0.25 mm
Exterior members exposed to aggressive environment	0.15 mm

Chemical Action

When we are dealing with the durability of concrete, chemical attack which results in volume change, cracking of concrete and the consequent deterioration of concrete becomes an important part of discussion.

Under chemical attack, we shall discuss about sulphate attack, alkali-aggregate reaction, carbonation, deicing effect of salt, acid attack and effect of sea water.

Sulphate Attack

Most soils contain some sulphate in the form of calcium, sodium, potassium and magnesium. They occur in soil or ground water. Because of solubility of calcium sulphate is low, ground waters contain more of other sulphates and less of calcium sulphate. Ammonium sulphate is frequently present in agricultural soil and water from the use of fertilizers or from sewage and industrial effluents. Decay of organic matters in marshy land, shallow lakes often leads to the formation of H_2S, which can be transformed into sulphuric acid by bacterial action. Water used in concrete cooling towers can also be a potential source of sulphate attack on concrete. Therefore sulphate attack is a common occurrence in natural or industrial situations.

Solid sulphates do not attack the concrete severely but when the chemicals are in solution, they find entry into porous concrete and react with the hydrated cement products. Of all the sulphates, magnesium sulphate causes maximum damage to concrete. A characteristic whitish appearance is the indication of sulphate attack.

The term sulphate attack denote an increase in the volume of cement paste in concrete or mortar due to the chemical action between the products of hydration of cement and solution containing sulphates. In the hardened concrete, calcium aluminate hydrate (C-A-H) can react with sulphate salt from outside. The product of reaction is calcium sulphoaluminate, forming within the framework of hydrated cement paste. Because of the increase in volume of the solid phase which can go up to 227 per cent, a gradual disintegration of concrete takes place.

The reactions of the various sulphates with hardened cement paste is shown below

Let us take the example of Sodium Sulphate attacking $Ca(OH)_2$

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

The reaction with calcium aluminate hydrate is as follows

$$2(3CaO \,.\, Al_2O_3 \,.\, 12H_2O) + 3(Na_2SO_4 \,.\, 10H_2O)$$

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

Calcium sulphate attacks only calcium aluminate hydrate producing calcium sulpho aluminate ($3CaO \,.\, Al_2O_3 \,.\, 3CaSO_4 \,.\, 32H_2O$) known as ettringite. Molecules of water may be 32 or 31.

On the other hand magnesium sulphate has a more far reaching action than other sulphates because it reacts not only with calcium hydroxide and hydrated calcium aluminates like other sulphates but also decomposes the hydrated calcium silicates completely and makes it a friable mass.

The rate of sulphate attack increases with the increase in the strength of solution. A saturated solution of magnesium sulphate can cause serious damage to concrete with higher water cement ratio in a short time. However, if the concrete is made with low water cement ratio, the concrete can withstand the action of magnesium sulphate for 2 or 3 years. The concentration of sulphates is expressed as the number of parts by weight of SO_3 per million parts. 1000 PPM is considered moderately severe and 2000 PPM is considered very severe, especially if $MgSO_4$ is the predominant constituent.

Another factor influencing the rate of attack is the speed in which the sulphate gone into the reaction is replenished. For this it can be seen that when the concrete is subjected to the pressure of sulphate bearing water on one side the rate of attack is highest. Similarly, alternate wetting and drying due to tidal variation or spraying leads to rapid attack.

Methods of Controlling Sulphate Attack

Having studied the mechanism of sulphate attack on concrete it will be easy for us to deal with the methods for controlling the sulphate attack.

(*a*) Use of Sulphate Resisting Cement

The most efficient method of resisting the sulphate attack is to use cement with the low C_3A content. This has been discussed in detail earlier in chapter I. In general, it has been found that a C_3A content of 7% gives a rough division between cements of good and poor performance in sulphate waters.

(*b*) Quality Concrete

A well designed, placed and compacted concrete which is dense and impermeable exhibits a higher resistance to sulphate attack. Similarly, a concrete with low water/cement ratio also demonstrates a higher resistance to sulphate attack.

(*c*) Use of air-entrainment

Use of air-entrainment to the extent of about 6% (six per cent) has beneficial effect on the sulphate resisting qualities of concrete. The beneficial effect is possibly due to reduction of segregation, improvement in workability, reduction in bleeding and in general better impermeability of concrete.

(*d*) Use of pozzolana

Incorporation of or replacing a part of cement by a pozzolanic material reduces the sulphate attack. Admixing of pozzolana converts the leachable calcium hydroxide into insoluble non-leachable cementitious product. This pozzolanic action is responsible for impermeability of concrete. Secondly, the removal of calcium hydroxide reduces the susceptibility of concrete to attack by magnesium sulphate.

(*e*) High Pressure Steam Curing

High pressure steam curing improve the resistance of concrete to sulphate attack. This improvement is due to the change of C_3AH_6 into a less reactive phase and also to the removal or reduction of calcium hydroxide by the reaction of silica which is invariably mixed when high pressure steam curing method is adopted.

(*f*) Use of High Alumina Cement

The cause of great resistance shown by high alumina cement to the action of sulphate is still not fully understood. However, it is attributed in part to the absence of any free calcium hydroxide in the set cement, in contrast to Portland cement. High alumina cement contains approximately 40% alumina, a compound very susceptible to sulphate attack, when in normal portland cement. But this percentage of alumina present in high alumina cement behaves in a different way. The primary cause of resistance is attributed to formation of protective films which inhibit the penetration or diffusion of sulphate ions into the interior. It should be remembered that high alumina cement may not show higher resistance to sulphate attack at higher temperature.

A comprehensive study of concrete exposed to natural sulphate soils and to pure sulphate solution in the laboratory for periods ranging upto 25 years, was reported by Miller and Manson. The conclusions derived from this extensive study is given below.

(*a*) There was a definite correlation between the sulphate resistance of Portland cement and the amount of tricalcium aluminate (C_3A) it contained. High resistance was found for Portland cements containing not more than 5.5 per cent C_3A.

(*b*) There was no indication that the finer grinding of the cements had any influence on sulphate resistance.

(*c*) The resistance of seven Portland-pozzolana cements varied over nearly as wide a range as was observed for the 122 Portland cements.

(*d*) Four calcium aluminate cements (Cement Fondu or similar-non Portland cement) consistently showed a very high resistance to the sulphate bearing water. There was however, some indication that these cements are not completely stable at temperatures above 21 to 38°C.

(*e*) Specimens cured in steam at temperatures of 100°C and especially at 176°C were highly resistant. The degree of improvement was greatest for those cements originally not highly resistant, that is, those with relatively high C_3A.

(*f*) Few of the 40 admixtures tried gave markedly improved resistance; many had no effect and some were deleterious. The most effective were linseed, soyabean, and tung oils.

A large scale study of resistance of concrete to sulphate soils formed a part of the long-time study of cement performance in concrete. This was carried out by the Portland Cement Association (USA) under the general supervision of an Advisory committee. About 1000 concrete beams of size 15 x 13 x 86 cm were embedded horizontally to half their 15 cm depth in soils containing about 10 per cent soluble sulphates. For half of the specimens the sulphate was principally sodium sulphate. For the other half, 2/3 number of specimen it was sodium sulphate and 1/3 magnesium sulphate. The soil in each basin was alternately made wet and dry. The prevailing temperature was above 0°C.

Twenty-seven different Portland cements including all five ASTM type, were used in three concrete mixtures containing cement 223, 307, 390 kg/m^3.

A report of results to 20 years was published in 1965. With respect to sulphate attack, the following conclusions were drawn. [9.18]

(*a*) The resistance of concrete to attack by solutions of sulphate salts increases with reduction of C_3A content in the cement. At 6 years, a C_3A content of 7% as calculated without correction for minor oxides provided a good separation between cements of good and poor sulphates resistance. After 20 years, it was concluded that a C_3A content of 5.5 per cent as corrected for minor oxides and about 3.5 per cent as determined by X-ray analysis, were fairly good values for separating superior and poor resistance in the richest mix. It is interesting to note that the 10 year report (1953) observed that beams in the soil containing $MgSO_4$ as well Na_2SO_4 were less attacked than those in the soil containing mainly Na_2SO_4. This result is contrary to expectations based on some studies conducted with concretes and mortars continually immersed in sulphate solutions. It was tentatively ascribed to differences in the nature of the salt deposit on the beams resulting from evaporation.

Table 9.11. Requirements for Concrete Exposed to Sulphate Attack. As per IS 456 : 2000

Sl.	Class	Concentration of Sulphates, Expressed as SO_3			Type of Cement	Dense, Fully compacted Concrete. Made with 20 mm Nominal Maximum Size Aggregates Complying with IS 383	
		In Soil					
		Total SO_3 Percent	SO_3 in 2 : 1 Water: Soil Extract g/l	In Ground Water g/l		Minimum Cement Content kg/m^3	Maximum Free Water/ Cement Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
(i)	1	Traces (< 0.2)	Less than 1.0	Less than 0.3	Ordinary Portland cement or Portland slag cement or Portland pozzolana cement	280	0.55
(ii)	2	0.2 to 0.5	1.0 to 1.9	0.3 to 1.2	Ordinary Portland cement or Portland slag cement or Portland pozzolana cement	330	0.50
					Supersulphated cement or sulphate resisting Portland cement	310	0.50
(iii)	3	0.5 to 1.0	1.9 to 3.1	1.2 to 2.5	Supersulphated cement or sulphate resisting Portland cement	330	0.50
					Portland pozzolana cement or Portland slag cement	350	0.45

Table 9.11. (Contd.)

(iv)	4	1.0 to 2.0	3.1 to 5.0	2.5 to 5.0	Supersulphated or sulphate resisting Portland cement	370	0.45
(v)	5	More than 2.0	More than 5.0	Morethan 5.0	Sulphate resisting Portland cement or supersulphated cement with protective coating.	400	0.40

Notes

1. Cement content given in this table is irrespective of grades of cement.
2. Use of supersulphated cement is generally restricted where the prevailing temperature is above 40°C.
3. Supersulphated cement gives an acceptable life provided that the concrete is dense and prepared with a water-cement ratio of 0.4 or less, in mineral acids, down to *p*H 3.5.
4. The cement contents given in col 7 of this table are the minimum recommended. For SO_3 contents near the upper limit of any class, cement contents above these minimum are advised.
5. 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.
6. Portland slag cement conforming to IS 455 with slag content more than 50 percent exhibits better sulphate resisting properties.
7. Where chloride is encountered along with sulphates in soil or ground water, ordinary Portland cement with C_3A 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.

(*b*) Air entrainment improved the performance of almost all of the specimens exposed to alternate drying and soaking in sulphate soils.

(*c*) Influence of cement content of concrete (and accompanying change in water-cement ratio) was highly significant. For the richest mix, attack was slow, at 6 years little difference in resistance between cements could be seen. For the intermediate and lean mixes, the attack was more rapid, and important differences related to cement composition, especially C_3A content, were clearly evident at that time.

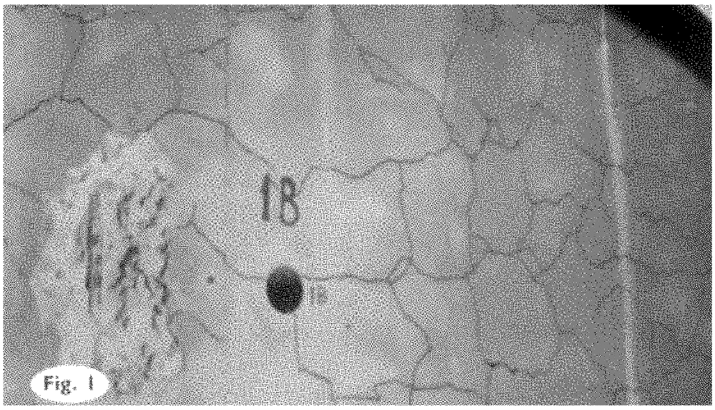
Where hardened Portland cement concrete is exposed to soil or ground water containing sulphate compounds, it is necessary to limit the permeability of the concrete. It is also recommended that with the higher sulphate concentration it is necessary to use cement with higher resistance to sulphates, higher cement content and lower water/cement ratio. IS 456 of 2000 gives the recommendations for the type of cement, maximum free W/C ratio and minimum cement content, which are required at different sulphate concentrations in near-neutral ground water having pH of 6 to 9. Table 9.11 shows the requirements for concrete exposed to sulphate attack. For very high sulphate concentrations class 5 in the table 9.11, some form of lining such as polyethylene or polychloroprene sheet, or surface coating based on asphalt, chlorinated rubber, expoxy, or polyurethene materials should be used to prevent access by the sulphate solution. IS 456 of 2000 also stipulates the sulphates in concrete in the following way.

Sulphates are present in most cements and in some aggregates: excessive amount of water-soluble sulphate from these or other mix constituents can cause expansion and disruption of concrete. To prevent this, the total water-soluble sulphate content of the concrete mix, expressed as SO_3, should not exceed 4 per cent 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 per cent limit does not apply to concrete made with supersulphated cement complying with IS 6909.

Alkali-Aggregate Reaction

Detailed discussion has been done on alkali-aggregate reaction in chapter 3 under aggregate and testing of aggregates. Alkali-aggregate reaction (AAR) is basically a chemical reaction between the hydroxyl ions in the pore water within concrete and certain types of rock minerals which sometimes occur as part of aggregates. Since reactive silica in the aggregate is involved in this chemical reaction it is often called alkali-silica reaction (ASR). Since the first paper published by Stantan during 1940's on this subject, a considerable studies have been made and now it is recognised as one of the major causes of cracking of concrete. Primarily the reaction produces what is called alkali-silica gel of unlimited swelling type under favourable conditions of moisture and temperature,



Typical map cracking due to Alkali-Aggregate reaction in voids and cracks and further it causes disruption and pattern cracking. The crack width can range from 0.1 mm to as much as 10 mm.

What was seen as a rare case in 1940's have been recognised now as one of the general occurrence in present day concrete to a greater or smaller magnitude. Aggregates used in large concrete construction should be suitably tested to detect tendency for alkali-aggregate reaction.

In the construction of nuclear power project at Kaiga initially they did not investigate the quality of aggregate. Later on they suspected the aggregate and as a remedial measure, they went in for low alkali cement having alkali content of less than 0.4.

As the concrete technologists are now more conscious about AAR, the cement manufactures are more careful about alkali content (K_2O and Na_2O) or what is called soda equivalent. This is calculated as the actual Na_2O content plus 0.658 times the K_2O content of the clinker. It should be less than 0.6 per cent by mass of cement. Alkali content of 0.6 could be considered as a threshold point of high alkali cement.

It is to be pointed out that alkali-silica reaction takes place only at high concentrations of OH^-, that is at high pH value in the pore water. The pH of the pore water depends on the alkali content of cement. Heigh alkali cement may lead to a pH of about 13.5 to 13.9 and low alkali cement results in a pH of about 12.7 to 13.1. An increase in pH of 1.0 represents a ten fold increase in hydrogen ion concentration. Therefore low alkali cement which produces low pH value in the pore water is safe against potentially reactive aggregate.

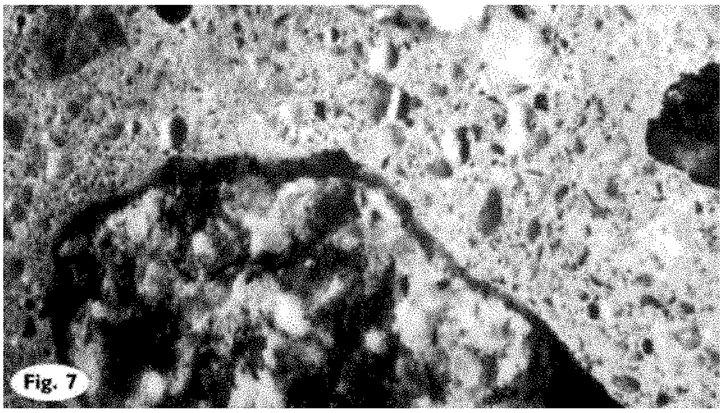


Fig. 7

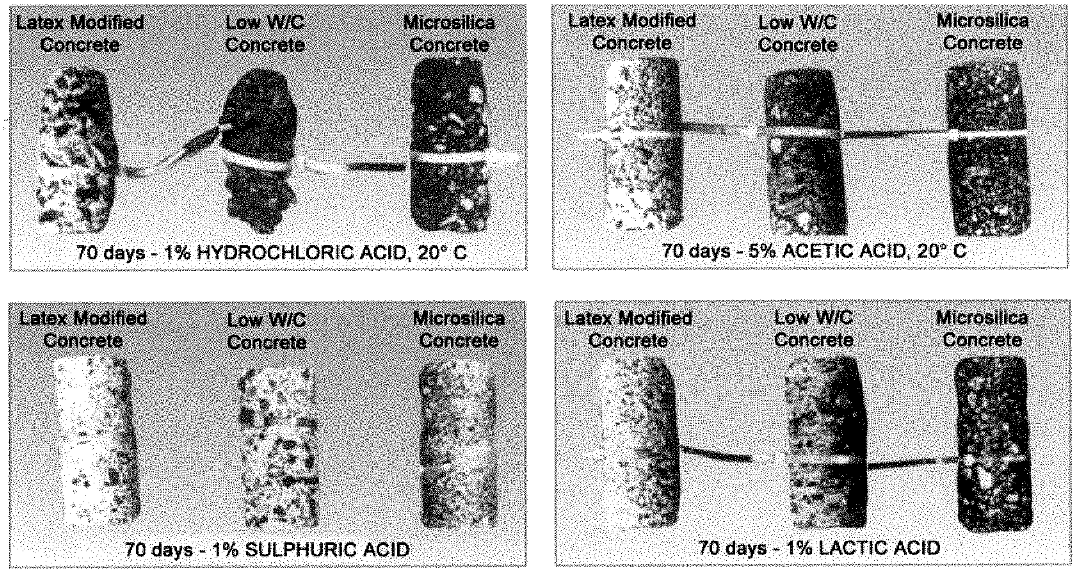
Dark reaction rim on aggregate border

Alkalis not only comes from cement but also comes from sand containing sodium chloride, admixtures, mixing water, sea water penetration, fly ash, blast furnace slag and deicing salt getting into concrete. Alkalis from all these sources must be included in finding the total alkalis. British standard 5328 : part 1 : 1091 specifies a maximum of 3.0 kg of alkalis (expressed as soda equivalent) in 1 m^3 of concrete in case of alkali reactive aggregates are used.

Acid Attack

Concrete is not fully resistant to acids. Most acid solutions will slowly or rapidly disintegrate portland cement concrete depending upon the type and concentration of acid. Certain acids, such as oxalic acid and phosphoric acids are harmless. The most vulnerable part of the cement hydrate is $Ca(OH)_2$, but C-S-H gel can also be attacked. Silicious aggregates are more resistant than calcareous aggregates.

Concrete can be attacked by liquids with pH value less than 6.5. But the attack is severe only at a pH value below 5.5. At a pH value below 4.5, the attack is very severe. As the attack



Relative acids resistance of concrete

Courtesy : P.K. Mehta

proceeds, all the cement compounds are eventually broken down and leached away, together with any carbonate aggregate material. With the sulphuric acid attack, calcium sulphate formed can proceed to react with calcium aluminate phase in cement to form calcium sulphoaluminate, which on crystallisation can cause expansion and disruption of concrete.

If acids or salt solutions are able to reach the reinforcing steel through cracks or porosity of concrete, corrosion can occur which will cause cracking.

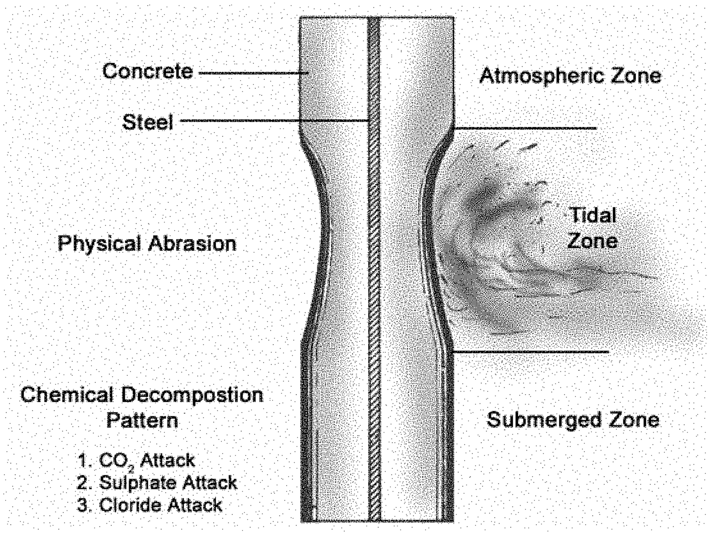
Concrete in Sea Water

Large number of concrete structures are exposed to sea water either directly or indirectly. For several reasons, effect of sea water on concrete deserves special attention. The coastal and offshore structures are exposed to simultaneous action of a number of physical and chemical deterioration process. The concrete in sea water is subjected to chloride induced corrosion of steel, freezing and thawing, salt weathering, abrasion by sand held in water and other floating bodies.

Sea water generally contains 3.5 per cent of salt by weight. The ionic concentration of Na^+ and Cl^- are the highest, typically 11,000 and 20,000 mg/litre respectively. It also contains Mg^{2+} and SO_4^{2-}, typically 1400 and 2700 mg/litre respectively. The PH of sea water varies between 7.5 and 8.4. The average value is 8.2. Sea water also contains some amount of CO_2.

We have already seen earlier in this chapter that magnesium sulphate reacts with free calcium hydroxide in set Portland cement to form calcium sulphate, at the same time precipitating magnesium hydroxide. $MgSO_4$ also reacts with the hydrated calcium aluminate to form calcium sulpho aluminate. These have often been assumed to be the actions primarily responsible for the chemical attack of concrete by sea water.

It is commonly observed that deterioration of concrete in sea water is often not characterised by the expansion found in concrete exposed to sulphate action, but takes more the form of erosion or loss of constituents from the parent mass without exhibiting undue expansion. It is inferred that the presence of chlorides in sea water may have retarded the swelling of concrete in sulphate solution. It is also found that concrete will have lost some part of lime content due to leaching. Both calcium hydroxide and calcium sulphate are considerably more soluble in sea water and this, will result in increased leaching action. To put it briefly, concrete undergoes several reactions concurrently when subjected to sea water. A concrete of not too massive dimensions exposed to sea water is more likely to show the effects of leaching than expansion, whereas massive structures like dock walls etc. may show the effects of expansion also. The rate of chemical attack is increased in temperate zones.



Diagrammatic representation of deterioration of concrete exposed to seawater.
Courtesy : P.K. Mehta

Experience has shown that most severe attack of sea water on concrete occurs just above the level of high water. The portion between low and high water marks is less affected and the parts below the water level which are continuously remain immersed are least affected. The crystallisation of salt in the portion of concrete above high water level is responsible for disruption of concrete. In place of cold climatic region, the freezing of water in pores at the spray level of concrete is responsible for causing lack of durability in concrete. Freezing of water may also take place between the tidal variation level.

It is to be admitted that concrete is not 100% impervious. The water that permeates into the concrete causes corrosion of steel. The product of corrosion being of higher volume than the material they replace, exert pressure which results in lack of durability to reinforced concrete. It is also seen that the lack off durability is more in case of reinforced concrete than the identical plain concrete.

Sea water holds certain quantity of sand and silt particularly in the shallow end. The velocity of wave action causes abrasion of concrete. The impact and the mechanical force of wave action also contributes to the lack of durability of concrete.

From the foregoing discussion it will be easy to formulate steps to improve the durability of concrete in sea water. Apart from the right type of cement with low C_3A content, the other factor to be considered is the use of rich concrete with low water/cement ratio. The rich concrete with low water/cement ratio mainly makes the concrete impervious to the attack of sea water, and also having very little capillary pores does not hold water, to cause expansion either by freezing or by crystallisation of salt. Provision of adequate cover is another desirable step for increasing durability of reinforced concrete. Use of pozzolanic material is yet another desirable step that could be taken to improve durability against sea water. A good compaction, well made construction joints etc. are other points helping the durability of concrete in sea water. Whenever possible, high pressure steam-cured prefabricated concrete elements should be used for better durability.

Carbonation

Carbonation of concrete is a process by which carbon dioxide from the air penetrates into concrete and reacts with calcium hydroxide to form calcium carbonates. We have seen earlier that the conversion of $Ca(OH)_2$ into $CaCO_3$ by the action of CO_2 results in a small shrinkage.

Now we shall see another aspect of carbonation. CO_2 by itself is not reactive. In the presence of moisture, CO_2 changes into dilute carbonic acid which attacks the concrete and also reduces alkalinity of concrete.

Air contains CO_2. The concentration of CO_2 in rural air may be about 0.03 per cent by volume. In large cities the content may go up to 0.3 per cent or exceptionally it may go up to even 1.0 per cent. In the tunnel, if not well ventilated the intensity may be much heigher.

The pH value of pore water in the hardened concrete is generally between 12.5 to 13.5 depending upon the alkali content of cement. The high alkalinity forms a thin passivating layer around steel reinforcement and protect it from action of oxygen and water. As long as steel is placed in a highly alkaline condition, it is not going to corrode. Such condition is known as passivation.

In actual practice CO_2 present in atmosphere in smaller or greater concentration, permeates into concrete and carbonates the concrete and reduces the alkalinity of concrete. The pH value of pore water in the hardened cement paste which was around 13 will be reduced to around 9.0. When all the $Ca(OH)_2$ has become carbonated, the pH value will reduce upto about 8.3. [9.19] In such a low pH value, the protective layer gets destroyed and the steel is exposed to corrosion.

The carbonation of concrete is one of the main reasons for corrosion of reinforcement. Of course, oxygen and moisture are the other components required for corrosion of embedded steel.

Rate of Carbonation: The rate of carbonation depends on the following factors.

- The level of pore water *i.e.*, relative humidity.
- Grade of concrete
- Permeability of concrete
- Whether the concrete is protected or not
- depth of cover
- Time

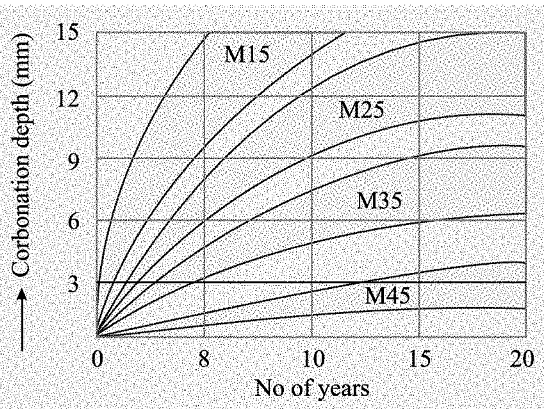


Fig. 9.21. Depth of carbonation with respect to strength (grade) of concrete

It is interesting to know that if pore is filled with water the diffusion of CO_2 is very slow. But whatever CO_2 is diffused into the concrete, is readily formed into dilute carbonic acid reduces the alkalinity.

On the other hand if the pores are rather dry, that is at low relative humidity the CO_2 remains in gaseous form and does not react with hydrated cement. The moisture penetration from external source is necessary to carbonate the concrete.

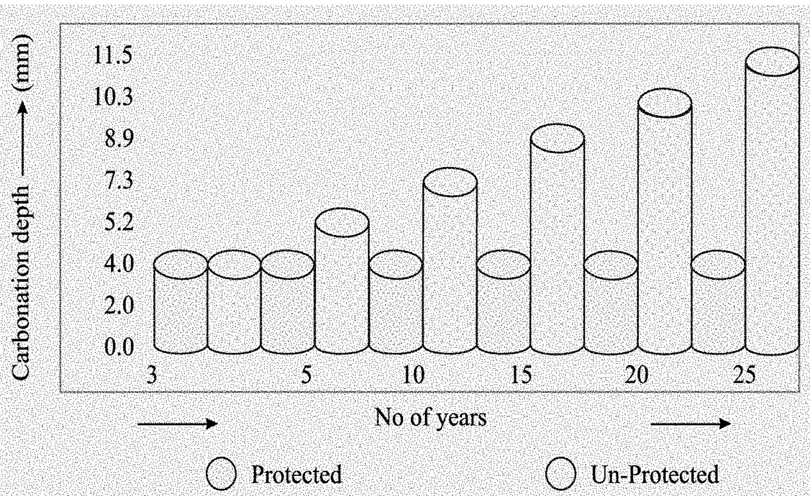


Fig. 9.22. Depth of carbonation for protected and unprotected concrete

Table 9.12. Depth of carbonation with age and grade of concrete.

Age-years	*Depth of Carbonation (mm)*	
	M 20	*M 40*
2	5.0	0.5
5	8.0	1.0
10	12.0	2.0
50	25.0	4.0

The highest rate of carbonation occurs at a relative humidity of between 50 and 70 per cent

The rate of carbonation depth will be slower in case of stronger concrete for the obvious reason that stronger concrete is much denser with lower W/C ratio. It again indicates that the permeability of the concrete, particularly that of skin concrete is much less at lower W/C and as such the diffusion of CO_2 does not take place faster, as in the case of more permeable concrete with higher W/C ratio. Fig. 9.21 and table 9.12 show the depth of carbonation in various grades of concretes.

It is now well recognised that concrete needs protection for longer durability. Protective coating is required to be given for long span bridge girders, flyovers, industrial structures and chimneys. The fig. 9.22 shows carbonation depth of protected and unprotected concrete.

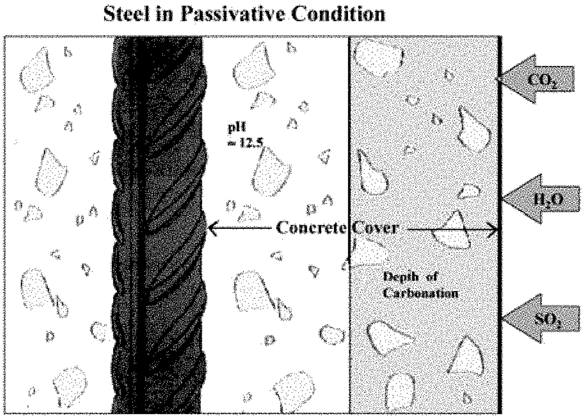
Depth of cover plays an important role in protecting the steel from carbonation. The table

9.13 shows relationships between W/C, depth of cover and time in years for carbonation depth to reach the reinforcement.

Table 9.13. Approximate relations between W/C, depth of cover and time in years for carbonation depth to reach the reinforcement.

W/C	*Depth of cover (mm)*			
ratio	*15*	*20*	*25*	*30*
0.45	100^+	100^+	100^+	100^+
0.50	56	99	100^+	100^+
0.55	27	49	76	100
0.60	16	29	45	65
0.65	13	23	36	52
0.70	11	19	30	43
	Time in years for carbonation			

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



Concrete is under continuous attack by aggressive environmental agencies. Good concrete and suffecient cover is the answer for durability.

Chloride Attack

Chloride attack is one of the most important aspects for consideration when we deal with the durability of concrete. Chloride attack is particularly important because it primarily causes corrosion of reinforcement. Statistics have indicated that over 40 per cent of failure of structures is due to corrosion of reinforcement.

We have already discussed that due to the high alkality of concrete a protective oxide film is present on the surface of steel reinforcement. The protective passivity layer can be lost due to carbonation. This protective layer also can be lost due to the presence of chloride in the presence of water and oxygen. In reality the action of chloride in inducing corrosion of reinforcement is more serious than any other reasons. One may recall that sulphates attack the concrete whereas the chloride attacks steel reinforcements.

Measurement of Depth of Carbonation

Pink colour indicates that $Ca(OH)_2$ is unaffected by carbonation. The uncoloured portion indicates that concrete is carbonated.

Chloride enters the concrete from cement, water, aggregate and sometimes from admixtures. The present day admixtures are generally contain negligible quantity of chloride or what they call chloride free. Chloride can enter the concrete by diffusion from environment. The Bureau of Indian Standard earlier specified the maximum chloride content in cement as 0.05 per cent. But it is now increased the allowable chloride content in cement to 0.1 per cent. IS 456 of 2000 limits the chloride content as (cl) in the concrete at the time of placing is shown in Table 9.14.

Table 9.14. Limits of Chloride Content of Concrete (IS 456 of 2000)

Sl. No	*Type or Use of Concrete*	*Maximum Total acid soluble chloride Content. Expressed as kg/m^3 of concrete*
1.	Concrete containing metal and steam cured at elevated temperature and prestressed concrete	0.4
2.	Reinforced concrete or plain concrete containing embedded metal	0.6
3.	Concrete not containing embedded metal or any material requiring protection from chloride	3.0

The amount of chloride required for initiating corrosion is partly dependent on the pH value of the pore water in concrete. At a pH value less than 11.5 corrosion may occur without the presence of chloride. At pH value greater than 11.5 a good amount of chloride is required.

Limiting values of chloride contents, above which corrosion may be imminent, for various values of pH are indicated in table 9.15. The total chloride in concrete is present partly as insoluble chlorialuminates and partly in soluble form. It is the soluble chloride, which is responsible for corrosion of reinforcement.

Table 9.15. Limiting Chloride Content Corresponding to pH of concrete[9.20]

pH	Chloride content g/litre	ppm
13.5	6.7400	6740
13.0	2.1300	2130
12.5	0.6720	672
12.0	0.2130	213
11.5	0.0670	67
11.0	0.0213	21
10.0	0.0021	2
9.02	0.0002	0.2

Chloride Permeability Based on Charge Passed (As per ASTM C 1202)

Chloride Permeality	Charges passed (Coulombs)	Type of Concrete
High	≤ 4000	High water-cement ratio, ≤ 0.6
Moderate	2000 to 4000	Moderate W/C ratio (0.4 to 0.5)
Low	1000 to 2000	Water-cement ratio ≤ 0.4
Very Low	100 to 1000	Latex modified concrete
Negligible	≤ 100	Polymer Impregnated concrete

Corrosion of Steel (Chloride induced)

Corrosion of steel in concrete is an electrochemical process. When there is a difference in electrical potential along the steel reinforcement in concrete, an electrochemical cell is set up. In the steel, one part becomes anode and other part becomes cathode connected by electrolyte in the form of pore water in the hardened cement paste. The positively charged ferrous ions Fe^{++} at the anode pass into solution while the negatively charged free electrons e^- pass through the steel into cathode where they are absorbed by the constituents of the electrolyte and combine with water and oxygen to form hydroxyl ions $(OH)^-$. These travel

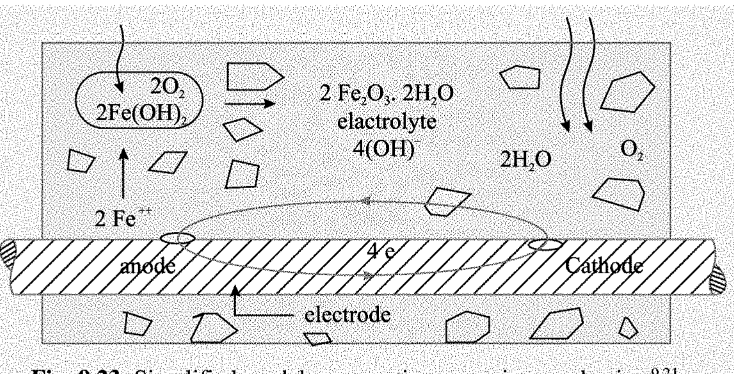


Fig. 9.23. Simplified model representing corrosion mechanism[9.21]

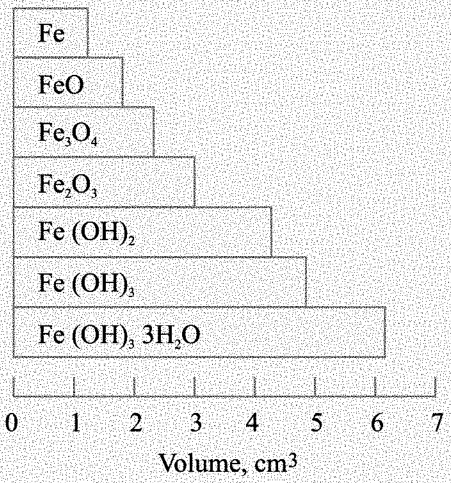


Fig. 9.24. Shows that, depending on the oxidation state, metallic iron can increase more than six times in volume 9.22

through the electrolyte and combine with the ferrous ions to form ferric hydroxide which is converted by further oxidation to rust. Refer Fig. 9.23.

The reactions are discribed below

Anodic reactions

$Fe \longrightarrow Fe^{++} + 2e^-$

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

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

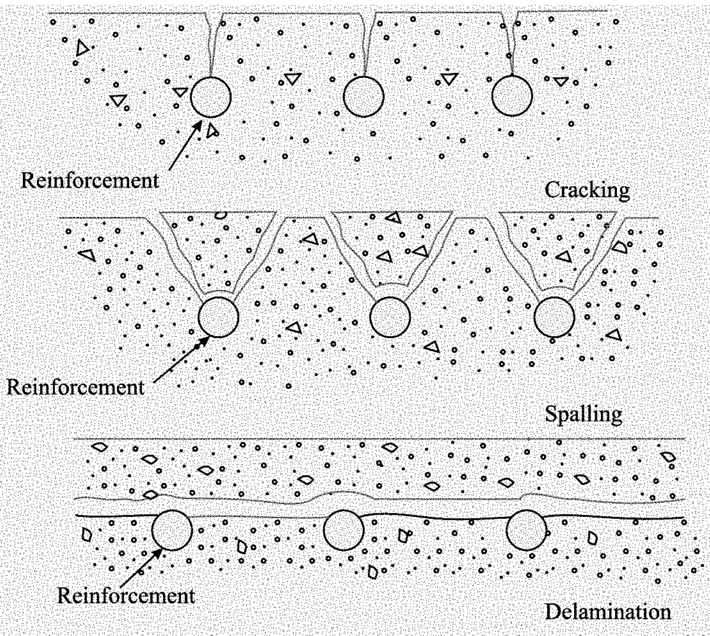


Fig. 9.25. Diagrammatic representation of damage induced by corrosion cracking spalling, and delamination

Cathodic reaction

$$4e^- + O_2 + H_2O \longrightarrow 4(OH)^-$$

It can be noted that no corrosion takes place if the concrete is dry or probably below relative humidity of 60 percent because enough water is not there to promote corrosion. It can also be noted that corrosion does not take place if concrete is fully immersed in water because diffusion of oxygen does not take place into the concrete. Probably the optimum relative humidity for corrosion is 70 to 80 per cent.

The products of corrosion occupy a volume as many as six times the original volume of steel depending upon the oxidation state. Fig. 9.24 shows the increase in volume of steel depending upon the oxidation state.

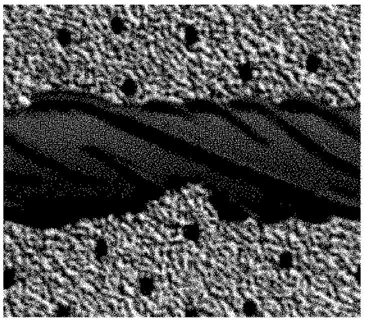
The increased volume of rust exerts thrust on cover concrete resulting in cracks, spalling or delamination of concrete. Refer Fig. 9.25. With this kind of situations concrete loses its integrity. The cross section of reinforcement progressively reduces and the structure is sure to collapse.

Corrosion Control

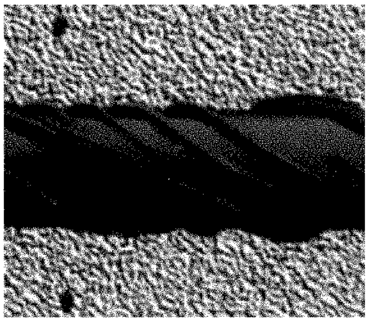
From the literature survey and case studies it has been reported that 40% of failure of structures is on account of corrosion of embedded steel reinforcement in concrete. Therefore corrosion control of steel reinforcement is a subject of paramount importance.

First and foremost for corrosion control is the good quality of concrete through good construction practices. It is a very vast subject touches the fundamentals of choosing constituent material and good rules to be followed during various stages of production of concrete. In particular the use of lowest possible water/cement ratio having regard to workability. In view of the general availability of superplasticizers, it should be used to cut down the W/C ratio to make dense concrete.

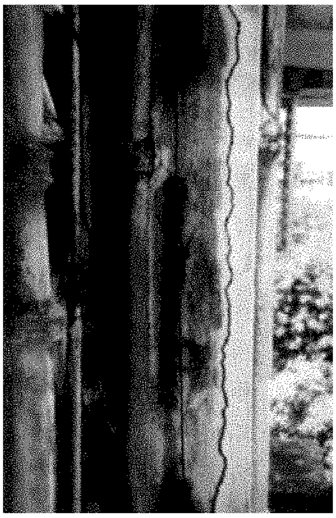
Proper mix design, use of right quality and quantity of cement for different exposure conditions is to be adopted. Recently it has been realised that lower W/C ratio which has been always associated with lower permeability is not enough to make impermeable concrete contributing to high durability. Use of supplementary cementitious materials such as fly ash, ground granulated blastfurnace slag (ggbs), silica fume etc. are required to be used as admixtures or in the form of blended cement in addition to lowest possible W/C ratio to make concrete dense. These materials improve more than one properties of concrete which will eventually reduce corrosion of reinforcement. Tests on mortar containing ggbs have shown that water permeability is reduced by a factor up to 100. It is also reported that 60 per cent ggbs reduced the diffusion of chloride ions into the concrete by as much as 10 times. Silica fume contributes to the all-round improvements in the quality of concrete which are responsible for reducing



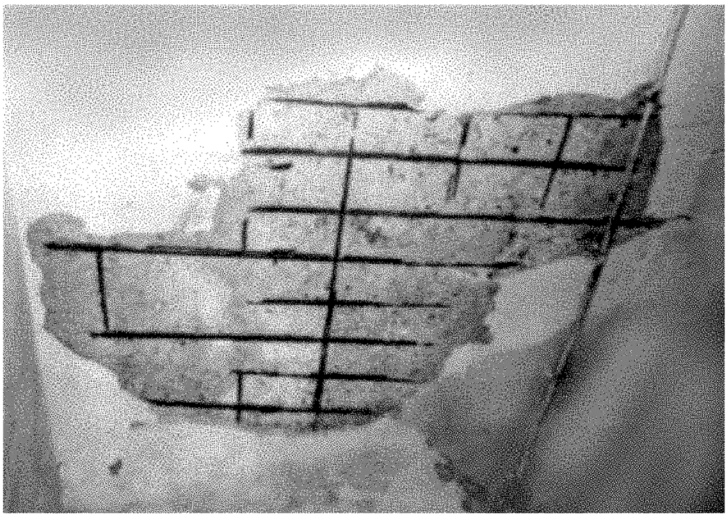
Electro Micrograph showing corrosion in poor concrete



Electro Micrograph showing no corrosion in good concrete



Crack formed due to bursting pressure on account of rusting of reinforcements



Example of delamination of concrete cover

corrosion of steel reinforcement. The improvement in the microstructure of hydrated cement paste is ultimately responsible for protecting the steel reinforcement from corrosion.

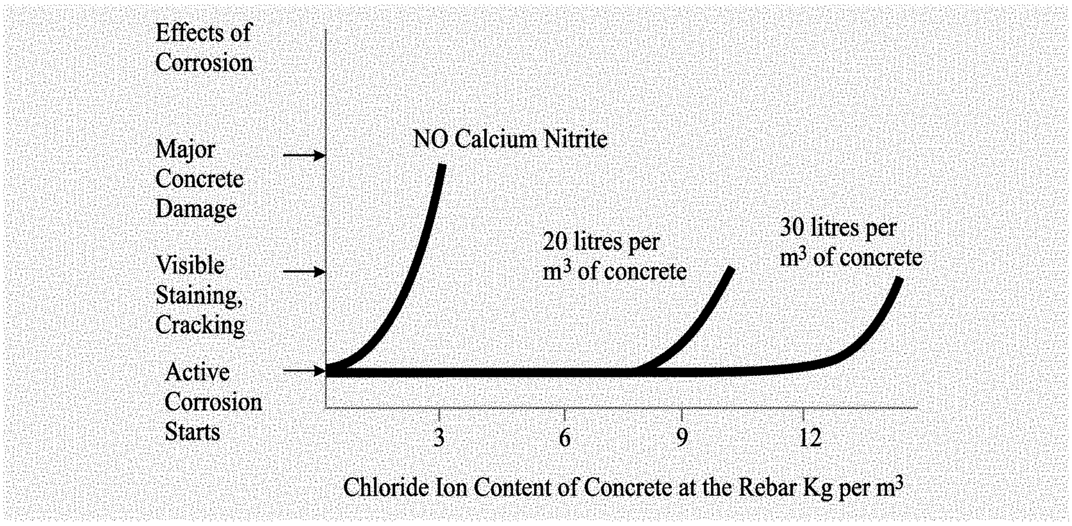
In short it can be said that if we make good concrete with low permeability and improved microstructure, it will be durable by itself and also it can take care of the reinforcement contained in it to a great extent. It is always not possible to make such ideal concrete, particularly, in view of the complex environmental and exposure conditions. Further, the inherent long term drying shrinkage and microcracks in concrete, the problems become more serious. This demands certain other measures to control the corrosion of steel reinforcement. They are listed and briefly explained.

- Metallurgical methods
- Corrosion inhibitors
- Coatings to reinforcement
- Cathodic protection
- Coatings to concrete
- Design and detailing

***Metallurgical Methods*:** Steel can be made more corrosion resistant by altering its structure through metallurgical processes. Different methods such as rapid quenching of the hot bars by series of water jets, or by keeping the hot steel bars for a short time in a water bath, and by such other process the mechanical properties and corrosion resistance property of steel can be improved. There are many situations where stainless steel reinforcements are used for long term durability of concrete structures.

***Corrosion inhibitors*:** Corrosion can be prevented or delayed by chemical method by using certain corrosion inhibiting chemicals such as nitrites, phosphates, benzoates etc. Of the available materials, the most widely used admixture is based on calcium nitrite. It is added to the concrete during mixing of concrete. The typical dosage is of the order of 10-30 litres per m^3 of concrete depending on chloride levels in concrete.

As mentioned earlier, in the high pH of concrete, the steel is protected by a passivating layer of ferric oxide on the surface of steel. However, the passivating layer also contain some

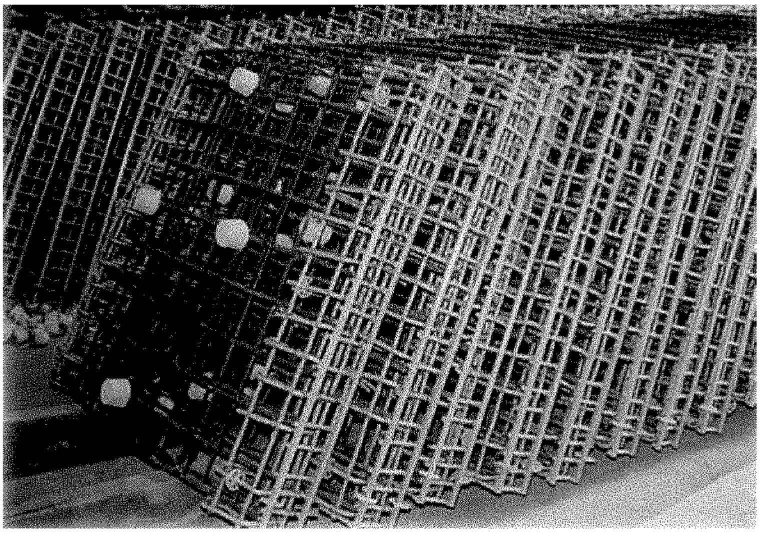


ferrous oxide which can initiate corrosion when the chloride ions reach the steel. The nitrite ions present in the corrosion inhibiting admixture will oxidise the ferrous oxide to ferric oxide, thus stabalising the passivating layer even in the presence of chlorides. The concentration of nitrite must by sufficient to cope up with the continuing ingress of chloride ions.

Calcium nitrite corrosion inhibitor comes in a liquid from containing about 30 per cent calcium nitrite solids by weight. The more corrosion inhibitor is added, the longer the onset of corrosion will be delayed. Since most structures in a chloride environment reach a level of about 7 kg of chloride iron per m^3 during their service life, use of less than 18 litres/m^3 of calcium nitrite solution is not recommended.

Fig. 9.26 shows that without an inhibitor the reinforcing steel starts to corrode when the chloride content at the rebar reaches a threshold level of 0.7 kg/m^3. Although the corrosion process starts when the threshold level is reacted, it may take several years for staining, cracking and spalling to become apparent and several more years before deterioration occurs. Adding calcium nitrite increases this corrosion threshold. When you add 20 litres/m^3, corrosion will not begin until over 7.7 kg/m^3 of chloride is present in the concrete at the rebar.

Coatings to reinforcement: The object of coating to steel bar is to provide a durable barrier to aggressive


Reinforcement cage, partly coated with fusion bonded epoxy, for precast segment.

materials, such as chlorides. The coatings should be robust to withstand fabrication of reinforcement cage, and pouring of concrete and compaction by vibrating needle.

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

Central Electro Chemical Research Institute, (CECRI) Karaikudi have suggested a method for prevention of corrosion in steel reinforcement in concrete. The steps involved in this process are

Derusting: The reinforcements are cleaned with a derusting solution. This is followed without delay by cleaning the rods with wet waste cloth and cleaning powder. The rods are then rinsed in running water and air dried.

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

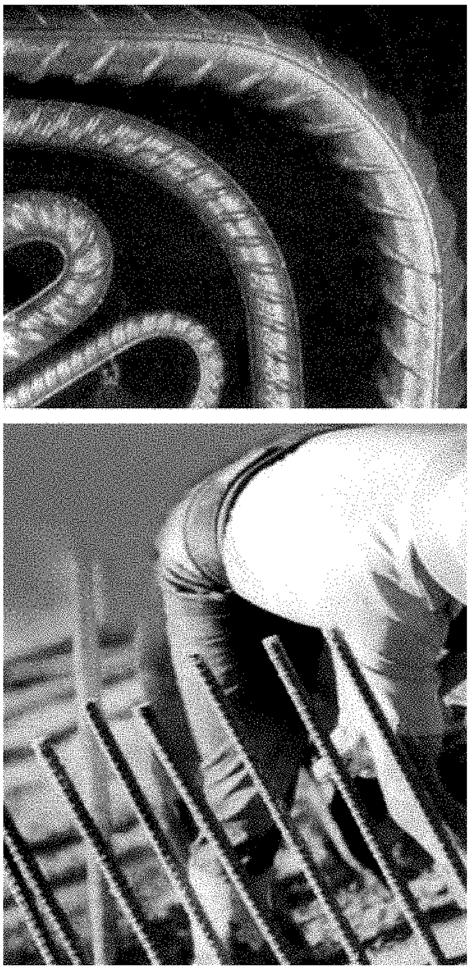
Cement coating: A slurry is made by mixing the inhibitor solution with portland cement and applied on the bar. A sealing solution is brushed after the rods are air cured. The sealing solution has an insite curing effect. The second coat of slurry is then applied and the bars are air dried.

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

The above is a patent method evolved by CECRI and license is given to certain agencies. Somehow or other this method has not become very popular. Some experienced consultants and engineers are doubting the efficacy of this method.

Fusion Bonded Epoxy Coating: It is one of the effective methods of coating rebars. The fusion bonded epoxy coating is a specialised job carried out in a factory and not at site of work. Plants are designed to coat the straight bars in a continuous process. Initially the bar is shot blasted to remove all mill scale and to give the kind of surface finish required. This ensures an adequate bond between epoxy and steel. The bar is then heated to a carefully controlled temperature, before passing through a spray booth. Electrostatically charged epoxy powder particles are deposited evenly on the surface of the bar. It looks, greenish in colour. The coating thickness may vary from 130 to 300 microns.

Although epoxy coated bars have an excellent protection to corrosion in aggressive environment, there are a few limitations.



Galvanising process adopted for corrosion resistance.

After the treatment, cutting and bending may injure the steel which needs certain site treatment. The site treatment is likely to be inefficient. The presence of any defect in the treated body can induce severe localised corrosion which defeat the very purpose. The bars can not be welded. The epoxy is not resistant to ultraviolet rays of sun. The bars should not be exposed to sun for long duration before use.

The coating may get damaged during vibration of concrete. The treatment is very costly: Nearly as costly as that of steel. All the same, this method of protection to the steel is being given to all the flyovers and other structures at Mumbai.

Galvanised reinforcement: Galvanising of reinforcement consists of dipping the steel bars in molten zinc. This results in a coating of zinc bonded to the surface of steel. The zinc surface reacts with calcium hydroxide in the concrete to form a passive layer and prevents corrosion

Cathodic Protection: Cathodic protection is one of the effective, well known, and extensively used methods for prevention of corrosion in concrete structures in more advanced countries. Due to high cost and long term monitoring required for this method, it is not very much used in India.

The cathodic protection comprises of application of impressed current to an electrode laid on the concrete above steel reinforcement. This electrode serves as anode and the steel reinforcement which is connected to the negative terminal of a DC source acts as a cathode. In this process the external anode is subjected to corrode and the cathodic reinforcement is protected against corrosion and hence the name "Cathodic protection".

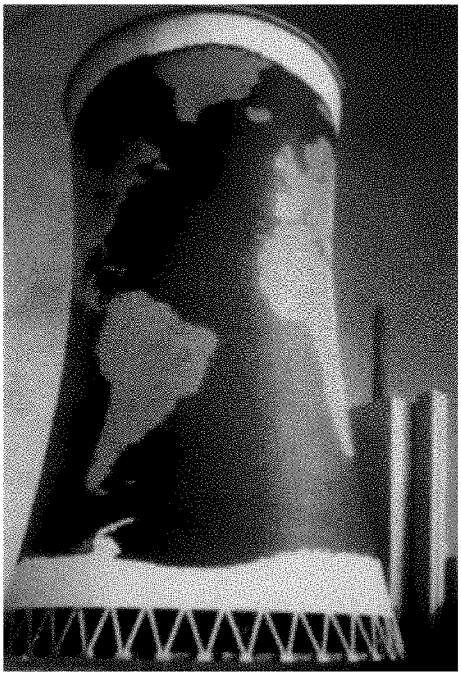
In this process the negative chloride ions which are responsible for the damage of the passivating film, are drawn away from the vicinity of steel towards the anode where they are oxidised to form chlorine gas. The environment around the steel reinforcement reverts back to alkaline condition which protects the steel.

The other recent development in corrosion control methods are Realkalisation and Desalination.

The realkalisation process allows to make the concrete alkaline again and passivate the reinforcing steel by electrochemical method. This brings back the lost alkalinity of concrete to sufficiently high level to reform and maintain the passive layer on the steel.

In the desalination process the chloride ions are removed from the concrete, particularly from the vicinity of the steel reinforcement by certain electrical method to restablish the passive layer on the steel.[9.25]

Coatings to Concrete: In the past it was believed that concrete by itself is a durable material which needs no protection or maintenance. This belief is no more hold good particularly on account of environmental pollution, industrial fumes and

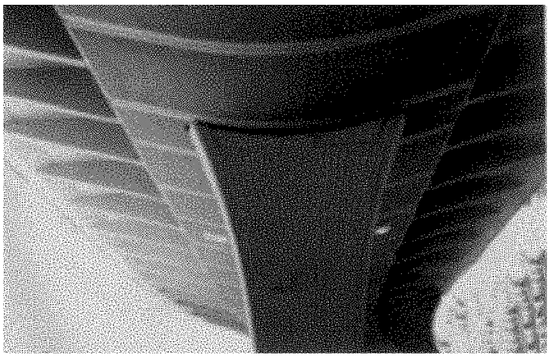


Biggest world map was drawn on cooling tower in Germany using Emce Colour - flex for protection of concrete subjected to aggressive acidic environment.

Courtesy : MC Bauchemie (India) Pvt. Ltd.

contamination of ground water. In addition to the coating of reinforcement by appropriate material, a surface coating to the concrete member is given to increase the durability further. The coatings serve the dual purpose of protection and decoration. Fig. 9.22 shows the reduction in depth of carbonation of the protected concrete.

Giving protective coatings to major concrete structures such as bridges, flyovers, industrial buildings and chimneys have become a common specification in India as in other countries. Four km long bridge on national highway at Cochin was recently coated with Emceecolor Flex, a material manufactured by MC-Bauchemie (Ind) Pvt. Ltd. Almost all the flyovers at Mumbai are being coated for additional durability.



Acrylic based protective cum decorative coating is given to J.J Flyover at Bombay (shown above) and to many other bridges and flyovers in India, particularly in coastal regions.

Courtesy : MC Bauchemie (India) Pvt. Ltd.

Bridge piers and girders are of considerable dimensions. Freshly made concrete members contain plenty of water in the pore structures. It takes long time to dry. Such freshly made concrete structures should not be coated with epoxy or other materials which will seal off and prevent the internal moisture from going out in consonance with atmospheric conditions. The moisture trapped inside the concrete can do untold harm to the durability of concrete in addition to damaging the protective coating itself. For better durability, the concrete should be able to "breathe" i.e, water vapour should be able to migrate from inside to outside and from outside to inside. But water as it is, should not be able to enter from outside to inside. The protective coating given to the concrete should be of the above characteristics.

Therefore, it is pointed out that the expoxy coating which does not allow the concrete to breathe should not be used for coating concrete members.

Instead, the protective coating should be based on acrylics which retains the breathing property of concrete, while protecting the concrete from other harmful environmental agencies, in particular entry of water and carbonation.

In addition, epoxy based coating material is not resistant to ultra violet rays when exposed to sunlight and also it is not flexible. Whereas the coating material based on acrylic polymer is resistant to ultra violet rays of sun and is flexible.

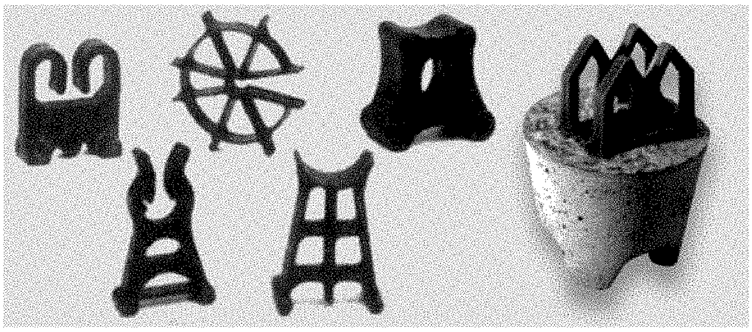
Coating is not only required for bridges, flyovers and industrial structures, it is also required for very thin members like fins, facade, sunbreakers and other delicate concrete structures where specified amount of cover can not be given. Therefore, acrylic based protective cum decorative coatings can be given for additional durability of such concrete members.

Design and Detailing

The structural designer should take all precautions in designing and detailing, with respect to spacing between bars for the concrete to flow between reinforcements, to facilitate vibration of concrete, to give proper cover to the steel reinforcements, to restrict the crack width etc.

Nominal Cover to Reinforcement: The nominal cover is applicable to all steel reinforcement including links.

For longitudinal reinforcing bars in a column, nominal cover in any case not less than 40 mm or less than the diameter of the bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used.



Typical cover blocks for reinforcement. The one in the extreme right is used in the Delhi Metro works.

For footings a minimum over should be 50 mm. Minimum values of nominal cover to all reinforcement including links is to be provided having taken into account the specified period of fire resistance up to 4 hrs.

Table 9.16. Nominal cover to meet the Durability Requirements as per IS 456 of 2000.

Exposure	*Nominal Concrete cover in mm not less than*
Mild	20
Moderate	30
Severe	45
Very Severe	50
Extreme	75

Notes: (1) For main reinforcement up to 12 mm diameter bar for mild exposures, the nominal cover may be reduced by 5 mm.

(2) Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by + 10 mm or 0 mm.

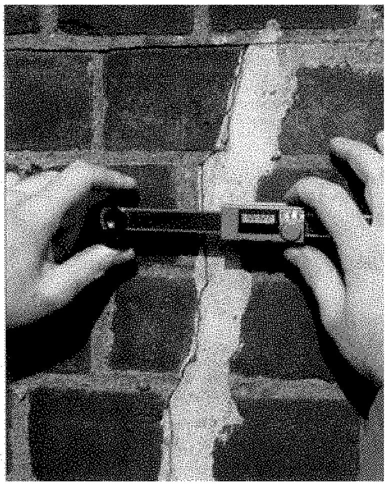
The quality of concrete in the cover region is likely to be of poor quality because of wall-effect. The packing of coarse aggregate is looser at the vicinity of formwork. There is always an increase of paste and less of aggregate in these areas. For long time, this wall-effect phenomenon was not considered as very important from durability point of view because concretes of low slump have been used. But, of late, concretes of slump higher than 200 mm are often used. This high slump concrete increases the risk of segregation and phase separation.

In such cases, addition of colloidal agent., also known as anti-washout, or anti-bleeding admixture could be used to eliminate phase separation problems.

Another method of improving the quality of skin concrete or what is sometimes called covercrete is to adopt permeable formwork technique. This technique is something like vacuum dewatering technique where extra unwanted water is removed by vacuum process. In the case of permeable formwork water from skin portion of concrete is removed by polypropylene fabrics fixed to plywood backing which contain drain holds. Thus, the formwork acts as a filter, through which air and bleed water escape. The permeable formwork lowers the W/C ratio in the concrete up to a depth of 20 mm, making the surface zone resistant against carbonation and diffusion of chloride ions.

Crack Width

Crack Detection Microscope



Digital Crack Measuring Gauge

IS 456 of 2000 specifies crack width as follows. The surface width of the cracks should not, in general, exceed 0.3 mm in members where cracking is not harmful and does not have any serious adverse effect upon the preservation of reinforcing steel nor upon the durability of structures. In members cracking of the tensile zone is harmful either because they are exposed to the effects of the weather or continuously exposed to moisture or in contact with soil or ground water, an upper limit of 0.2 mm is suggested for the maximum width of cracks. For aggressive environment such as severe category of exposure conditions, the surface width of cracks should not in general exceed 0.1 mm

Some specifications limit the crack widths at points near the main reinforcements instead of at the surface. FIP (International Prestressing Federation) recommend the maximum crack width at the main reinforcement to be 0.004 times the nominal cover. If the nominal cover is 50 mm the crack width at the surface comes to 0.004 x 50 = 0.2 mm.

Crack widths at the surface play an important role in the durability of concrete structures. The structural designer should understand and give serious considerations while designing the structures.

If we take a survey of what we have dealt so far in this chapter we come to know that durability of concrete structures get effected by the development of cracks in concrete due to different reasons as described in table 9.6

Before we conclude the discussion on durability of concrete a few more additional information that affects the durability of concrete, which are not directly related to the cracks in concrete, but will have considerable bearing on durability is dealt in subsequent pages.

Deterioration of Concrete by Abrasion, Erosion and Cavitation

Concrete used in certain situations is required to exhibit good abrasion and erosion properties. Abrasion refers to wearing away of the surface by friction. Erosion refers to the same action by fluids. The cavitation refers to the damage due to non-linear flow of water at velocities more than 12 metres per second. The concrete used in the roads, floors and pavements and the concrete used in the hydraulic structures should exhibit resistance against abrasion, erosion and cavitation.

The resistance against these is closely connected with the compressive strength of concrete. The more the compressive strength the higher is the resistance to abrasion. Hardness of aggregate, particularly the coarse aggregate is important to abrasion resistance. Although for concrete of strength of 56 MPa and above, the effect of aggregate hardness is not so important.

The shape and surface texture of aggregate also plays an important part in the abrasion resistance of concrete. A smooth rounded aggregate, when subjected to lateral load, may get dislodged due to lack of bond and interlocking effect of the aggregate. Once the aggregate is dislodged and removed, the paste does not withstand the abrasion action. Rough and angular aggregate with better bond and interlocking effect stands up well against abrasion. If the aggregates are firmly embedded in the matrix, the wearing out of the surface will be uniform without pitting. A placing of homogeneous unsegregated concrete will also exhibit better abrasion resistance. In closed conduits or in a sheet of water flowing over a weir, vapour bubbles are formed in running water whenever the pressure at a point in the water is reduced to its vapour pressure at the existing temperature. Vapour bubbles flow downstream with the water and when they enter a region of higher pressure, they collapse with great impact. The formation of vapour bubbles and their subsequent collapse is called cavitation. The energy given up on their collapse causes "cavitation damage". At higher velocities, the forces of cavitation may be strong enough to damage the concrete surface in a very serious manner.

The method adopted for avoiding cavitation, apart from careful designing of the structure, making smooth surface free from irregularities is other effective step. The concrete should be of high strength, well cured and finished smooth. Epoxy screeding and polymer application to the surface is said to be effective against cavitation.

Effects of Some Materials on Durability

Action of mineral oils

Mineral such as petrol, and petroleum distillates in general, do not attack hardened concrete though they seriously affect hardening process of fresh concrete. Creosotes which contain phenols may have some effects on concretes. Lubricating oils which are entirely of mineral origin do not attack concrete. As a matter of fact, often concrete tanks are used as a storage tanks for mineral oils. To reduce permeability, rich concrete is used and also some kind of surface treatment such a four coats of sodium silicate is applied.

Action of Organic Acids

There are a number of organic acids which sometimes come into contact with concrete and cause deleterious effect on it. Acetic acid, lactic acid and butyric acid attack concrete with severity depending upon concentration and temperature. Fresh milk has so little lactic acid that it does not harm concrete. Formic acid is corrosive to concrete. Tannic acid and phenols are only mildly corrosive. Oleic, stearic acids occurring in various oils and fats, though insoluble in water have some corrosive action on concrete.

Vegetables and Animal Oils and Fats

Many vegetable oils contain small amounts of free fatty essence and produce a slow deterioration of concrete surfaces. Most fresh animal oils contain little acid but rancid animal oils contain considerably more acid and therefore are corrosive. Some fish oils are said to be more corrosive than other animal oils. Laboratory tests show that Portland cements concrete are rapidly attacked by cotton-seed oils.

Action of Sugar on Concrete

Sugar is a powerful retarding agent, but its action on hardened concrete is not of much consequence, though it may gradually corrode the concrete. Concrete tanks have been used for the storage of molasses with satisfactory results. It should be noted that concrete tanks should be well cured for at least 28 days before being charged with syrups and molasses. It is also recommended that the surface of concrete tanks may be treated with sodium silicate solution or tar or asphalt.

Action of Sewage

Domestic sewage has not got detrimental effect on good concrete. As such, concrete pipes are used for conveying sewage; also concrete is used for constructing sewage treatment plants. Hydrogen sulphide gas which may be evolved from septic sewage in sewer or sludge digestion tank, though by itself is not harmful, may promote the formation of sulphuric acid which can attack the concrete surface above the liquid level. Concrete sewers running full are not attacked.

If the sewage contains more than 150 ppm of soluble sulphate salts (as SO_4), sulphate attack may take place. Domestic sewage rarely contains this amount of sulphate salts but discharge of certain industrial wastes into sewer could increase the concentration of sulphate salts.

Concrete pipes to carry sewage should be of low permeability to minimise penetration of liquid. This could be done by rich concrete, low water cement ratio and good compaction to increase the durability of the sewer. It should be seen that formation of sulphuric acid is avoided by keeping sufficient quantity of flow, proper ventilation of sewer and by avoiding the stagnation or septicity of sewage.

Surface Treatments of Concrete

There are large varieties of materials, which are applied to surface of concrete either to waterproof the surface or to render it resistant to attack of chemical agencies. Sometimes, they are added to give a quick hardening effect so that concrete is made stronger within a short period to enable it to withstand destructive forces. Some of the materials used for surface treatments are listed below:

(*a*) Aqueous solution of sodium silicate.
(*b*) Magnesium or zinc silico fluoride.
(*c*) Drying oils such as linseed or tung oil.
(*d*) Chlorinated rubber paints.
(*e*) Neoprene paints.
(*f*) Epoxy paints or coal tar epoxy paints.
(*g*) Silicon fluoride (SiF_4) treatment.

Concrete surface can be hardened and rendered more resistant to abrasion, or less liable to create dust, by suitable treatment. The surface of the hardened and air-dry concrete may be treated with a solution of sodium silicate, aluminium, or zinc sulphate, or silicofluorides, or with drying oil like linseed oil or tung oil; alternatively a proportion of carborundum or fused alumina, or one of the finely divided iron-ammonium chloride preparations, may be incorporated in the surface layer of the concrete while placing.

Carborundum, fused alumina, and finely divided iron are effective in rendering a concrete surface less slippery and more-resistant to abrasion, but they are not very effective in rendering the surface less dusty. Treatment with a solution of sodium silicate hardens a concrete surface and also renders it less dusty. Treatment with the sulphate solutions and slilicoflouride is also effective.

Tung oil and linseed oil are applied to concrete surface either neat, hot or thinned with turpentine or white spirits. The treatment gives a hard surface and freedom from dust. Floor paints also have a reasonable durability if the conditions of wear are not heavy. Oil paints with a tung-oil medium, or bituminous paints, can be used, but paints containing synthetic resins particularly polyurethanes or epoxyesters, or chlorinated rubber have a greater resistance to wear. None of these surface treatments are effective on a weak or friable concrete surface.

Treatment with the Sodium Silicate and Silicofluoride only affords protection against mild conditions of attack either by aqueous solutions or organic liquids. Drying oils can exert a protective influence for some years against dilute aqueous solutions of aggressive soils. The surface coatings by chlorinated rubber paints, neoprene paints, epoxy paints etc. are found to be effective in protecting concrete against aqueous solutions of salts and dilute acids. Epoxy paints or synthetic resin lacquers specially prepared for the treatment of concrete surfaces have given good protection.

The above materials have often been applied for protecting concrete surfaces from abrasion, erosion and general deteriorating action of concrete piles, jetty piers and other hydraulic structures. A treatment by bitumen and coal tar has been found to give protection against insects and borers. Some plastic materials, rubber, latex glass fibre coatings, PVC linings have also been given to concrete in certain situations for increasing durability.

One of the important surface treatment adopted to increase the durability of concrete is what is known as 'Ocrate Process'. In this, concrete member is impregnated with Silico fluoride under pressure. This method has been adopted to increase the durability of precast concrete piles and pipes carrying sewage.

Lastly, one of the recent application to improve the durability of concrete is the technique of polymer impregnation. Polymer application to the concrete surface, improves the durability of concrete manifold. Though, this process is at present very costly, there is to doubt that the technique of polymer application to concrete will have a bright future.

Before we end this chapter, it is pertinent to bring out a few tables on exposure conditions which have serious bearing on durability of concrete. As per IS 456 of 2000, the general environment to which the concrete will be exposed during its working life is classified into five levels of severity, that is, mild, moderate, severe, very severe and extreme. This is described in Table 9.17

Table 9.17. Environmental Exposure Conditions (IS 456 of 2000)

Environment	*Exposure Conditions*
Mild	Concrete surfaces protected against weather or aggressive conditions except those situated in coastal areas
Moderate	Concrete surfaces sheltered from severe rain or freezing whilst wet Concrete exposed to condensation and rain. Concrete continuously under water. Concrete in contact or buried under non-aggressive soil/

	ground water.
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation. Concrete Completely immersed in Sea water.
Very Severe	Concrete surfaces exposed to sea water spray, corrosive fumes or severe freezing conditions whilst wet. Concrete in contact or buried under aggressive subsoil/ground water. Concrete exposed to coastal environment
Extreme	Surface of members in tidal zone, members in dired contact with liquid/solid aggressive chemicals

It is stressed time and again that the free W/C ratio is very important factor in governing the durability of concrete. It should always be the lowest value. Appropriate values for minimum cement content, the maximum free W/C ratio and the minimum grade of concrete are given in Table 9.18. as given in IS 456 of 2000, for different exposure conditions.

The minimum cement content and maximum W/C ratio apply to 20 mm nominal maximum size aggregate. For other sizes of aggregate, they should be changed as per Table 9.19.

Maximum Cement Content: Cement content not including fly ash and ground granulated blast furnace slag in excess of 450 kg/m^3 should not be used unless special consideration has been given in design to the increased risk of cracking due to drying shrinkage in thin sections, or to early thermal cracking and to the increased risk of damage due to alkali silica reaction.

Table 9.18. Minimum Cement Content, Maximum W/C Ratio and Minimum Grade of Concrete for Different Exposures with Normal Weight Aggregates of 20 mm Nominal Maximum size. IS 456 : 2000

Sl. No.	*Exposure*	*Plain Concrete*			*Reinforced Concrete*		
		Minimum cement contents kg/m^3	*Maximum Free W/C ratio*	*Minimum Grade of concrete*	*Minimum Cement Content kg/m^3*	*Maximum Free W/C ratio*	*Minimum Grade of Concrete*
1.	Mild	220	0.60	–	300	0.55	M 20
2.	Moderate	240	0.60	M 15	300	0.50	M 25
3.	Severe	250	0.50	M 20	320	0.45	M 30
4.	Very Severe	260	0.45	M 20	340	0.45	M 35
5.	Extreme	280	0.40	M 25	360	0.40	M 40

Notes: (1) Cement content prescribed in this table is irrespective of the grade of cement and it is inclusive of all supplementary cementitious materials. The additions of all supplementary cementitious materials may be taken into account in the concrete composition with respect to the cement content and W/C ratio if the suitability is established and as long as the maximum amounts taken into account do not exceed the limit prescribed in relevant codes.

(2) Minimum grade for plain concrete under mild exposure condition is not specified.

Table 9.19. Adjustments to Minimum Cement Contents for Aggregates other than 20 mm Nominal Maximum Size as per IS 456-2000

Sl. No.	*Nominal Maximum Aggregate Size mm*	*Adjustment to Minimum cement contents in table 9.18. kg/m³*
1.	10	+ 40
2.	20	0
3.	40	– 30

The above two tables *i.e.*, Table No. 9.18 and Table No. 9.19 are important from the points of concrete mix design by IS method. With the introduction of revised by IS 456 of 2000, the IS 10262 of 1982 for concrete mix design has become in-operational. However, when the I.S. mix design method is revised the above two tables forms back bone of the concrete mix design.

For general imformation and also to carry out the concrete mix design as per British method (DOE method), Table 9.20 and Table 9.21 are also given.

Table 9.20. Requirements of BS 8110: Part I: 1985 to Ensure Durability Under Specified Exposure Conditions of Reinforced and Prestressed Concrete Made with Normal Weight Aggregate.

Condition of exposure	*Nominal Cover of Concrete in mm*				
Mild	25	20	20	20	20
Moderate	–	35	30	25	20
Severe	–	–	40	30	25
Very Severe	–	–	50	40	30
Extreme	–	–	–	60	50
Maximum Water/ Cementitious material ratio	0.65	0.60	0.55	0.50	0.45
Minimum content of cementitious Material in kg/m³	275	300	325	350	400
Minimum grade MPa	30	35	40	45	50

Note: (1) Grade is characteristic cube strength

(2) This table applies when maximum size of aggregate is 20 mm. When it is 10 mm and 14 mm respectively, the content of cementitious material should be increased by 40 kg/m³ and 20 kg/m³. Conversely for maximum size of aggregate of 40 mm, the content of cementitious material can be reduced by 30 kg/m³. Prestressed concrete must contain at least 300 kg/m³ of cementitious material.

(3) For exposure to freezing and thawing, air-entrainment should be used.

Table 9.21. Requirements of BS 8110: Part 1-1985 to Ensure Durability under Specified Conditions of Exposure of Plain Concrete.

Environment	*Exposure Condition*	*Maximum free water/ cementitious material ratio*	*Minimum Content of cementitious material kg/m³ for nominal maximum size of aggregate*				*Minimum grade MPa*
			40 mm	*20 mm*	*14 mm*	*10 mm*	
Mild	Concrete surfaces protected against weather or aggressive condition.	0.80	150	180	200	220	20
Moderate	Concrete surfaces sheltered from severe rain whilst wet. Concrete subject to condensation. Concrete surfaces continuously under water. Concrete in contact with non-aggressive soil.	0.65	245	275	295	315	30
Severe	Concrete surfaces exposed to severe rain, alternating wetting and drying or occasional freezing or severe condensation.	0.60	270	300	320	340	35
Very Severe	Concrete surfaces exposed to sea water spray de-icing salts (directly or indirectly), corrosive fumes or severe freezing conditions whilst wet.	0.55	295	325	345	365	35
Extreme	Concrete surfaces exposed to abrasive action, *e.g.*, sea water carrying solids or flowing water with pH £ 4.5 or machinery or vehicles.	0.50	320	350	370	390	45

Note: (1) Cementitious materials is inclusive of slag or fly ash.
(2) Grade of concrete is the characteristic cube strength
(3) For very severe conditions only air-entrained concrete to be used.

Concluding Remarks on Durability

Strength and durability are the two important properties of concrete. They are like two legs of a human body. In India with the availability of good cement since about 1985, good strength can be obtained if reasonable care is taken. But, samething can not be said with regard to durability, because of the rapid deterioration in environmental conditions and the use of concrete has spread to much more hostile regions than ever before.

Concrete is a very loyal and dependable construction material. It should not be abused to the limit as it is done today. It should be used with understanding, love and care.

All that is written in this chapter is a scientific exposition at macro and micro-level of durability problem. It will give an indepth knowledge and better understanding for making good standard concrete for our infrastructural development which we have just started. The information contained in this chapter may be little difficult for the consumption of our common builders who are involved with about 80 per cent of total concrete made in our country.

What is relevant to him in respect of durability is what is written in the chapters from 1 to 6. These chapters deal with the good concrete construction practices for making durable concrete. They are to be considered as introductory part of chapter 9 *i.e.*, on durability.

It may not be out of place to emphasize that we, civil engineers, common builders, site engineers, teachers, concrete technologists and all others who are involved in making concrete, have a lot of responsibilities for making durable concrete for which our country spends about 25 per cent of nation's annual budget. Which other section of our society has more responsibility than we–friends of concrete.

AIM SHOULD BE TO MAKE OUR CONCRETE AS DURABLE AS THESE MONUMENTS!

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Top