

Shrinkage of concrete

What is Shrinkage?

Shrinkage of concrete is the time-dependent strain measured in an unloaded and unrestrained specimen at constant temperature

To understand this aspect more closely, shrinkage can be classified in the following ways:-

1. Plastic Shrinkage
2. Drying Shrinkage
3. Autogeneous Shrinkage
4. Carbonation Shrinkage

1. Plastic shrinkage :

Shrinkage of this type manifests itself soon after the concrete is placed in the forms while the concrete is still in the plastic state. Loss of water by evaporation from the surface of concrete or by the absorption by aggregate or subgrade, is believed to be the reasons of plastic shrinkage. The loss of water results in the reduction of volume. The aggregate particles or the reinforcement comes in the way of subsidence due to which cracks may appear at the surface or internally around the aggregate or reinforcement.

In case of floors and pavements where the surface area exposed to drying is large as compared to depth, when this large surface is exposed to hot sun and drying wind, the surface of concrete dries very fast which results in plastic shrinkage.

Sometimes even if the concrete is not subjected to severe drying, but poorly made with a high water/ cement ratio, large quantity of water bleeds and accumulates at the surface. When this water at the surface dries out, the surface concrete collapses causing cracks.

Plastic concrete is sometimes subjected to unintended vibration or yielding of formwork support which again causes plastic shrinkage cracks as the concrete at this stage has not developed enough strength. From the above it can be inferred that high water/ cement ratio, badly proportioned concrete, rapid drying, greater bleeding, unintended vibration etc., are some of the reasons for plastic shrinkage. It can also be further added that richer concrete undergoes greater plastic shrinkage.

Plastic shrinkage can be reduced mainly by preventing the rapid loss of water from surface. This can be done by covering the surface with polyethylene sheeting immediately on finishing operation; by monomolecular coatings by fog spray that keeps the surface moist; or by working at night. An effective method of removing plastic shrinkage cracks is to revibrate the concrete in a controlled manner. Use of small quantity of aluminium powder is also suggested to offset the effect of plastic shrinkage. Similarly, expansive cement or shrinkage compensating cement also can be used for controlling the shrinkage during the setting of concrete. The principal property of such cement is that the expansion induced in the plastic concrete will almost offset

the normal shrinkage due to loss of moisture. Under correct usage, the distance between the joints can sometimes be tripled without increasing the level of shrinkage cracking. Further, use of unneeded high slump concrete, over sanded mix, higher air entraining should be discouraged in order to reduce the higher plastic shrinkage.

2. Drying Shrinkage:

Just as the hydration of cement is an ever lasting process, the drying shrinkage is also an ever lasting process when concrete is subjected to drying conditions. The drying shrinkage of concrete is analogous to the mechanism of drying of timber specimen. The loss of free water contained in hardened concrete, does not result in any appreciable dimension change. It is the loss of water held in gel pores that causes the change in the volume. Under drying conditions, the gel water is lost progressively over a long time, as long as the concrete is kept in drying conditions. It is theoretically estimated that the total linear change due to long time drying shrinkage could be of the order of 10,000 microns. But values upto 4,000 microns have been actually observed.

Cement paste shrinks more than mortar and mortar shrinks more than concrete. Concrete made with smaller size aggregate shrinks more than concrete made with bigger size aggregate. The magnitude of drying shrinkage is also a function of the fineness of gel. The finer the gel the more is the shrinkage. It has been pointed out earlier that the high pressure steam cured concrete with low specific surface of gel, shrinks much less than that of normally cured cement gel.

3. Autogeneous Shrinkage:

In a conservative system i.e. where no moisture movement to or from the paste is permitted, when temperature is constant some shrinkage may occur. The shrinkage of such a conservative system is known as a autogeneous shrinkage.

Autogeneous shrinkage is of minor importance and is not applicable in practice to many situations except that of mass of concrete in the interior of a concrete dam. The magnitude of autogeneous shrinkage is in the order of about 100 microns.

4. Carbonation Shrinkage:

Carbonation shrinkage is a phenomenon very recently recognised. Carbon dioxide present in the atmosphere reacts in the presence of water with hydrated cement. Calcium hydroxide gets converted to calcium carbonate and also some other cement compounds are decomposed. Such a complete decomposition of calcium compound in hydrated cement is chemically possible even at the low pressure of carbon dioxide in normal atmosphere. Carbonation penetrates beyond the exposed surface of concrete only very slowly.

The rate of penetration of carbon dioxide depends also on the moisture content of the concrete and the relative humidity of the ambient medium. Carbonation is accompanied by an increase in weight of the concrete and by shrinkage. Carbonation shrinkage is probably caused by the dissolution of crystals of calcium hydroxide and deposition of calcium carbonate in its place. As the new product is less in volume than the product replaced, shrinkage takes place.

Carbonation of concrete also results in increased strength and reduced permeability, possibly because water released by carbonation promotes the process of hydration and also calcium carbonate reduces the voids within the cement paste. As the magnitude of carbonation shrinkage is very small when compared to long term drying shrinkage, this aspect is not of much significance. But carbonation reduces the alkalinity of concrete which gives a protective coating to the reinforcement against rusting. If depth of carbonation reaches up to steel reinforcements, the steel becomes liable for corrosion

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One of the most important factors that affects shrinkage is the drying condition or in other words, the relative humidity of the atmosphere at which the concrete specimen is kept. If the concrete is placed in 100 per cent relative humidity for any length of time, there will not be any shrinkage; instead there will be a slight swelling. The typical relationship between shrinkage and time for which concrete is stored at different relative humidities is shown in Figure. The graph shows that the magnitude of shrinkage increases with time and also with the reduction of relative humidity.

The rate of shrinkage decreases rapidly with time. It is observed that 14 to 34 per cent of the 20 year shrinkage occurs in 2 weeks, 40 to 80 per cent of the 20 year shrinkage occurs in 3 months and 66 to 85 per cent of the 20 year shrinkage occurs in one year. Another important factor which influences the magnitude of shrinkage is water/cement ratio of the concrete. The richness of the concrete also has a significant influence on shrinkage. Aggregate plays an important role in the shrinkage properties of concrete. The quantum of an aggregate, its size, and its modulus of elasticity influence the magnitude of drying shrinkage.

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Harder aggregate with higher modulus of elasticity like quartz shrinks much less than softer aggregates such as sandstone.

Moisture Movement Concrete shrinks when allowed to dry in air at a lower relative humidity and it swells when kept at 100 per cent relative humidity or when placed in water.

Just as drying shrinkage is an ever continuing process, swelling, when continuously placed in water is also an ever continuing process. If a concrete sample subjected to drying condition, at some stage, is subjected to wetting condition, it starts swelling. It is interesting to note that all the initial drying shrinkage is not

recovered even after prolonged storage in water which shows that the phenomenon of drying shrinkage is not a fully reversible one.

Just as the **drying shrinkage** is due to loss of adsorbed water around gel particles, swelling is due to the adsorption of water by the cement gel. The water molecules act against the cohesive force and tend to force the gel particles further apart as a result of which swelling takes place. In addition, the ingress of water decreases the surface tension of the gel.

Effect of Concrete Ingredients on Shrinkage

Many researchers have studied the factors associated with the shrinkage of concrete mixtures. The most influential factor is the type of coarse aggregate used. Hard, dense aggregate is able to restrain the shrinkage of the cement paste. In contrast, using aggregate with a higher compressibility can increase the shrinkage of the concrete mixture by about 120 to 150 percent. Therefore, locally-available materials play a critical role in the shrinkage behavior of the concrete. The properties of aggregate from various quarries should be considered if shrinkage is to be minimized. Some recommendations include using a large top-size aggregate and optimizing the gradation of the aggregate and combining aggregate sources to minimize gap-grading and corresponding paste content of the concrete. However, the overall benefit of these suggestions is dependent on the aggregate properties used. If the aggregate is of poor inherent quality, maximizing the size, gradation, and content may have little effect on the concrete shrinkage. Likewise, blending a large aggregate with poor qualities to a mid-size aggregate with good properties may increase the resulting shrinkage behavior of the concrete mixture.

Other factors that have been found to have a significant impact on the shrinkage of concrete mixtures includes the use of shrinkage-promoting admixtures (such as accelerators), the use of dirty aggregate which increases water demand and using a cement with high shrinkage characteristics. The cumulative effect of these factors has been found to be multiplicative and not additive. So, combined factors can easily increase concrete shrinkage by several hundred percent. Therefore, the specific impact of any set of materials should be determined by laboratory testing.

The shrinkage of a concrete mixture can have a significant impact on the performance of floors on ground. With the increasing demand for structural load-carrying capability and corresponding floor performance, shrinkage has become a growing issue. As repairs and maintenance can be costly, good joint performance is essential for industrial concrete floors. Therefore, it is important for slabs to remain in contact with the supporting base (minimal warping) and the joints to have minimal widening. The shrinkage potential of the concrete mixture must be well-understood so proper design and construction methods can result in the expected long-term serviceability for the owner. Even for commercial floors where heavy loading does not occur, concrete shrinkage can result in warping relaxation subsequent to installation of floor coverings or coatings. When surfaces are reprofiled in preparation for installation of floor finishes, slab distortion can result in delamination and buckling of the flooring. Knowledge of potential concrete shrinkage can help minimize such problems.

Effects of Shrinkage

If concrete members were free to shrink, without restraint, shrinkage of concrete would not be a major concern to structural engineers. However, this is not the case. The contraction of a concrete member is often restrained by its supports or by the adjacent structure. Bonded reinforcement also restrains shrinkage. Each of these forms of restraint involve the imposition of a gradually increasing tensile force on the concrete which may lead to time-dependent cracking (in previously uncracked regions), increases in deflection and a widening of existing cracks. Restraint to shrinkage is probably the most common cause of unsightly cracking

in concrete structures. In many cases, these problems arise because shrinkage has not been adequately considered by the structural designer and the effects of shrinkage are not adequately modelled in the design procedures specified in codes of practice for crack control and deflection calculation.

The advent of shrinkage cracking depends on the degree of restraint to shrinkage, the extensibility and strength of the concrete in tension, tensile creep and the load induced tension existing in the member. Cracking can only be avoided if the gradually increasing tensile stress induced by shrinkage, and reduced by creep, is at all times less than the tensile strength of the concrete. Although the tensile strength of concrete increases with time, so too does the elastic modulus and, therefore, so too does the tensile stress induced by shrinkage. Furthermore, the relief offered by creep decreases with age. The existence of load induced tension in uncracked regions accelerates the formation of time-dependent cracking. In many cases, therefore, shrinkage cracking is inevitable. The control of such cracking requires two important steps. First, the shrinkage-induced tension and the regions where shrinkage cracks are likely to develop must be recognised by the structural designer. Second, an adequate quantity and distribution of anchored reinforcement must be included in these regions to ensure that the cracks remain fine and the structure remains serviceable.

Drying shrinkage is the reduction in volume caused principally by the loss of water during the drying process. Chemical (or endogenous) shrinkage results from various chemical reactions within the cement paste and includes hydration shrinkage, which is related to the degree of hydration of the binder in a sealed specimen. Concrete shrinkage strain, which is usually considered to be the sum of the drying and chemical shrinkage components, continues to increase with time at a decreasing rate. Shrinkage is assumed to approach a final value, ϵ_{sh}^* , as time approaches infinity and is dependent on all the factors which affect the drying of concrete, including the relative humidity and temperature, the mix characteristics (in particular, the type and quantity of the binder, the water content and water-to-cement ratio, the ratio of fine to coarse aggregate, and the type of aggregate), and the size and shape of the member.

Drying shrinkage in high strength concrete is smaller than in normal strength concrete due to the smaller quantities of free water after hydration. However, endogenous shrinkage is significantly higher.

For normal strength concrete ($f_c' \leq 50$ MPa), AS3600 suggests that the design shrinkage (which includes both drying and endogenous shrinkage) at any time after the commencement of drying may be estimated from

$$\epsilon_{sh} = k_1 \epsilon_{sh,b} \quad (1)$$

where $\epsilon_{sh,b}$ is a basic shrinkage strain which, in the absence of measurements, may be taken to be 850×10^{-6} (note that this value was increased from 700×10^{-6} in the recent Amendment 2 of the Standard); k_1 is obtained by interpolation from Figure 6.1.7.2 in the Standard and depends on the time since the commencement of drying, the environment and the concrete surface area to volume ratio. A hypothetical thickness, $t_h = 2A / u_e$, is used to take this into account, where A is the cross-sectional area of the member and u_e is that portion of the section perimeter exposed to the atmosphere plus half the total perimeter of any voids contained within the section.

AS3600 states that the actual shrinkage strain may be within a range of plus or minus 40% of the value predicted (increased from $\pm 30\%$ in Amendment 2 to AS3600-1994). In the writer's opinion, this range is still optimistically narrow, particularly when one considers the size of the country and the wide variation in shrinkage measured in concretes from the various geographical locations. Equation 1 does not include any of the effects related to the composition and quality of the concrete. The same value of ε_{cs} is predicted irrespective of the concrete strength, the water-cement ratio, the aggregate type and quantity, the type of admixtures, etc. In addition, the factor k_1 tends to overestimate the effect of member size and significantly underestimate the rate of shrinkage development at early ages.

The method should be used only as a guide for concrete with a low water-cement ratio (<0.4) and with a well graded, good quality aggregate. Where a higher water-cement ratio is expected or when doubts exist concerning the type of aggregate to be used, the value of ε_{cs} predicted by AS3600 should be increased by at least 50%. The method in the Standard for the prediction of shrinkage strain is currently under revision and it is quite likely that significant changes will be proposed with the inclusion of high strength concretes.

A proposal currently being considered by Standards Australia, and proposed by Gilbert (1998) [9], involves the total shrinkage strain, ε_{cs} , being divided into two components, endogenous shrinkage, ε_{cse} , (which is assumed to develop relatively rapidly and increases with concrete strength) and drying shrinkage, ε_{csd} (which develops more slowly, but decreases with concrete strength). At any time t (in days) after pouring, the endogenous shrinkage is given by

$$\varepsilon_{cse} = \varepsilon_{cse}^* (1.0 - e^{-0.1t}) \quad (2)$$

where ε_{cse}^* is the final endogenous shrinkage and may be taken as $\varepsilon_{cse}^* = (3f'_c - 50) \times 10^{-6}$, where f'_c is in MPa. The basic drying shrinkage ε_{csd}^* is given by

$$\varepsilon_{csd}^* = (1100 - 8f'_c) \times 10^{-6} \geq 250 \times 10^{-6} \quad (3)$$

and at any time t (in days) after the commencement of drying, the drying shrinkage may be taken as

$$\varepsilon_{csd} = k_1 \varepsilon_{csd}^* \quad (4)$$

The variable k_1 is given by
$$k_1 = \frac{k_4 k_5 t^{0.8}}{t^{0.8} + (t_h / 7)} \quad (5)$$

where $k_4 = 0.8 + 1.2e^{-0.003k_5}$ and k_5 is equal to 0.7 for an arid environment, 0.6 for a temperate environment and 0.5 for a tropical/coastal environment. For an interior environment, k_5 may be taken as 0.65. The value of k_1 given by Equation 5 has the same general shape as that given in Figure 6.1.7.2 in AS3600, except that shrinkage develops more rapidly at early ages and the reduction in drying shrinkage with increasing values of t_h is not as great.

The final shrinkage at any time is therefore the sum of the endogenous shrinkage (Equation 2) and the drying shrinkage (Equation 4). For example, for specimens in an interior environment with hypothetical thicknesses $t_h = 100$ mm and $t_h = 400$ mm, the shrinkage strains predicted by the above model are given in Table 1.

Table 1 Design shrinkage strains predicted by proposed model for an interior environment.

ϵ_h	f'_c	ϵ_{se}^* (x 10 ⁻⁶)	ϵ_{sd}^* (x 10 ⁻⁶)	Strain at 28 days			Strain at 10000 days		
				(x 10 ⁻⁶)			(x 10 ⁻⁶)		
				ϵ_{se}	ϵ_{sd}	ϵ_{cs}	ϵ_{se}	ϵ_{sd}	ϵ_{cs}
100	25	25	900	23	449	472	25	885	910
	50	100	700	94	349	443	100	690	790
	75	175	500	164	249	413	175	493	668
	100	250	300	235	150	385	250	296	546
400	25	25	900	23	114	137	25	543	568
	50	100	700	94	88	182	100	422	522
	75	175	500	164	63	227	175	303	478
	100	250	300	235	38	273	250	182	432

Shrinkage in Unrestrained and Unreinforced Concrete (Gilbert, 1988) [7]

Drying shrinkage is greatest at the surfaces exposed to drying and decreases towards the interior of a concrete member. In Fig.1a, the shrinkage strains through the thickness of a plain concrete slab, drying on both the top and bottom surfaces, are shown. The slab is unloaded and unrestrained.

The mean shrinkage strain, ϵ_{cs} in Fig. 1, is the average contraction. The non-linear strain labelled $\Delta\epsilon_{cs}$ is that portion of the shrinkage strain that causes internal stresses to develop. These self-equilibrating stresses (called eigenstresses) produce the elastic and creep strains required to restore compatibility (ie. to ensure that plane sections remain plane). These stresses occur in all concrete structures and are tensile near the drying surfaces and compressive in the interior of the member. Because the shrinkage-induced stresses develop gradually with time, they are relieved by creep. Nevertheless, the tensile stresses near the drying surfaces often overcome the tensile strength of the immature concrete and result in surface cracking, soon after the commencement of drying. Moist curing delays the commencement of drying and may provide the concrete time to develop sufficient tensile strength to avoid unsightly surface cracking.

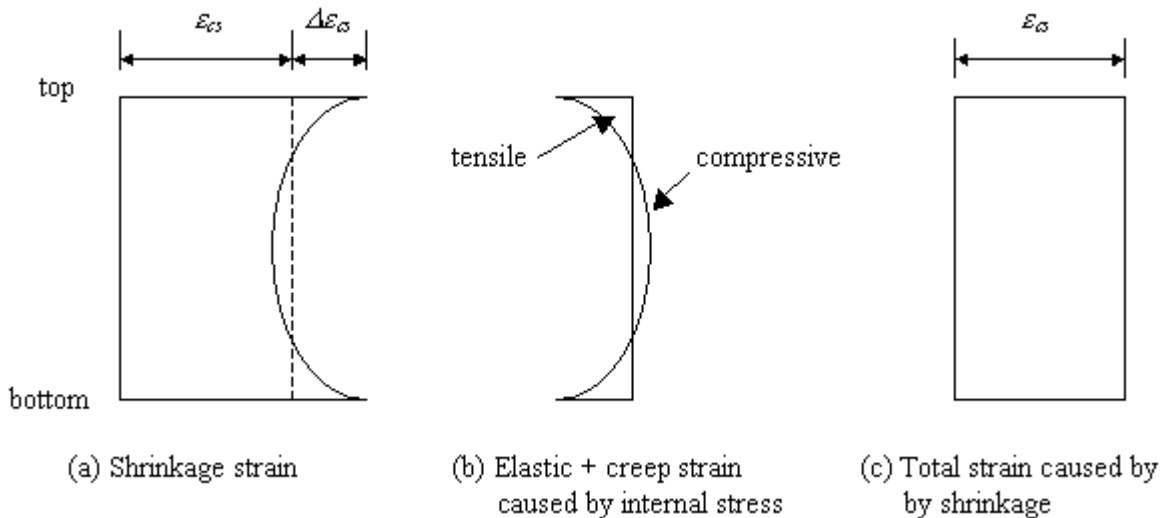


Fig. 1 - Strain components caused by shrinkage in a plain concrete slab.

The elastic plus creep strains caused by the eigenstresses are equal and opposite to $\Delta\epsilon_{cs}$ and are shown in Fig. 1b. The total strain distribution, obtained by summing the elastic, creep and shrinkage strain components, is linear (Fig. 1c) thus satisfying compatibility. If the drying conditions are the same at both the top and bottom surfaces, the total strain is uniform over the depth of the slab and equal to the mean shrinkage strain, ϵ_{cs} . It is this quantity that is usually of significance in the analysis of concrete structures. If drying occurs at a different rate from the top and bottom surfaces, the total strain distribution becomes inclined and a warping of the member results.

3.3 Shrinkage in an unrestrained reinforced concrete member (Gilbert, 1986)

In concrete structures, unrestrained contraction and unrestrained warping are unusual. Reinforcement embedded in the concrete provides restraint to shrinkage. As the concrete shrinks, the reinforcement is compressed and imposes an equal and opposite tensile force on the concrete at the level of the reinforcement. If the reinforcement is not symmetrically placed on a section, a shrinkage-induced curvature develops with time. Shrinkage in an unsymmetrically reinforced concrete beam or slab can produce deflections of significant magnitude, even if the beam is unloaded.

Consider the unrestrained, singly reinforced, simply-supported concrete beam shown in Figure 2a and the small beam segment of length Δx . The shrinkage induced stresses and strains on an uncracked and on a cracked cross-section are shown in Figures 2b and 2c, respectively.

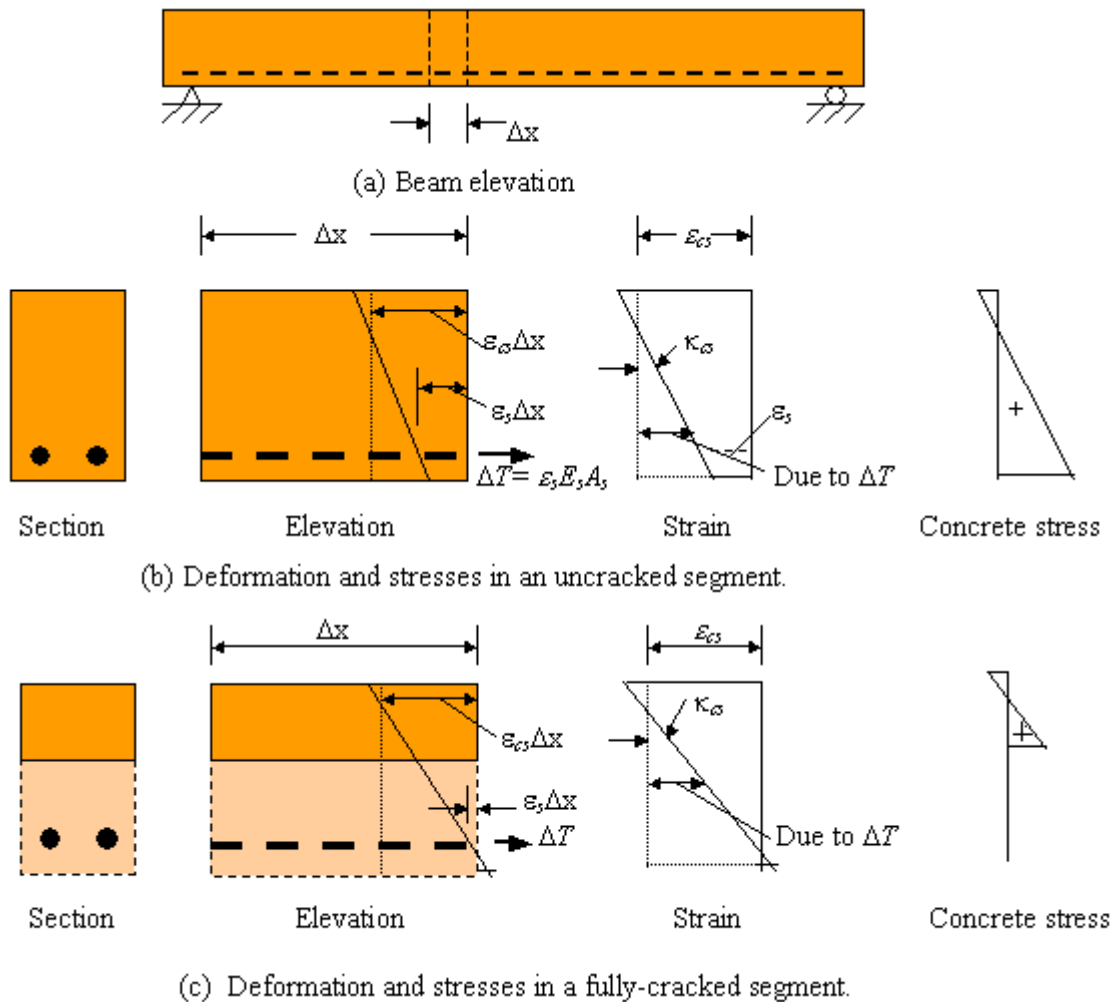


Fig. 2 - Shrinkage warping in a singly reinforced beam.

As the concrete shrinks, the bonded reinforcement imposes a tensile restraining force, ΔT , on the concrete at the level of the steel. This gradually increasing tensile force, acting at some eccentricity to the centroid of the concrete cross-section, produces curvature (elastic plus creep) and a gradual warping of the beam. It also may cause cracking on an uncracked section or an increase in the width of existing cracks in a cracked member. For a particular shrinkage strain, the magnitude of ΔT depends on the quantity of reinforcement and on whether or not the cross-section has cracked.

Shrinkage strain is independent of stress, but shrinkage warping is not independent of the load and is significantly greater in a cracked beam than in an uncracked beam, as indicated in Fig. 2. The ability of the concrete section to carry tensile stress depends on whether or not the section has cracked, ie. on the magnitude of the applied moment, among other things. ΔT is much larger on the uncracked section of Fig. 2b than on the cracked section of Fig. 2c. Existing design procedures for the calculation of long-term deflection fail to adequately model the additional cracking that occurs with time due to ΔT and the gradual breakdown of tension stiffening with time (also due to ΔT), and consequently often greatly underestimate final deformations.

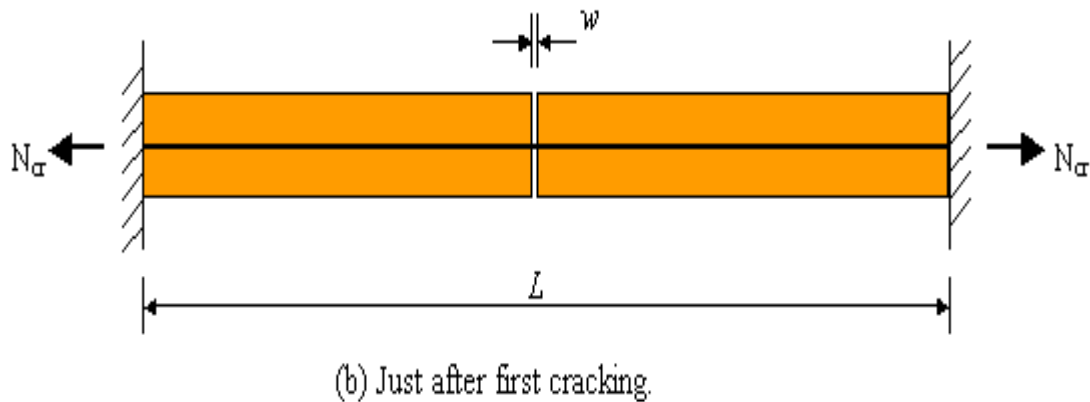
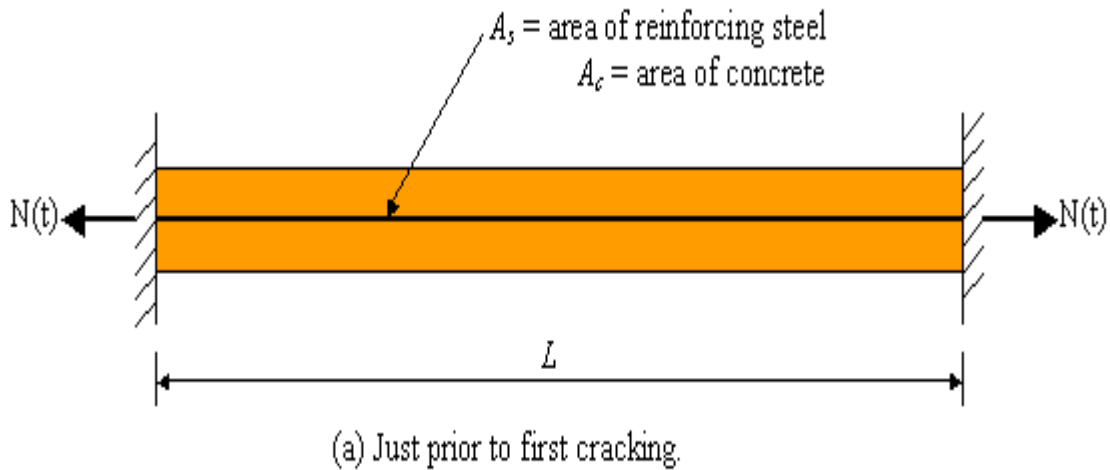
Compressive reinforcement reduces shrinkage curvature. By providing restraint at the top of the section, in addition to the restraint at the bottom, the eccentricity of the resultant tension in the concrete is reduced and, consequently, so is the shrinkage curvature. An uncracked, symmetrically reinforced section will suffer no shrinkage curvature. Shrinkage will however induce a uniform tensile stress which when added to the tension caused by external loading may cause time-dependent cracking.

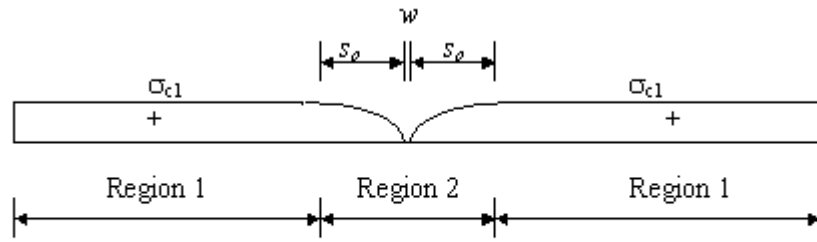
3.4 Shrinkage in a restrained reinforced concrete member (Gilbert, 1992) [8]

Structural interest in shrinkage goes beyond its tendency to increase deflections due to shrinkage warping. External restraint to shrinkage is often provided by the supports of a structural member and by the adjacent structure. When flexural members are also restrained at the supports, shrinkage causes a build-up of axial tension in the member, in addition to the bending caused by the external loads. Shrinkage is usually accommodated in flexural members by an increase in the widths of the numerous flexural cracks. However, for members not subjected to significant bending and where restraint is provided to the longitudinal movements caused by shrinkage and temperature changes, cracks tend to propagate over the full depth of the cross-section. Excessively wide cracks are not uncommon. Such cracks are often called *direct tension cracks*, since they are caused by direct tension rather than by flexural tension. In fully restrained direct tension members, relatively large amounts of reinforcement are required to control the load independent cracking.

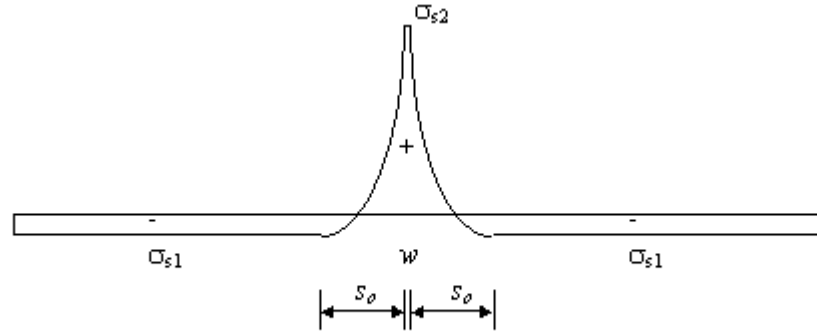
Consider the fully-restrained member shown in Fig. 3a. As the concrete shrinks, the restraining force $N(t)$ gradually increases until the first crack occurs when $N(t) = A_c f_t$, usually within two weeks from the commencement of drying, where A_c is the cross-sectional area of the member and f_t is the tensile strength of the concrete. Immediately after first cracking, the restraining force reduces to N_{cr} , and the concrete stress away from the crack is less than the tensile strength of the concrete. The concrete on either side of the crack shortens elastically and the crack opens to a width w , as shown in Fig. 3b. At the crack, the steel carries the entire force N_{cr} and the stress in the concrete is obviously zero. In the region immediately adjacent to the crack, the concrete and steel stresses vary considerably and there exists a region of partial bond breakdown. At some distance s_o on each side of the crack, the concrete and steel stresses are no longer influenced directly by the presence of the crack, as shown in Figs 3c and 3d.

In Region 1, where the distance from the crack is greater than or equal to s_o , the concrete and steel stresses are σ_{c1} and σ_{s1} , respectively. Since the steel stress (and hence strain) at the crack is tensile and the overall elongation of the steel is zero (full restraint), σ_{s1} must be compressive. Equilibrium requires that the sum of the forces carried by the concrete and the steel on any cross-section is equal to the restraining force. Therefore, with the force in the steel in Region 1 being compressive, the force carried by the concrete ($A_c \sigma_{c1}$) must be tensile and somewhat greater than the restraining force (N_{cr}). In Region 2, where the distance from the crack is less than s_o , the concrete stress varies from zero at the crack to σ_{c1} at s_o from the crack. The steel stress varies from σ_{s2} (tensile) at the crack to σ_{s1} (compressive) at s_o from the crack, as shown.





(c) Average concrete stress just after first cracking.



(d) Steel stress just after first cracking.

Fig. 3 - First cracking in a restrained direct tension member.

To determine the crack width w and the concrete and steel stresses in Fig. 3, the distance s_o over which the concrete and steel stresses vary, needs to be known and the restraining force N_{cr} needs to be calculated. An approximation for s_o maybe obtained using the following equation, which was proposed by Favre et al. (1983) [6] for a member containing deformed bars or welded wire mesh:

$$s_o = d_b / 10 \rho \quad (6)$$

where d_b is the bar diameter, and ρ is the reinforcement ratio A_s / A_c . Base and Murray (1982) used a similar expression.

Gilbert (1992) showed that the concrete and steel stresses immediately after first cracking are

$$\sigma_{c1} = \frac{N_{cr} - \sigma_{s1} A_s}{A_c} = \frac{N_{cr}(1 + C_1)}{A_c}; \quad \sigma_{s1} = \frac{2 s_o}{3L - 2 s_o} \frac{N_{cr}}{A_s} = -C_1 \frac{N_{cr}}{A_s}; \quad \text{and} \quad \sigma_{s2} = \frac{N_{cr}}{A_s} \quad (7)$$

where $C_1 = 2 s_o / (3L - 2 s_o)$. If n is the modular ratio, E_s / E_c , the restraining force immediately after first cracking is

$$N_{cr} = \frac{n \rho f_t A_c}{C_1 + n \rho (1 + C_1)} \quad (8)$$

With the stresses and deformations determined immediately after first cracking, the subsequent long-term behaviour as shrinkage continues must next be determined. After first cracking, the concrete is no longer fully restrained since the crack width can increase with time as shrinkage continues. A state of partial restraint therefore exists after first cracking. Subsequent shrinkage will cause further gradual increases in the restraining force $N(t)$ and in the concrete stress away from the crack, and a second crack may develop. Additional cracks

may occur as the shrinkage strain continues to increase with time. However, as each new crack forms, the member becomes less stiff and the amount of shrinkage required to produce each new crack increases. The process continues until the crack pattern is established, usually in the first few months after the commencement of drying. The concrete stress history in an uncracked region is shown diagrammatically in Fig. 4. The final average crack spacing, s , and the final average crack width, w , depend on the quantity and distribution of reinforcement, the quality of bond between the concrete and steel, the amount of shrinkage, and the concrete strength. Let the final shrinkage-induced restraining force be $N(\infty)$.

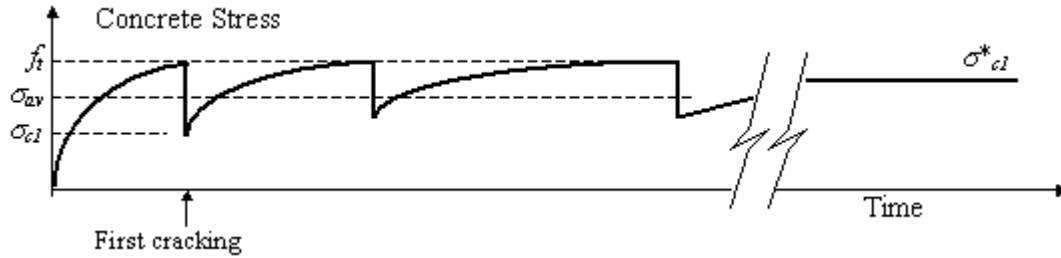


Fig. 4 - Concrete stress history in uncracked Region 1 (Gilbert, 1992) [8]

After all shrinkage has taken place and the final crack pattern is established, the average concrete stress at a distance greater than s_o from the nearest crack is σ^*_{c1} and the steel stresses at a crack and at a distance greater than s_o from a crack are σ^*_{s2} and σ^*_{s1} , respectively. Gilbert (1992) [8] developed the following expressions for the final restraining force $N(\infty)$ and the final average crack width w :

Provided the steel quantity is sufficiently large, so that yielding does not occur at first cracking or subsequently, the final restraining force is given by

$$N(\infty) = -\frac{n^* A_s}{C_2} (\sigma_{av} + \varepsilon^*_{cs} E^*_e) \quad (9)$$

ε^*_{cs} is the final shrinkage strain; E^*_e is the final effective modulus of the concrete and is given by $E^*_e = E_c / (1 + \phi^*)$; ϕ^* is the final creep coefficient; n^* is the effective modular ratio (E_s / E^*_e); $C_2 = 2s_o / (3s - 2s_o)$; and σ_{av} is the average stress in the uncracked concrete (see Fig. 4) and may be assumed to be $(\sigma_{cl} + f_t) / 2$. The maximum crack spacing is

$$s = \frac{2s_o(1 + \xi)}{3\phi^*} \quad (10)$$

and ξ is given by

$$\xi = \frac{-n^* \rho (\sigma_{av} + \varepsilon^*_{cs} E^*_e)}{n^* \rho (\sigma_{av} + \varepsilon^*_{cs} E^*_e) + f_t} \quad (11)$$

The final steel stress at each crack and the final concrete stress in Regions 1 (further than s_o from a crack) are, respectively,

$$\sigma^*_{s2} = N(\infty) / A_s \quad \text{and} \quad \sigma^*_{c1} = N(\infty)(1 + C_2) / A_c < f_t \quad (12)$$

Provided the steel at the crack has not yielded, the final crack width is given by

$$w = - \left[\frac{\sigma_{c1}^*}{E_c^*} \left(s - \frac{2}{3} s_o \right) + \epsilon_{cs}^* s \right] \quad (13)$$

When the quantity of steel is small, such that yielding occurs at first cracking, uncontrolled and unserviceable cracking will result and the final crack width is wide. In this case,

$$\sigma_{c1}^* = \frac{n^* \rho f_y + \epsilon_{cs}^* E_s}{1 + n^* \rho}; \quad \sigma_{s2}^* = f_y; \quad \text{and} \quad \sigma_{c1}^* = \frac{f_y A_s - \sigma_{c1}^* A_c}{A_c} \quad (14)$$

and the final crack width is

$$w = - \frac{\sigma_{c1}^* (3L - 2 s_o) + 2 s_o f_y}{3 E_s} \quad (15)$$

where L is the length of the restrained member.

Numerical Example:

Consider a 5 m long and 150 mm thick reinforced concrete slab, fully restrained at each end. The slab contains 12-mm diameter deformed longitudinal bars at 300 mm centres in both the top and bottom of the slab ($A_s = 750 \text{ mm}^2/\text{m}$). The concrete cover to the reinforcement is 30 mm. Estimate the spacing, s , and final average width, w , of the restrained shrinkage cracks.

Take $\phi^* = 2.5$, $\epsilon_{cs}^* = -600 \times 10^{-6}$, $f_t = 2.0 \text{ MPa}$, $E_c = 25000 \text{ MPa}$, $E_s = 200000 \text{ MPa}$, $n = 8$ and $f_y = 400 \text{ MPa}$. The reinforcement ratio is $\rho = 0.005$ and from Equation 6,

$$s_o = \frac{12}{10 \times 0.005} = 240 \text{ mm.}$$

The final effective modulus is $E_c^* = \frac{25,000}{1 + 2.5} = 7143 \text{ MPa}$ and the corresponding effective modular ratio is $n^* = 28$. The constant $C_1 = 2 s_o / (3L - 2 s_o) = 2 \times 240 / (3 \times 5000 - 2 \times 240) = 0.0331$ and from Equation 8, the restraining force immediately after first cracking is

$$N_{cr} = \frac{8 \times 0.005 \times 2.0 \times 150,000}{0.0331 + 8 \times 0.005 (1 + 0.0331)} = 161,300 \text{ N/m}$$

The steel stress at the crack $\sigma_{s2} = 161300/750 = 215 \text{ MPa}$ and the concrete stress σ_{c1} is obtained from Equation 7:

$$\sigma_{c1} = 161300 (1 + 0.0331) / 150000 = 1.11 \text{ MPa.}$$

The average concrete stress may be approximated by $\sigma_{av} = (1.11 + 2.0)/2 = 1.56 \text{ MPa}$ and from Equation 11:

$$\xi = \frac{-28 \times 0.005(1.56 - 0.0006 \times 7143)}{28 \times 0.005(1.56 - 0.0006 \times 7143) + 2.0} = 0.236$$

The maximum crack spacing is determined using Equation 10:

$$s = \frac{2 \times 240(1 + 0.236)}{3 \times 0.236} = 837 \text{ mm}$$

The constant C_2 is obtained from $C_2 = (2 \times 240)/(3 \times 839 - 2 \times 240) = 0.236$ and the final restraining force is calculated using Equation 9:

$$N(\infty) = -\frac{28 \times 750}{0.236} (1.56 - 0.0006 \times 7143) = 242,670 \text{ N/m}$$

From Equation 12, $\sigma_{s2}^* = 323 \text{ MPa}$, $\sigma_{c1}^* = 1.99 \text{ MPa}$ and, consequently, $\sigma_{s1}^* = -76.4 \text{ MPa}$. The final crack width is determined using Equation 13:

$$w = -\left[\frac{1.99}{7143} \left(839 - \frac{2}{3} \times 240 \right) - 0.0006 \times 839 \right] = 0.313 \text{ mm.}$$

Tables 2 and 3 contain results of a limited parametric study showing the effect of varying steel area, bar size, shrinkage strain and concrete tensile strength on the final restraining force, crack width, crack spacing and steel stress in a 150 mm thick slab, fully-restrained over a length of 5 m.

Table 2 Effect of steel area and shrinkage strain on direct tension cracking ($\phi^* = 2.5$, $f_t = 2.0 \text{ MPa}$ and $d_b = 12 \text{ mm}$)

A_s mm ²	ρ	$\epsilon_{cs}^* = -0.0006$				$\epsilon_{cs}^* = -0.00075$				$\epsilon_{cs}^* = -0.0009$			
		$N(\infty)$	σ_{s2}^*	s	w	$N(\infty)$	σ_{s2}^*	s	w	$N(\infty)$	σ_{s2}^*	s	w
		kN	MPa	mm	mm	kN	MPa	mm	mm	kN	MPa	mm	mm
375	.0025	150	400	-	1.37	150	400	-	2.03	150	400	-	2.68
450	.003	180	400	-	1.35	180	400	-	2.01	180	400	-	2.66
600	.004	240	400	-	1.22	234	390	913	0.49	216	360	717	0.50
750	.005	243	324	837	0.31	220	294	601	0.33	197	264	469	0.34
900	.006	233	259	601	0.23	206	229	427	0.24	179	199	332	0.24
1050	.007	224	214	453	0.18	193	184	320	0.18	161	154	247	0.19
1200	.008	215	170	354	0.14	179	149	248	0.15	143	119	191	0.15

Table 3 Effect of bar diameter and concrete tensile strength on direct tension cracking.

($\phi^* = 2.5$, $\epsilon_{cs}^* = -0.0006$, $A_s = 900 \text{ mm}^2$ and $\rho = 0.006$)

d_b (mm)	$f_t = 2.0 \text{ MPa}$				$f_t = 2.5 \text{ MPa}$			
	$N(\infty)$	σ_{s2}^*	s	w	$N(\infty)$	σ_{s2}^*	s	w

	kN	MPa	mm	mm	kN	MPa	mm	mm
6	237	263	317	0.12	323	359	482	0.14
10	234	260	508	0.19	320	356	758	0.23
12	233	259	601	0.23	319	354	889	0.27
16	232	257	781	0.30	317	352	1141	0.35
20	230	256	956	0.37	315	350	1385	0.42

EARLY Movement in Concrete

Almost as soon as water is added to the mix, a chemical reaction between water and cement

(hydration) is initiated, although its effects may not be apparent for the first few hours. The impact

of this time-dependent reaction on the setting, stiffening, hardening and strength development

of concrete is well documented, but the fact that shrinkage occurs in the first few hours of its life has not been adequately recognised

Recent research sponsored by Cement Concrete & Aggregates Australia (CCAA) at the University

of Queensland^{1,2,3} has shown that early-age shrinkage can develop strains in concrete of similar

magnitude to those resulting from drying shrinkage. These develop at a time when the concrete has very

little tensile strength and hence cracking is likely. Similar cracking may also be what is known

as 'plastic cracking'. This phenomenon is well documented⁴ and is associated with drying

conditions and loss of moisture through the concrete surface. The fact that cracking sometimes

occurs at early ages in very benign environments (eg indoors where drying conditions and moisture

loss are not evident and plastic cracking therefore unlikely) is evidence of the existence of early-age shrinkage.

Early-age Shrinkage:-

Concrete maintains almost semi fluid properties for a few hours. Hydration reactions are very slow

during the first 3-4 hours of a concrete's life but accelerate over the following 8-12 hours when

the concrete changes from a semi fluid state into a more rigid one capable of cracking. The time

period depends on many variables including the constituents of the concrete, the mix proportions and weather conditions.

In freshly consolidated concrete, water in the spaces between the cement and the aggregate

particles exert a positive pore pressure on the mix. As the surface layer of the concrete dries out, this

situation changes and the concrete moves from a saturated to a partially saturated state. This causes

capillary tension in the pore water which creates a capillary suction that draws the solid particles closer together. Where the concrete is restrained from contracting in this manner the shrinkage causes stresses to develop in the external layers. Where this stress exceeds the tensile capacity of

(lbs0.5kg/m²/hr).

(lbs0.5kg/m²/hr).

concrete provides more than adequate internal restraint to cause these surface or near-surface tension stresses to develop. It is principally this internal **restraint** that leads to early-age shrinkage

This internal restraint mechanism supplemented by external restraint from formwork

subgrade friction, member geometry, etc can lead to early cracking

Early-age shrinkage cracks may be obvious immediately or become apparent at a later time

Immediate Cracking: -

Immediate cracking can be evident within a few hours due to the internal

and external restraints producing stresses in the upper layers of the concrete that exceed the tensile capacity of the concrete.

Later Cracking :-

Early-age shrinkage effects can leave a 'weakness' in the concrete that

will provide a strain release point or crack propagation location in the concrete when

drying conditions occur.

INFLUENCES ON EARLY-AGE SHRINKAGE AND CRACKING:-

The following factors have a significant impact on the extent and magnitude of early-age shrinkage and associated cracking

. Restraint Against Movement:-

Restraint against movement is one of the prime tensile stress on the concrete and the likelihood of the concrete

cracking . The greater the restraint, the higher the risk of cracking. All concrete is subject to some form of

external restraint and some level of internal restraint as it dries out

Compaction:-

voids and reducing the early-age shrinkage and potential for cracking. Good compaction increases the tensile strength of the concrete and hence its ability to resist tensile stress without cracking.

Voids near the surface of the concrete are of particular concern as they significantly reduce the tensile capacity of the concrete in that location and can provide sufficient weakness for crack formation.

Bleeding:-

Inadequate bleed water on the surface of concrete will increase the early-age shrinkage and the propensity of the concrete to crack. The rate at which concrete bleeds is more important than the total amount of bleed water in predicting whether or not the concrete will dry, shrink and crack at an early age. While bleed water remains on the surface of the concrete there can be no suction created in the capillaries of the concrete and no danger of early-age shrinkage and cracking

Surface Drying:-

Surfaces that dry in the first few hours will exhibit higher early-age shrinkage and are more prone to cracking. Early-age shrinkage will lead to cracking only if the surface of the concrete is allowed to dry in the early hours of its life. This is of concern in all environmental conditions and not merely in hot or drying weather.

Shrinkage tests:-

Shrinkage tests were conducted following the ASTM C-157 "Standard Test

Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete".

ASTM C-157 specifies that the specimens are to be maintained in an environment

at 73 +/- 3 degrees Fahrenheit and a relative humidity of 50 +/- 4% and that the

air movement past the specimens shall be such that the evaporation is 77 +/-30 mL/(24h) from an atmometer.

The storage conditions at the University met the temperature requirement, but did

not attempt to match the relative humidity or air movement requirements. This

variation was made to reduce the cost of setting up special storage conditions for

this test. Since both the "normal" and the SCC mixes were subjected to the same

conditions, the measured differences in shrinkage still provided a valid basis for

judgment of the SCC mix.

Shrinkage test specimens were 4-inch square prisms 11.25 inches in length and were cast at the Spancrete and County Materials plants. The test specimens were initially subjected to the same curing conditions during the first 24 hours

(temperature and moisture) as used in the highway bridge girder curing. The test prisms were placed in water for 30 minutes before initial measurement.

This initial immersion was a variance from the C-157 procedure with lime water submersion and was deemed appropriate for the purpose of measuring prestress loss in the highway bridge girders.

The initial length readings were taken subsequent to the immersion. ASTM C-157 specifies that after the initial readings, the specimens are to be stored in lime water to 28 days and then in air storage as noted above. However, since the creep test was started at 2 days of age, it was deemed appropriate to deviate from ASTM in this respect. After the prisms were removed from their molds and held in water for 30 minutes, they were then kept at room conditions matching those of the creep specimens - allowing the measurements of the creep specimens to be corrected for the measured shrinkage under the same temperature and humidity conditions.

Succeeding length readings were taken at 4, 8, 14, and 28 days after casting, followed by bi-monthly, then monthly readings using an HM-250D Length Comparator with digital indicator.

Three specimens cast from different batches of each of the four different concrete mixes were measured. (Spancrete had an additional fourth batch described later.)

Figures 1 – 3 show shrinkage specimen preparation and testing.

Creep tests:-

The creep testing generally followed ASTM C-512 “Standard Test Method for Creep in Compression”. ASTM C-512 specifies that the creep test specimens be stored at 73.4 ± 2 degrees Fahrenheit and a relative humidity of $50 \pm 4\%$. The storage conditions for this test varied from those specified in C-512, but aimed to meet the requirements described previously for the shrinkage test specimens. As stated previously, these variations were made to reduce the cost of setting up special storage conditions for this test. Since both the “normal” and the SCC mixes were subjected to the same conditions, the measured differences in creep provide a valid basis for judgment of the SCC mix.

ASTM C-512 specifies that the length between header plates used to apply a constant compression force to the test specimens cannot be greater than 70 inches (5.83 feet). A length of 84 inches (7 feet) was used in these tests to accommodate test specimens in series. The ASTM-specified ages at initial loading (2 days, 7 days, 28 days, 90 days and 1 year) were not used for these tests. The date of initial loading for the primary set of test specimens was selected as 2 days of age to simulate the age at which bridge girders are subjected to prestress. A second set of specimens was loaded at a 28 day reference age for initial loading. Due to initial problems in the creep testing, one set of Spancrete specimens was inadvertently started at 12 days and another at 32 rather than 2 and 28 days. An replacement set of specimens was subsequently obtained (labeled G) from Spancrete and testing was started at 2 days. Thus, for some Spancrete mixes there was one set of specimens started at 2 days and another set at 12 days.

ASTM specifies that the specimens should be loaded at an intensity of not more than 40% of the compression strength at time of loading. Since these tests were simulating prestressed girders, the ASTM load intensity was modified.

The stress in the concrete of prestressed girders, adjacent to the steel strands, varies along the length of a girder and varies with age and applied live loading. At time of prestress transfer, the concrete compression stress near the end of a girder may reach 3800 psi and may be 3500 psi near the center of the girder. Under permanent dead load from the bridge structure, the concrete compression stress at midspan may drop to near zero, while remaining high at the girder end.

For the purpose of this study, (1) the specimens that were loaded at 2 days of age were to be subjected to 3800 psi for the first 28 days. After 28 days, the compression load was to be reduced to 2000 psi to simulate the effect of placing the weight of a concrete deck on bridge girders and reducing the initial compression at the bottom of the girder. (2) The specimens loaded at 28 days age were to receive only the 2000 psi of compressive stress.

Creep measurement were conducted for a period of 1 year after the loading was applied to the specimens.

Six specimens cast from different batches of each of the four different concrete mixes were measured. Half the specimens were to be loaded at 2 days, while the other half of the specimens were loaded at 28 or 32 days. One unplanned set was loaded at 12 days. Figures 4-11 show the creep test setup.

HOW TO PREVENT PLASTIC SHRINKAGE CRACKS

ABSTRACT:

One of the problems that contractors face is to identify the weather conditions and to determine if the concrete placements should proceed. From past research, it has been shown that the three weather conditions (air temperature, humidity and wind velocity) and the concrete temperatures are the best indicators to determine if plastic shrinkage cracks will develop. A Nomograph was developed and published in many

American Concrete Institute (ACI) committee reports as well as by the Portland Cement Association (PCA). Unfortunately, it is difficult to get the required information and the Nomograph is not easy to use.

Plastic shrinkage cracks can be prevented, provided the contractor properly assessed the weather and the fresh concrete conditions. This paper will present an assessment of some of the available inexpensive equipment that can be used by the contractor's field personnel to quickly determine both weather conditions and concrete temperatures. The paper will also include computer programs that use the above information to assist the contractor in making the decisions on what they can do to prevent the formation of plastic shrinkage cracks.

Keywords: plastic shrinkage cracks, weather conditions, Nomograph, concrete temperatures

INTRODUCTION:-

One of the problems contractors face is to reduce the number of plastic shrinkage cracks on their projects. Although the mechanism of what causes plastic shrinkage cracks is well understood and the parameters have been researched and well documented, there is a disconnect in applying this information to the construction site.

MECHANISM OF PLASTIC SHRINKAGE CRACKING:-

The American Concrete Institute (ACI) in ACI 116 defines plastic shrinkage cracking as "cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic." These cracks form because of the loss of bleed water from the surface of the fresh concrete by evaporation. The tensile strength of the fresh concrete is very low since the concrete has not had time to set, thus the volume change caused by this evaporation of the bleed water results in the formation of plastic shrinkage cracks.

The research in the parameters that effect evaporation rates of the bleed water goes

back to Dalton is work in 1802. His research developed the formula for evaporation rate from water surfaces (ponds, reservoirs, lakes, etc). Carl Menzel of the Portland Cement Association simplified the formulae and established the evaporation rates that would result in the formation of plastic shrinkage cracks in fresh concrete.

Delmar Bloem of the National Ready Mixed Concrete Association developed a Nomograph that related the air temperature, relative humidity, concrete temperature and wind velocity parameters, which are relatively easy to measure, to predict the evaporation rates for a concrete surface with bleed water. These Nomographs have been published by many sources such as ACI, “Hot Weather Concrete” (ACI 305) and the PCA, “Design and Control of Concrete Mixtures.”

What Evaporation rate causes plastic shrinkage cracking to form:-

In the United States of America (USA), two values of evaporation rates are used to provide guidance to the contractors on when plastic shrinkage cracks will form.

When the evaporation rates exceed 0.2 lb/ft²/hr (1.0 kg/m²/hr) plastic shrinkage cracks are expected. Precautionary measures are almost manitory. Some of these are erecting a wind screen, cooling the concrete, using a fogging system, and placing concrete at night. When the evaporation rate is between 0.1 and 0.2 lb/ft²/hr (0.5 and 1.0 kg/m²/hr) plastic shrinkage cracking may occur. Since evaporation rates in these ranges can occur and to avoid plastic shrinkage cracks, the above precautionary measures are recommended. When the evaporation rates are 0.1 lb/ft²/hr (0.5 kg/m²/hr), plastic shrinkage cracks are not expected.

WHY IS THE NOMOGRAPH AND FORMULAE NOT WIDELY USED?

1. Contractors, Ready Mixed Concrete Companies personnel, and Designers are unaware of the Nomographs:

Using the Nomograph and formulae are not a part of the ACI’s Building Code, and

its use is not manitory. As stated before, the Nomograph is in many publications, however, most people do not read these until problems occur on their jobsite. From my experience, engineering and construction education at the universities spend little or no time discussing plastic shrinkage cracks. In examining several of the textbooks, commonly used in university concrete materials courses, plastic shrinkage cracking is either omitted or receives cursory coverage. Thus, few people know that the Nomograph or equation exist or have used experience in using them.

2. The Nomograph and equations are not easy to use:

Once a person does find or becomes aware of the Nomograph and the equation, they find that they are not easy to use. The Nomograph is a series of three (3) graphs and the user must go from graph one (1) to graph two (2) to graph three (3). Although this is not difficult, most people do not routinely use these types of graphic techniques and are uncomfortable with this approach. The equations require both understanding and experience in advanced mathematical skills. Again, most people are not experienced in solving an equation that raises a number to the 2.5 power.

Neither the Nomograph nor the equations will allow the user to do a “what if lower the concrete temperatures, how low do the temperatures need to be so that plastic shrinkage cracks do not occur?” It is one thing to assess the current situation. However, the user needs to know how he/she can modify the concrete or the construction environments so that the project will not have plastic shrinkage cracks.

3. Unaware of how to get weather information:

The Nomograph gives exact locations at where weather is to be measured. The wind is to be the average wind speed and is to be measured at 20 inches (500 mm) above the evaporating surface. The air temperature and relative humidity is to be measured at 4 to 6 feet (1.2 – 1.8 meters) above the concrete surface on the

windward side and shielded from the sun.

When consultants are asked to evaluate plastic shrinkage cracking, they have to find the available weather information. Typically, they will use weather information provided by the nearest airport. The airport measures wind speed at 33 feet (10 meters) above the ground. The wind speed on the job site may vary from the airport. Typically, the wind speed is less than the wind speeds measured at the airport. If the construction project is remote from the nearest weather station, the air temperature and the relative humidity measured at the airport may also vary from those that exist on the jobsite. Thus, using weather information from the airport, the Nomograph or the equation may inaccurately predict the evaporation rate and whether plastic shrinkage cracking would occur at jobsites.

COMPUTER PROGRAM:-

Recently, a computer program was developed that makes the Nomograph and the equations easy to use. The user simply enters the air temperature, concrete temperature, relative humidity, and wind velocity. The program will determine the evaporation rate and warn the user if precautionary methods are needed. To further emphasize the fact that some conditions may cause or might cause plastic shrinkage cracking, the answers are color coded. The color codes are similar to a traffic light. The output is in red when plastic shrinkage cracks are expected and precautions against plastic shrinkage cracking are mandatory. This occurs when the evaporation rates are more than 0.2 lbs/ ft²/ hr (1.0kg/m²/hr). The output is in yellow when the plastic shrinkage cracks can occur. This occurs when the evaporation rates are between 0.1 to 0.2 lbs/ ft²/ hr (0.5 to 1.0kg/m²/hr). The output is green is used when the plastic shrinkage cracking is not expected. This occurs when the evaporation rates are less than 0.2 lbs/ ft²/ hr (1.0kg/m²/hr).

The computer program also allows the user to solve problems or do the “what if” questions about plastic shrinkage cracks. If the plastic shrinkage cracking is likely to occur, the user can select one of the variables they would like to control. The program will calculate how much change is required so that plastic shrinkage cracks are not expected or when the evaporation rates are below 0.1lbs/ft²/hr(1bs0.5kg/m²/hr). As part of the research, several handheld weather stations were also examined that allow the user to measure the wind velocity, relative humidity, and air temperature at the appropriate location on the jobsite as stated in the Nomograph. From the research, it was found that each of these pieces of equipment can accurately predict the humidity, air temperature, and wind speed when compared to a certified weather station.