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*Fifth Revision of*
**IS 456 Indian Standard Code of Practice for Plain and Reinforced Concrete**

# Draft IS 456
# Structural Concrete

## *Chapter 6*
## *STRUCTURAL MATERIALS*

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**Bureau of Indian Standards**
*1 Bahadur Shah Zafar Marg*
*New Delhi*

# Chapter 6
# MATERIALS

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# 60. STRUCTURAL MATERIALS

This standard provides minimum requirements that shall be achieved on site by ingredients, grades, testing, physical properties, mix, composition, production and storage of concrete, reinforcing steel and prestressing steel, to meet requirements of design and detailing specified in this standard.

Structural materials shall comply with the requirements laid down in this standard and in other relevant national standards, so that the required performance of the structure is assured as envisaged throughout the *intended design life* of structures built using these structural materials.

Alternative structural materials, different than those specified in this standard, are permissible provided that it can be shown that the materials comply with the relevant performance requirements and the use is approved by the statutory body on the recommendation of a duly constituted professionally competent Peer Review Committee.

# C60. STRUCTURAL MATERIALS

This clause provides details of for: (a) performance of construction materials, (b) standard of workmanship, (c) measure of quality control, and appropriate construction method and maintenance of three sets of primary materials used in the construction of plain, reinforced and prestressed concrete structures. The design intent is deemed to have been met with if the individual materials and their workmanship, quality control and processes are in keeping with the requirements specified in this clause.

The clause permits use of new materials. But, the use is subject to a detailed technical assessment by a professionally competent technical peer group.

## 61. CONCRETE

The *specifications* shall be as specified hereunder for the *ingredients*, *grades*, *methods of testing*, *physical properties*, *mix* and *production* of concrete.

## C61. CONCRETE

Professional practices vary across countries. For concrete constructions in India, the specifications provided in this Clause should be adopted and practiced. These specifications have been cast based on the experience collected so far in the country and the good practices worldwide. These specifications are open to scrutiny; they can be reviewed, in case:

(1) The materials used in the making of concrete, reinforcing steel, prestressing steel and other materials used in the structure adversely affect the *safety*, *functionality*, *sustainability* and *aesthetics* of the structure;
(2) The professional practice experiences *difficulties in adopting these specifications*;
(3) The concrete structures constructed based on the specifications *do not meet the performance requirements*; or
(4) *New knowledge has been gained* leading to improvement in the existing specifications or introduction of new specifications.

**61.1 Ingredients**

The basic ingredients of concrete shall be chosen based on the specifications specified hereunder.

When ingredients not listed hereunder are intended to be used, the implications of the use of the same shall be investigated thoroughly to meet the *performance requirements* laid down in **Clause 4** for the six *Limit States*, namely *Safety*, *Serviceability*, *Durability*, *Robustness*, *Integrity* and *Restorability*. The results of the said investigations on the new material shall be deemed to have passed the performance expectations of this standard, if the same are admitted favorably by the professional peer review and are duly approved by the competent local statutory authority.

**C61.1 Ingredients**

Currently, the specifications are provided for basic ingredients (*i.e., Aggregates*, *Cement*, *Water*, *Grout*, *Admixtures* and *Fibers*) that are available commonly in the country.

The clause is open-minded about the newer products that may be proposed by the Industry overtime. Hence, it lays down the basic expectations and professional process for admitting any new material that is proposed for use in concrete construction.

**61.1.1** *Aggregates*

(a) Normal weight aggregates derived from natural sources and from other than natural sources (manufactured aggregates such as iron slag, copper slag, bottom ash from thermal power plants, recycled concrete aggregates and recycled aggregates), shall comply with the requirements of IS 383. Normal weight aggregates are natural or synthetic with in-place density (unit weight) between 2240 to 2400 kg/m$^3$.

(b) Heavyweight aggregates are natural or synthetic with in-place density (unit weight) more than 2400 kg/m$^3$ and can range up to 4500 kg/m$^3$. Heavy weight aggregate is most commonly used for radiation shielding, counterweights and other applications where a high mass-to-volume ratio is desired. Goethite, Limonite, Barite, Illmenite, Magnetite, Hematite, Ferrophosphorus and Steel are some examples of heavy weight aggregate.

The engineering properties and durability of concrete made with heavy weight aggregates are different than normal concrete. Therefore the properties and design formulae given in this code ( for normal concrete ) shall not be applicable. Specialist literature should be consulted. Also refer cl. ….. on "Special Concretes".

(c) Lightweight aggregates are natural or synthetic with in-place density (unit weight) between 1440 to 1840 kg/m$^3$. The lightweight is due to the cellular or high internal porous structure, which gives this type of aggregate a low specific gravity. The most important aspect of lightweight aggregate is the porosity. They have high absorption values, which requires a modified approach to concrete proportioning. Light weight aggregates namely sintered fly ash aggregates may be used for making light weight structural concrete (a type of special concrete). Sintered flyash coarse aggregates when used shall conform to IS 9142 (Part 2).

The engineering properties and durability of concrete made with light weight aggregates are different than normal concrete. Therefore the properties and design formulae given for normal concrete shall not be applicable. Specialist literature should be consulted to take into consideration the important aspects of design such as development length, durability (cover for fire resistance), shear and torsion resistance of beams, deflection of beams, additional moments in slender columns etc. when lightweight aggregates are used. Also refer details given in chapter on "Special Concretes".

*Commentary: Code gives information and requirements with respect to concretes made with coarse and fine aggregates obtained from natural sources and from other than natural sources (manufactured aggregates such as iron slag, copper slag, bottom ash from thermal power plants, recycled concrete aggregates and recycled aggregates) and the aggregates should conform to IS: 383-2016.*

*IS: 2386 (Parts I to VIII)-1963 gives the methods of tests for aggregates for concrete. The aggregates should be free from deleterious materials such as iron pyrites, coal, mica, shale, clay, alkali, soft fragments, sea shells and organic impurities. IS: 383- 2016 gives the limits of deleterious materials, such as coal, clay lumps, soft fragments, shale and material passing 75micron IS sieve. It draws attention also to the necessity of avoiding use of aggregates containing reactive silica, such as chert and chalcedony. Also soft limestone, soft sandstone or other porous or weak aggregates should not be used for concrete in sea water.*

*Manufactured aggregates shall meet the additional requirements such as Total alkali content as Na2O equivalent, Total sulphate content as SO3, Acid soluble chloride content, calcium oxide, total sulphur, total iron, magnesium oxide, basicity, chlorine, water absorption, specific gravity, etc.as given in IS:383-2016. The engineer-in-charge may get the iron and steel and copper slag aggregates checked for hazardous substances, at appropriate frequency meeting the requirements of IS: 383-2016.*

*Fine aggregates should be free from dust, slit and organic 'impurities. Inadequate washing in some cases leaves clay films over the surface of the aggregates. This should be guarded against, as it prevents adhesion of cement to the aggregate and results in a weak concrete. Clay and silt will increase water content because they are fine materials passing through 75-micron IS sieve. Similarly, the dust produced during the crushing of the aggregates, if left adhering to them in sufficient quantities, may be detrimental to concrete.*

*In considering light weight aggregate concrete, the properties of any particular type of aggregate can be established far more accurately than for most naturally occurring materials and the engineer may, therefore, obtain specific data directly from the aggregate producer. Formula for tensile strength and modulus of elasticity of concrete and the data on creep shrinkage and thermal expansion are likely to* ***be*** *different when lightweight aggregates are used. Among the provisions of the Code, the following may require further consideration where lightweight aggregates are used:*

- *Development length*
- *Durability, especially cover for fire resistance and other aggressive environment*
- *Shear and torsion resistance of beams*
- *Deflection of beams: for example, the span/depth ratios and Additional moments in slender columns*

*It is also to be noted that IS: 9142 Part-2 permits the concrete grades upto M 35 for concrete made with Sintered flyash lightweight coarse aggregates.*

(d) Size *of Aggregate*

The nominal maximum size of coarse aggregate should be as large as possible within the limits specified but in no case greater than one-fourth of the minimum thickness of the member, provided that the concrete can be placed without difficulty so as to surround all reinforcement thoroughly and fill the corners of the form. For most work, 20 mm nominal maximum size aggregate is suitable. Where there is no restriction to the flow of concrete into sections, 40 mm or larger size may be permitted. In high strength concrete, size lower than 20 mm is more suitable. In concrete elements with thin sections, closely spaced reinforcement or small cover, consideration should be given to the use of 10 mm nominal maximum size.

Plums (large aggregate used in mass concrete) above 160 mm and up to any reasonable size may be used in plain concrete work up to a maximum limit of 20 percent by volume of concrete when specifically permitted by the engineer-in-charge. The plums shall be distributed evenly and shall not be closer than 150 mm from the surface.

For heavily reinforced concrete members, the nominal maximum size of the aggregate should usually be restricted to 5 mm less than the minimum clear distance between the parallel bars or 5 mm less than the minimum cover to the reinforcement whichever is smaller.

*Commentary: The Size limitations on aggregates are given in the clause for plain concrete as well as for lightly reinforced sections, and in* ***6.1.1.1*** *for heavily reinforced sections. These limitations are mainly intended to ensure that the bars are encased properly and honeycomb pockets are avoided. The largest possible size, properly graded, should be used in order to reduce the water demand.*

*The clause* ***6.1.1.1*** *should be read in conjunction with clause ------where the spacing between bars is dealt. It will be convenient to decide upon the nominal maximum size of aggregate first and then to decide on the clear spacing between bars or groups of bars. The minimum cover requirements are given in clause ------ and in many circumstances this requirement may be governed by durability or fire resistance requirements.*

**61.1.2** *Cement*

i) For general use in plain, reinforced and prestressed concrete construction, cement used shall be any of the following and the type selected should be appropriate for the intended use :
   a) Ordinary Portland cement conforming to IS 269
   b) Portland slag cement conforming to IS 455
   c) Portland pozzolana cement (fly ash based) conforming to IS 1489 (Part 1)
   d) Portland pozzolana cement (calcined clay based), conforming to IS 1489 (Part 2)

ii) Other cements as given below can be used for specific applications. (Refer 6.1.2.1 to 6.1.2.8) below:
   e) Sulphate resisting Portland cement conforming to IS 12330
   f) Composite cement conforming to IS 16415
   g) Microfine Ordinary Portland Cement conforming to IS 16993
   h) Hydrophobic cement conforming to IS 8043
   i) Low heat Portland cement conforming to IS 12600
   j) Rapid-hardening Portland cement conforming to IS 8041
   k) High alumina cement conforming to IS 6452

l) Supersulphated cement conforming to IS 6909
m) White Cement conforming to IS: 8042

**61.1.2.1 Sulphate Resisting Portland Cement (SRPC):** Sulphate Resisting Portland Cement is a type of Portland Cement in which the amount of tri-calcium aluminate (C3A) is restricted to lower than 5% and 2(C3A +C4AF) lower than 25%. The use of SRPC is particularly beneficial in such conditions where the concrete is exposed to the risk of deterioration due to sulphate attack, for example, in contact with soils and ground waters containing excessive amounts of sulphates as well as for concrete in sea water or exposed directly to sea coast. Sulphate resisting cement is not suitable where there is danger of chloride attack as it leads to corrosion of rebar.

**61.1.2.2** Composite Cement conforming to IS 16415 may be used only for plain concrete construction not containing any reinforcement or embedded metal

*Commentary: Composite cement conforming to IS 16415 is a mixture of Portland cement clinker (IS: 16353-2015), fly ash (IS: 3812 (Part 1) -2013), granulated slag (IS: 12089-1987) and gypsum. Typical range of these components is clinker (35% to 65%), fly ash (15% to 35%), granulated slag (20% to 50%) and gypsum (3% to 5%).*

*Add a para reg. ……………………. (PNO) clinker content - ………….*

**61.1.2.3** Low heat Portland cement conforming to IS 12600 shall be used with adequate precautions with regard to removal of formwork, etc. Low Heat Portland Cement is particularly suited for mass concrete works like making concrete for dams and many other types of water retaining structures, bridge abutments, massive retaining walls, piers and slabs etc.

**61.1.2.4** High alumina cement conforming to IS 6452 may be used only under special circumstances with the prior approval of the engineer-in-charge. Specialist literature may be consulted for guidance regarding the use of these types of cements. This cement is mainly a refractory cement ~~but in some cold regions, this cement may find use as a structural material taking advantage of high heat of hydration and high early strength development.~~

*Commentary: This type of cement undergoes conversion or a sudden change of volume under humid and hot environments (> $18^0$C), thereby leading to loss of strength and disintegration. Therefore, high alumina cement should be used with extreme care and caution, ~~both in workmanship as well as in structural design.~~ High alumina cement should always be used in accordance with the manufacturer's recommendations. It should not be mixed with either other hydraulic cements, lime, calcium chloride or with sea water.*

61.1.2.5 Supersulphated cement is used where concrete is likely to be attacked by sulphate. It is a product of granulated blast furnace slag, calcium sulphate and a small quantity of Portland cement or Portland cement clinker or any other suitable source of lime. This cement has chemical resistance to most of the aggressive conditions generally encountered in construction and to the attack of sulphates in particular. It is used generally for marine works, mass concrete jobs to resist the attack by aggressive waters, reinforced concrete pipes in ground waters, concrete construction in sulphate-bearing soils, in chemical works involving exposure to high concentration of sulphates of weak solutions of mineral acids, underside of bridges, over railways (steam driven locos) and for concrete sewers carrying industrial effluents. Its use is not recommended by IS: 6909- 1973, when the prevailing temperature is above 40°C. It should not be used for steam cured concrete products

**61.1.2.6** Microfine Ordinary Portland Cement conforming to IS: 16993 may be used for special applications like rock grouting, grouting concrete structures and underground construction for leak prevention, soil stabilization, etc.

61.1.2.7 Hydrophobic cement: Hydrophobic Portland Cement is manufactured on special requirement for high rainfall areas to improve the shelf life of the cement.

*Commentary: This cement is obtained by intergrinding ordinary Portland cement clinker with certain hydrophobic agents (such as oleic acid, stearic acid, naphthenic acid, pentachlorphenol, etc) which imparts a water repelling property to the cement. With the use of hydrophobic cements, a longer period for mixing may be necessary. Hydrophobic cement should not be confused with water proofing cements.*

61.1.2.8 Rapid Hardening Portland Cement: Rapid Hardening Portland Cement (RHPC) has similar properties to that of ordinary Portland cement except that RHPC is more finely ground with slightly altered chemical composition.

*Commentary: The RHPC is basically the super fine Portland cement covered under IS: 8041-1990. Rapid hardening cement gains strength more rapidly at earlier ages, but has a strength comparable to that of ordinary Portland cement at 28 days. High strength OPC may also be used in places of RHPC to achieve high early strength.*
*The type of cement selected should be appropriate to the intended use. The different types of cements are generally made by the adjustment in relative proportions of chemical compounds and the fineness to suit the particular requirement. The choice of a particular type of cement may also necessitate modifications in other clauses of the Code. Selection of cement based on suitability for different concrete construction is important for durability considerations of structures. Thus, making wise choice of cement type for particular construction site, prevents structure from deteriorating and saves much repair and rehabilitation cost later. Therefore, selection of cement for the intended use shall be on the basis mechanical, physical, chemical and durability requirements.*
*PSC for example is more suitable for marine and off shore structures prone to high chloride and sulphate attack, sewage disposal treatments works, water treatment plants, constructions which are expected to be attacked by dissolved chlorides and sulphate ions.*

**61.1.3** *Water*

Water used for mixing shall be clean and free from excessive amounts of TDS, oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to steel and setting and strength development of concrete.

**61.1.3.1 Classification of types of water**

In general, the suitability of water for the production of concrete depends upon its origin. The following types can be distinguished.

**Potable water**

This water is considered as suitable for use in concrete if meeting the requirements of IS: 10500 wherein the chloride content shall be limited to 500 PPM and alkalinity is less than or equal to 25 ml (6.1.3.2). Such water needs no further testing. In case of any doubt it shall be tested for conformity according to clause 6.1.3.2.

**Natural surface water:** This water if found suitable can be used in concrete, but shall satisfy the requirements of clause 6.1.3.2.

Treated Water: Appropriately treated water from industrial, municipal and other sources can be used in concrete, but shall satisfy the requirements of clause 6.1.3.2.

**Water recovered from processes in the concrete industry:** The water sources given below recovered from process in the concrete industry is normally suitable for use in concrete, but shall satisfy the requirements of clause 61.1.3.2.

- water that was part of surplus concrete,
- water used to clean the inside of the stationery mixers, mixing drums of truck mixer or agitators and concrete pumps,
- process water from sawing, grinding and water blasting of hardened concrete,

The water may be taken from:

- Basins provided with suitable equipment that distributes the solid matter evenly throughout the water;
- Sedimentation basins or similar installations, provided the water is left in the basin for a sufficient amount of time to allow the solids to settle properly.

Water recovered from process in the concrete industry contains varying concentrations of very fine particles, the size of which is generally less than 0.25 mm. Water recovered from processes in the concrete industry or combined water may be used as mixing water for concrete with or without reinforcement or embedded metal and also for pre-stressed concrete, provided the following requirements are met:

- The additional mass of solid material in the concrete resulting from the use of water recovered from processes in the concrete industry shall be less than 1 % mass fraction of the total mass of aggregates present in the concrete
- The additional mass of solid material in the concrete resulting from the use of water recovered from processes in the concrete industry shall be less than 1 % mass fraction of the total mass of aggregates present in the concrete
- The amount of recovered water shall be spread as evenly as possible over a day's production

**Water from underground sources:** This water can be suitable for use in concrete, but shall satisfy the requirements of clause 6.1.3.2.

**Sea water or brackish water:** Sea water or brackish water shall not be used for mixing of concrete because of presence of harmful salts. Desalinated water conforming to clause 6.1.3.2 can be used.

Recycled water: Water recovered from concrete production operation or water generated from sewage, greywater or stormwater systems and treated to a standard that is appropriate for its intended use. Recycled water can be used for making and curing of concrete provided the water is meeting the specifications as mentioned above in clause 6.1.3.2 and 6.1.3.3.

## 61.1.3.2 Testing and Requirements of Water

(a) The water shall be examined in accordance with the test procedures stated in Table 1. Water not in accordance with one or more of the requirements in Table below may be used only if it can be shown to be suitable for use in concrete in accordance

**Table—1**
**Requirements and test procedures for preliminary inspection of mixing water**

| Sl.No. | Parameter | Requirements | |
|---|---|---|---|
| 1 | Oils and fats | Not more than visible traces | **6.1.3.2.1(a)** |
| 2 | Detergents | Any foam should disappear within 2 min. | |
| 3 | Colour | Water not from sources classified as potable: the colour shall be assessed qualitatively as pale yellow or pale | |
| 4 | Odour | Water from sources classified as potable: no smell, except the odour allowed for potable water and a slight smell of cement; where blast-furnace slag is present in the water, a slight smell of hydrogen sulfide | |
| | | Water from other sources: no smell, except the odour allowed for potable water; no smell of hydrogen sulfide after addition of Hydrochloric Acid | |
| 5 | Acids | pH≥6 | |
| 6 | Humic Matter | The colour shall be assessed qualitatively as yellowish / brown or paler after addition of NaOH | **6.1.3.2.1(b)** |

A small subsample shall be assessed as soon as possible after sampling for oil and fats, detergents, colour, suspended matter, odour and humic matter. Bring any material that has settled back into suspension by shaking the sample. Pour 80 ml of the sample into a 100 ml measuring cylinder. Seal with a suitable stopper and shake the cylinder vigorously for 30 s. Smell the sample for any odours other than those of clean water. If in doubt about the odour, test the water for its odour level in accordance with national regulations for potable water. The odour level of the water shall be lower than the maximum level accepted for potable water. Observe the surface for foam. Set the cylinder in a place free from vibration and allow to stand for 30 min. After 2 min, check the sample for the continuing presence of foam and signs of any oils or fats. After 30 min have elapsed, observe the apparent volume of the settled solids and the colour of the water. Measure the pH using indicator paper or a pH meter. Then, add 0.5 ml hydrochloric acid, mix and then smell or test for the presence of hydrogen sulfide

Put 5 ml of the sample into a test tube. Bring it to a temperature between 15 °C and 25°C by allowing it to stand indoors. Add 5 ml of 3 % sodium hydroxide solution, shake and leave for 1 h. Observe the colour.

**61.1.3.2.1** Permissible limits for solids shall be as given in Table 2.

**Table 1 Permissible Limit for Solids for Mixing Water**

| Sl No | Tested as per | Permissible Limit, Max |
|---|---|---|
| Organic | IS 3025 ( Part 18 ) | 200 mg/l |
| Inorganic | IS 3025 ( Part 18 ) | 3000 mg/l |
| Sulphate (as SO3) | IS 3025 ( Part 24 ) | 400 mg/l |
| Chloride ( as Cl) | IS 3025 ( Part 32 ) | 2000 mg/l for concrete not containing embedded steel and 500 mg/l for reinforced concrete work. |
| Suspended Matter | IS 3025 ( Part 17 ) | 2000 mg/l |
| Note: 1000 mg/l chloride in water for reinforced concrete work can be permitted provided the chloride content of hardened concrete is checked at the time of concrete mix design and conforms to requirement of total chloride content in the concrete as per this code. (refer cl…… - durability chapter.) | | |

**61.1.3.2.2** If it is expected to use potential alkali aggregate reactive aggregates in concrete, and exposure conditions can promote deleterious reaction, the water shall be tested for its alkali content. The equivalent sodium oxide content shall not exceed 1500 mg/l unless it can be shown that the alkali content of the concrete does not exceed the maximum value recommended. If these limits are exceeded, the water may be used only if it can be shown that actions have been taken to prevent deleterious alkali-silica reactions. (Test method……)

**61.1.3.2.3** The pH value of water shall be not less than **6**

**61.1.3.2.4** Further for alkalinity and acidity, following concentrations give the maximum permissible values:

a) To neutralize 100 ml sample of water, using phenolphthalein as an indicator, it should not require more than 5 ml of 0.02 normal NaOH. The details of test are given in **8.1** of IS 3025 (Part 22).

b) To neutralize 100 ml sample of water, using mixed indicator, it should not require more than 25 ml of 0.02 normal $H_2SO_4$. The details of 'test shall be as given in 8 of IS 3025 (Part 23).

However, in case of doubt regarding development of strength and setting of concrete due to alkalinity/acidity or higher organic matter the suitability of water for making concrete shall be ascertained by the compressive strength and initial setting time tests specified below:

Average 28 days compressive strength of at least three 150 mm concrete cubes prepared with water proposed to be used shall not be less than 90 percent of the average of strength of three similar concrete cubes prepared with distilled water. The cubes shall be prepared, cured and tested in accordance with the requirements of IS 516.

The initial setting time of test block made with the appropriate cement and the water proposed to be used shall not be less than 30 min and shall not differ by ± 30 min from the initial setting time of control test block prepared with the same cement and distilled water. The test blocks shall be prepared and tested in accordance with the requirements of IS 4031 (Part 5).

In addition to the requirement for the alkalinity/ acidity or higher organic matter tests mentioned above, the same water with higher alkalinity /acidity or higher organic matter shall be used for concrete mix design in the laboratory for adoption at construction site.

**61.1.3.2.5** A suitable means of ensuring uniform distribution of the solid material in recovered water with a density greater than 1.01 kg/l shall be provided. Water with a density less than or equal to 1.01 kg/l may be assumed to contain negligible amounts of solid material

**61.1.3.3 Curing Water**

Water suitable for making concrete is also suitable for curing purpose. However water meeting following requirements can also be used for curing.

**Testing and Requirements of Curing Water**

Permissible limits for solids shall be as given in Table 3. Water used for curing should not produce any objectionable stain or unsightly deposition on the concrete surface. The presence of tannic acid or iron compounds is objectionable.

**Table 3**
**Permissible Limit for Solids for Curing Water**

| **Sl No** | **Tested as per** | **Permissible Limit, Max** |
|---|---|---|
| Organic | IS 3025 ( Part 18 ) | 200 mg/l |
| Inorganic | IS 3025 ( Part 18 ) | 3000 mg/l |
| Sulphate (as $SO_3$) | IS 3025 ( Part 24 ) | 800 mg/l |
| Chloride (as Cl) | IS 3025 ( Part 32 ) | 1000 mg/l for reinforced concrete or prestressed concrete work and 2000 mg/l for concrete not containing embedded steel. |
| Suspended Matter | IS 3025 ( Part 17 ) | 2000 mg/l |

**61.1.3.3.1 Sampling**

A sample of water not less than 5 liters shall be taken. The sample of water taken for testing shall represent the water proposed to be used for concreting, due account being paid to seasonal variation. The sample shall be stored in a clean container previously rinsed out with similar water. Water from different sources permitted in the code shall be tested at frequency of every three months to have clear insight into the fluctuation of water composition for both mixing and curing water. Thereafter, if fluctuation is less, the frequency may be increased to six months for both mixing and curing water.

**C61.1.3** *Water*

The quality of the mixing water for production of concrete can influence the setting time, the strength development of concrete and the protection of the reinforcement against corrosion.

Marsh waters, mine and colliery waste waters, several industrial waste waters and sea water are not likely to meet the requirements of the clause; they should be used only after careful consideration. Mineral oil (not mixed with animal or vegetable oils) in concentrations greater than 2% by weight of cement reduce the concrete strength by more than 20%. The limits of acids and alkalis given are on the conservative side. Any salt that causes durability problems, sulphates and chlorides are more commonly encountered and the limits for the same are given in **Table 6.3(A)**.

Water containing sugar may not have an adverse effect on concrete strength, if the sugar content (in mixing water) is less than *500 ppm*. Organic materials, *e.g.,* algae, present in mixing water cause excessive reduction in strength. Generally, sea water is not recommended for mixing. Sea water used for mixing may show efflorescence in concretes, and may corrode the reinforcement bars in reinforced concrete. Most sea waters will not meet the limit on chlorides prescribed in **Table 6.3(A)**.

**61.1.5** *Admixtures*

Admixtures can be used to:

(1) Alter the *composition* of concrete by replacing some of the cement in the concrete (and are called *Mineral Admixtures*

(2) Guide or enhance a certain *function* of concrete (and are called *Chemical Admixtures*).

Chemical admixtures can be achieved by a spectrum of materials and manufacturing processes, but shall comply with **IS 9103**. Further:

(1) Previous experience with and data on such materials shall be considered in relation to the likely standard of supervision and workmanship to the work being specified;

(2) The workability, compressive strength and the slump loss of concrete with and without the use of admixtures shall be established during the trial mixes before use of admixtures.

(3) The relative density of liquid admixtures shall be checked for each drum containing admixtures and compared with the specified value before acceptance.

(4) If two or more admixtures are used simultaneously in the same concrete mix, data shall be obtained to assess their interaction and to ensure their compatibility.

(5) Admixtures shall not impair durability of concrete nor combine with the constituent to form harmful compounds nor increase the risk of corrosion of reinforcement.

(6) The chloride content of admixtures shall be independently tested for each batch before acceptance.

**(a) Mineral Admixtures** *(Supplementary Cementitious Materials-SCMs)*

SCMs as given below may be used as part replacement of cement. SCMs can be pozzolanic material or reactive mineral admixture or combination of both.

**a)** *Fly ash (pulverized fuel ash)*

Fly ash conforming to IS 3812 (Part-1) may be used as part replacement of ordinary Portland cement. For concrete having steel, the replacement level of flyash shall be restricted to 35% by weight of cementitious content wherein fly ash shall have fineness of 280 $m^2/kg$ and above and Lime Reactivity is 4.5 $N/mm^2$ or more.

**b)** *Silica fume*

Silica fume conforming to IS 15388 may be used as part replacement of cement. Silica fume is usually used in proportion of 3 to 10 percent of the cementitious content of a mix.

The total replacement by fly ash and silica fume shall not be more than 35 percent for reinforced cement concrete. The total replacement by GGBF slag and silica fume/UFGBS shall not be more than 60 percent for reinforced cement concrete.

The above limits shall also apply where PPC or PSC is used. The flyash or slag content mentioned on the bag shall be considered.

**c)** *Rice husk ash*

Rice husk ash giving required performance and uniformity characteristics may be used with the approval of the Engineer-in-Charge, as part replacement of ordinary Portland cement only. The replacement level shall me maximum upto 20 percent. The lime reactivity shall be 4.5 N/mm$^2$ or more and Loss on Ignition value shall be lesser than 5 percent.

NOTE – Rice husk ash is produced by burning rice husk and contains large proportion of silica. To achieve amorphous state, rice husk may be burnt at controlled temperature. It is necessary to evaluate the product from a particular source for performance and uniformity since it can be as deleterious as silt when incorporated in concrete. Water demand and drying shrinkage should be studied before using rice husk.

**d)** *Metakaolin*

Metakaolin conforming to IS 16354 may be used as pozzolanic material in concrete as part replacement of ordinary Portland cement only. The replacement level shall be maximum up to 15 percent.

**e)** *Ground Granulated Blast Furnace Slag*

Ground granulated blast furnace slag conforming to IS 16714 may be used as part replacement of Ordinary Portland Cement. For concrete having steel, the replacement level of *Ground Granulated Blast Furnace Slag* shall be restricted to 50% by weight of cementitious content.
"For resistance to sulphate and alkali aggregate reaction, GGBS up to 60 % can be used".

**f)** Ultrafine Ground Granulated Blast Furnace Slag
Ultrafine Ground Granulated Blast Furnace Slag conforming to IS 16715 may be used as part replacement of cement. The replacement level shall be maximum up to 20 percent.

CAUTIONARY NOTE –
The use of more than one type of mineral admixture with ordinary Portland cement or a mineral admixture with blended cements such as Portland pozzolana cement and Portland slag cement can be permitted only in plain concrete, after necessary trials. However, silica fume may be used as additional mineral admixture in reinforced concrete also subject to limits specified above. Uniform blending of the mineral admixtures with the cement should be ensured

***Commentary:***

- *Mineral admixtures when used with proper proportioning of concrete mix, can help improve the performance of concrete.*
- *For concrete made with mineral admixtures, the setting time and rate of gain of strength may be different from those of concrete made with ordinary Portland cement alone. Cognizance of such modified properties shall be taken into account in deciding de-shuttering time, rate of movement of formwork in slipform construction, initial time of prestressing, longer curing period and for early age loading. The compatibility of chemical admixtures and cementitious materials should be ensured by trials.*
- *Concrete containing high amount of mineral admixtures such as fly ash and ground granulated blast furnace slag, may increase carbonation in case of reinforced concrete.*
- *Concrete containing mineral admixtures may exhibit an increase in plastic shrinkage cracking because of its low bleeding characteristics. The problem may be avoided by ensuring that such concrete is protected against drying, both during and after finishing.*
- *Some other properties of concrete such as modulus of elasticity, tensile strength, creep and shrinkage are not likely to be significantly different. For design purposes, it will be sufficiently accurate to adopt the same values as those used for concrete made with ordinary Portland cement alone.*
- *Mixes that contain very fine mineral admixture such as silica fume, can be sticky and difficult to finish.*
- *Concrete made using blended cements such as Portland pozzolana cement and Portland slag cement shall also adhere to 5.2.3.1, 5.2.3.2 and 5.2.3.3.*

**(b) Chemical Admixtures**

Chemical Admixtures as mentioned in **Table 6.4** if used shall comply with **IS 9103**. Previous experience with and data on such materials should be considered in relation to the likely standards of supervision and workmanship to the work being specified.

Admixtures should not impair durability of concrete nor combine with the constituent to form harmful compounds nor increase the risk of corrosion of reinforcement. The workability, compressive strength and the slump loss of concrete with and without the use of admixtures shall be established during the trial mixes before use of admixtures. The relative density of liquid admixtures shall be checked for each drum containing admixtures and compared with the specified value before acceptance. The chloride content of admixtures shall be independently tested for each batch before acceptance. If two or more admixtures are used simultaneously in the same concrete mix, data should be obtained to assess their interaction and to ensure their compatibility.

The amount of admixture added to a mix shall be recorded in the production record. Additional dose of admixture may be added at project site and mixed adequately in mixer itself to regain the workability of concrete, if necessary, with the mutual agreement between the producer/supplier and the purchaser/user of concrete. But, the producer/supplier shall assure the ultimate quality of concrete supplied by him and maintain record of quantity and time of addition.

Shrinkage compensating admixtures *shall not be used* in structural concrete.

**Table 6.4**: Chemical Admixtures: *Types*, *Effects*, *Benefits* and *Typical Active Constituents*

| *Admixture type* | *Effects and Benefits* | *Typical Active Constituents* |
|---|---|---|
| Air-entraining | Improves durability in freezing and thawing, deicer, sulfate, and alkali-reactive environments<br><br>Improves workability | Salts of wood resins,<br>Some synthetic detergents,<br>Salts of sulfonated lignin,<br>Salts of petroleum acids,<br>Salts of proteinaceous material,<br>Fatty and resinous acids and their salts,<br>Tall oils and gum rosin salts,<br>Alkylbenzene sulfonates,<br>Salts of sulfonated hydrocarbons |
| Retarding | Delays the setting of cement paste, and hence of mixtures containing cement (*e.g.,* mortar or concrete) | Unrefined Lignosulphonate containing Sugars<br>Modified Derivatives of unrefined Lignosulphonate containing Sugars<br>Hydroxycarboxylic Acids and their Salts,<br>Carbohydrates including Sugars,<br>Heptonates related to Sugars and Starches |
| Accelerating | Accelerates setting and early-strength development | Calcium chloride<br>Triethanolamine<br>Sodium thiocyanate<br>Sodium formate or Calcium formate<br>Sodium nitirite or Calcium nitrite<br>Calcium nitrate<br>Aluminates<br>Silicates |
| Water-reducing | Reduces water content at least 5% | Lignosulfonic acids and their salts<br>Hydroxylated carboxylic acids and their salts<br>Polysaccharides<br>Melamine polycondensation products<br>Naphthalene polycondensation products<br>Polycarboxylates |
| High-range water-reducing : *Normal and Retarding* | Reduces water content by at least 20 to 40%, increases slump, increases flowability of concrete (used in *Self-Consolidating Concrete*) | Melamine sulfonate polycondensation products<br>Naphthalene sulfonate polycondensation products<br>Polycarboxylates |
| Shrinkage-reducing | Reduces drying shrinkage up to 30–50% | Polyoxyalkylene alkyl ether<br>Propylene glycol |
| Corrosion-inhibiting | Reduces significantly rate of steel corrosion and extends time for onset of corrosion | Amine carboxylates aminoester organic emulsion<br>Calcium nitrite<br>Organic alkyidicarboxylic |

| | | |
|---|---|---|
| | | Chromates<br>Phosphates<br>Hypophosphites<br>Alkalis<br>Fluorides |
| Permeability-reducing admixture | | |
| *Non-hydrostatic conditions* (PRAN) | Water-repellent surface, reduced water absorption | Long-chain fatty acid derivatives (stearic, oleic, caprylic capric)<br>Soaps and oils (tallows, soya-based)<br>Petroleum derivatives (mineral oil, paraffin,)<br>Fine particle fillers (silicates, talc) |
| *Hydrostatic conditions* (PRAH) | Reduced permeability, increased resistance to water penetration under pressure. | Crystalline hydrophilic polymers (latex, water-soluble, or liquid polymer). |

***Commentary:*** *Chemical Admixtures are added to the concrete mix before or during mixing, in order to modify one or more of the properties of fresh or hardened concrete. They confer certain beneficial effects to concrete, including frost resistance, sulfate resistance, controlled setting and hardening, improved workability, increased strength, etc.*

*Chemical admixtures namely superplasticisers functions as in the increment in slump value, improves the workability of concrete and to reduce the water amount for same workability. Chemical admixtures also have functionality of either fasten setting of concrete and helps in gaining high early-strength development (as an accelerator) or delaying its setting (as a retarder).*

*Air entraining agents which entrain air bubbles of about 0.1 mm diameter. These bubbles have the effect of increasing the fluidity of the mix and also increase the resistance to frost damage by providing unsaturated void space. Air entrainment is therefore used extensively in any climate where wet concrete may be subjected to freezing and thawing.*

***Corrosion-inhibiting admixtures*** *fall into the specialty admixture category and are used to slow corrosion of reinforcing steel in concrete. Corrosion inhibitors can be used as a defensive strategy for concrete structures, such as marine facilities, highway bridges, and parking garages, that will be exposed to high concentrations of chloride.*

*Other specialty admixtures include shrinkage-reducing admixtures and permeability reducing admixtures. The shrinkage reducers are used to control drying shrinkage and minimize cracking, while permeability reducing admixtures reduces permeability and increases resistance to water penetration under pressure.*

*Sometimes problems due to cement admixture compatibility are experienced. An admixture that produces all the desired effects with one cement may not do the same with another cement. This problem is particularity experienced when the supply of cement and/or admixture is changed midway through a project. Cement admixture incompatibility may result in rapid loss of workability, bleeding and segregation of concrete, acceleration/retardation of setting, low rates of strength gain and entertainment of air. Compatible combination of cement and Superplasticiser may be selected by trials or by simple tests like marsh cone test.*

*Chemical admixtures that contain relatively large amounts of chloride may accelerate corrosion of prestressing steel. In case of reinforced concrete, to minimize the chances of deterioration of concrete, the total chloride content in the concrete should be limited as specified in IS: 456.*

**61.1.6** *Fibers*

Fibers may be added to concrete for special applications to enhance select properties. But, such fibers shall be approved by the Competent Authority on the recommendation of a duly constituted competent *Peer Review Committee*.

In any case, the presence of fibers shall not be taken advantage of in the estimation of strength to meet the requirements of *Limit State of Safety* specified in **Clause 83**.

**C61.1.6** *Fibers*

Fibers available in the market are of many types and have different properties. Detailed studies are required to establish the suitability of each fiber type.

The use of fibers of short lengths assists in reducing crack widths, when the structure is subjected to *service loads* (whose combinations are specified in **Table 5.4**). But, when the structure is subjected to *factored loads* to meet requirements of *Limit State of Safety* (whose combinations are specified in **Table 5.3**), the crack widths can be large and sometimes larger than the length of the fibers used. Therefore, the use of fibers is meant for improving local behaviour and not global behaviour. Fibers, which get dissolved in concrete after few years, should not be used.

**61.2 Grades**

The provisions of this standard shall be admissible to grades of concrete listed in **Table 6.1**. The *Characteristic Compressive Strength* mentioned therein shall be taken as the strength of concrete below which not more than 5% of the test results are expected to fall, when finalizing the design mix of concrete.

Further:

(1) Concrete of grades lower than those given in **Table 6.5** shall be used only as *lean concrete* meant to be a leveling course, foundation for masonry boundary walls and other small temporary RC construction of single storey, provided the strength of such concrete is sufficient to resist the loads appearing on the concrete.

(2) For concrete using mineral admixtures and those using high early strength cements, the properties of setting time and time dependent strength gain are different from those of standard and ordinary concretes. Cognizance of such modified properties shall be taken in deciding de-shuttering time, curing period and early age loading.

The minimum grade of concretes for *plain concrete* (PC), *reinforced concrete* (RC) and *prestressed concrete* (PSC) shall be taken as per **Table 85.5**

**Table 6.5**: Grades of Concrete admissible as per this Standard

| *Group* | *Grade Designation* | *Specified Characteristic Compressive Strength (MPa) based on strengths of 150mm cubes at 28 days* |
|---|---|---|
| Ordinary Concrete | M10 | 10 |
| | M15 | 15 |
| | M20 | 20 |
| Standard Concrete | M25 | 25 |
| | M30 | 30 |
| | M35 | 35 |
| | M40 | 40 |
| | M45 | 45 |
| | M50 | 50 |
| | M55 | 55 |
| | M60 | 60 |
| High Strength Concrete | M65 | 65 |
| | M70 | 70 |
| | M75 | 75 |
| | M80 | 80 |
| | M85 | 85 |
| | M90 | 90 |
| | M95 | 95 |
| | M100 | 100 |

## C61.2 Grades

The grade of concrete is an identifying number, which is numerically equal to the characteristic strength at 28 days expressed in *MPa*. In the designation of concrete mix, M refers to the mix and the number to the specified *Characteristic Compressive Strength* of 150 mm size cubes at 28 days from the day of casting of concrete, expressed in *MPa*.

The principal strength of concrete is its strength in compression ascertained from direct compression test conducted on 150mm concrete cubes at 28 days after the day of casting. Also, by definition, the *Characteristic Strength* requires more than 20 cubes, so that each cube represents less than 5% of samples. Typically, if 25 samples are used to establish the characteristic compressive strength of concrete, one cube represents to 4% of the samples. Thus, one cube can fall below the characteristic strength. This exercise is performed during the process of finalising the *design mix* of concrete to be used at the particular site for a specific project. The use of cube strength at 28 days for specifying the grade designation has arisen out of convenience as major part of the long term strength of concrete made with normally used cements which is attained at this age. But, if *high alumina cement* is used, the strength development at earlier ages also may have to be specified.

The *Design Engineer* should decide the grade of concrete required for each part of the structure, and recognize the influences of the limitations (as specified in the relevant clauses of this standard) of the constituent materials and mix proportions on the design quantity being estimated.

~~Generally, minimum grades of concretes are:~~
~~(1) M20 for flexural members.~~
~~(2) M20 or higher in members when concrete is expected to come in direct contact with sea water or to be exposed directly along the sea coast.~~

The use of 28-day cube strength for specifying the grade designation has arisen out of convenience as major part of the long-term strength of concrete made with normally used cements, which is attained at this age. But, if *High Alumina Cement* is used, the strength development at earlier ages also should be specified. The concrete is designated by group and its grade designation based on characteristic strength as given in **Table 6.5**. Three groups of concrete are: *Ordinary*, *Standard*, and *High Strength Concrete*. Further:

(1) *Ordinary Concrete* is made on the basis of *nominal mix* or *design mix* proportioned by weight of its ingredients. In addition to cement, aggregates and water, it may or may not contain *chemical admixtures*.
(2) *Standard Concrete* is made on the basis of design mix proportioned by weight of its ingredients. In addition to cement, aggregates and water, it contains chemical admixtures to achieve certain target values of various properties in fresh condition, achievement of which is monitored and controlled during production by suitable tests. Generally, concretes up to strength Grade M60 are included in this type.
(3) *High Strength Concrete* is similar to standard concrete, but contains additional one or more mineral admixtures providing binding characteristics and partly acting as inert filler material which increase its strength, reduce its porosity and modify its other properties in fresh as well as hardened condition. Concretes up to Grade M100 are included in this type.

**61.2.1** *Standard and High Strength Concretes*

Concretes of grades up to *60MPa* shall be termed as *Standard Concretes* and those of *65 MPa & higher* as *High Strength Concretes*.

**C61.2.1** *Standard and High Strength Concretes*

On the stress-strain curve (**Figure C6.1**), after the peak stress is reached, the strength drop is:

(1) Gradual in normal strength concretes, and

(2) Sudden in high strength concretes.

Clearly, high strength concretes are *more brittle* than the normal strength concretes, and should be used with care and caution, only by competent engineers.

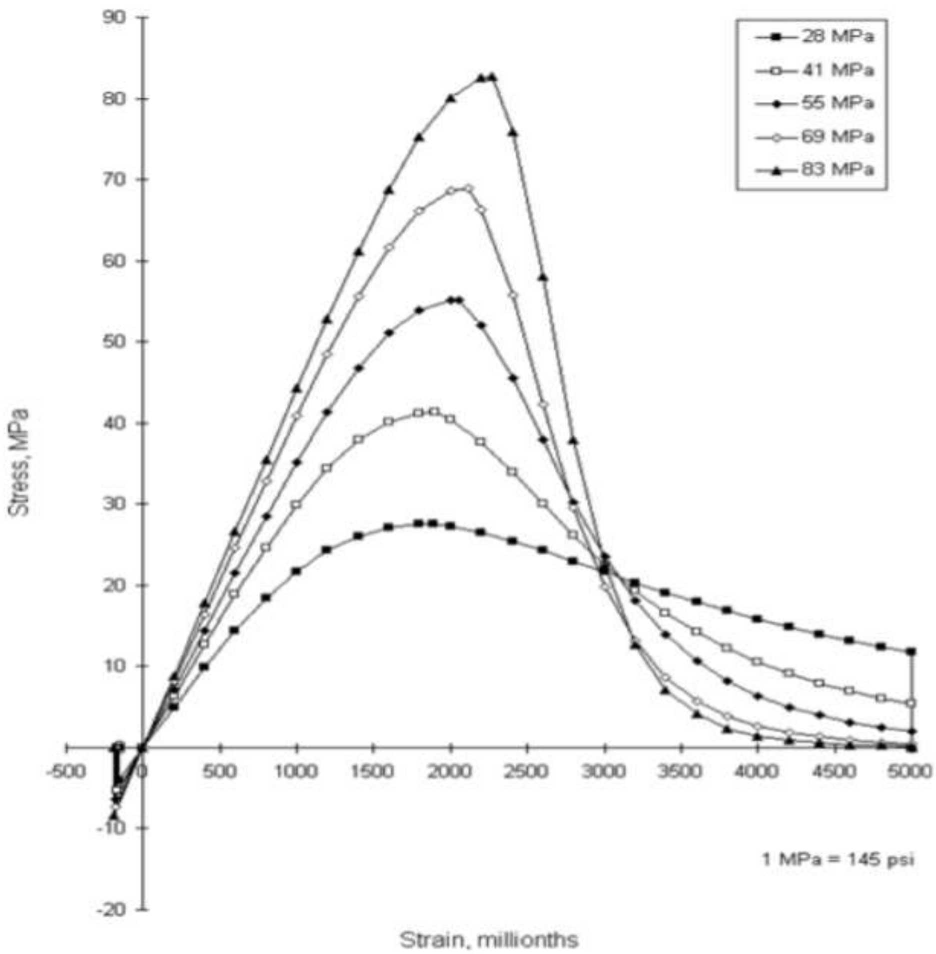



**Figure C6.1:** Stress-strain curve of concretes of different compressive strengths
**(REDRAW USING DATA FROM INDIA, and DROP THE REFERENCE)**

M M Islam
Critical investigation of the Finite Element Models of steel fiber Reinforced Concrete (SFRC): Evaluation Of The Governing Parameters to predict the Flexural Capacities
https://www.researchgate.net/figure/277301966_fig2_Figure-4-Typical-uniaxialcompressive-and-tensile-stress-strain-curve-for-concrete

**61.3 Concrete Mix**

*61.3.1 Mix Proportion*

The mix proportions shall be selected to ensure required:

(a) Workability of the *fresh concrete*, and

(b) Strength, durability and surface finish of *hardened concrete*.

The determination of the *proportions* of cement, aggregates and water to attain the required strengths shall be made by:

(a) Designing concrete mix; such concrete is called *Design Mix Concrete*, or

(b) Adopting predetermined concrete mix; such concrete is called *Nominal Mix Concrete*.

*Design Mix Concrete* shall be preferred to *Nominal Mix Concrete*. When the former cannot be used for any reason in projects with grades of M20 or lower; the latter may be used with the permission of *Engineer-in-Charge*.

**61.3.2 Information required for Concrete Mix Design**

While specifying a *particular grade of concrete for Concrete Mix design,* the following information shall be included:

(a) Type of mix, *i.e.,* design mix concrete or nominal mix concrete;

(b) Mix proportion (for nominal mix concrete);

(c) Grade designation;

(d) Type and grade of cement;

(e) Minimum and maximum cement content (for design mix concrete);

(f) Type of fine aggregate, including whether it is natural sand, crushed stone, gravel sand, mixed sand, or manufactured;

(g) Maximum nominal size of aggregate;

(h) Type of coarse aggregate, including whether it is angular, sub-angular or gravel with some crushed particles, rounded gravel or manufactured;

(i) Maximum water-cement ratio; Workability;

(j) Exposure conditions as per **Clause 84.2**;

(k) Use of admixture, and if yes, whether mineral or chemical admixture (including its type, extent and conditions of use);

(l) Maximum temperature of concrete at the time of placing;

(m) Method of placing, including workability required at the time of placement;

(n) Degree of supervision;

(o) Level (*i.e.,* normal, high or very high) of quality assurance;

(p) Transportation time;

(q) Design service life-The period for which the structure or structural element is to be used for its intended purpose with anticipated maintenance, but without substantial repair being necessary;

(r) Durability test requirements as applicable as per specification, if any; and

(s) Any other specific requirement like early age strength requirements etc.

**C61.3 Mix**

The clause identifies the factors influencing the design mix. The *nominal mix concrete* should be restricted to works of minor nature, in which the strength of concrete is not critical.

**C61.3.1** *Mix Proportion*

The condition of handling and placing, the amount of reinforcement and the method of compaction together determine the requirements of workability. In one instance, for underwater concreting, the clause specifies the workability of concrete as between 100 mm and 180 mm of slump. Otherwise apart from the guidance given in **Clause 61.4.7**, workability requirements are left to the judgment of the engineer-in-charge. The clause requires that the concrete shall be designed to give required workability and characteristic strength. It is difficult to determine, within a reasonable time of say one month, whether the hardened concrete will have the required durability. The intention of the clause is to draw attention to the minimum cement content and the water-cement ratios before determining the mix proportion.

The choice of *Design Mix Concrete* or *Nominal Mix Concrete* is based on strength requirements; the latter likely to involve higher cement content. Therefore, the mix proportion arrived at should be re-appraised with respect to durability and surface finish. Requirements of surface finish are not given in the standard. But, specifications available in literature for the type of surface finish required may be used.

The proportions of nominal mix suggested are on the basis of *weights* of cement and aggregates. For the determination of mix proportion for design mix concrete, the target strength should be higher than the specified *characteristic strength*, and is aimed at ensuring that the characteristic strength is attained. The concrete mix is to be proportioned to give an average strength that will be higher than the specified strength by an amount which will ensure that not more than the acceptable percentage of results (5%) will fall below the specified strength (the *Characteristic Strength*).

**61.3.2** *Design Mix Concrete*

The *Design Mix Concretes* up to *M 40* grades shall meet the following requirements:

(1) As the *guarantor of quality* of concrete used in construction, the *Contractor* shall carry out the mix design, and the mix so designed (not the *method of design*) shall be approved by the *Client* (or her/his representative) and the *Engineer-in-Charge*, within the limitations of parameters and other stipulations laid down by this standard. If sought by the *Engineer-in-Charge* supporting data (including graphs showing strength versus water-cement ratio for range of proportions, complete trial mix proportioning details) shall be provided to substantiate the choice of cement content, fine and coarse aggregate content, water, mineral admixtures, chemical admixtures, etc.

(2) The mix shall be designed to produce the densest possible concrete, following the approved *procedures* of mix designs and trials.

(3) The grade of concrete shall have the required *workability*, durability characteristics and *characteristic strength* not less than the values given in **Table 6.6**.

(4) The target *Mean Strength* $f_{ck,target}$ of concrete mix shall be estimated as:

$$f_{ck,target} = Max\left[f_{ck} + 1.65\sigma; f_{ck} + X\right]$$

where

$f_{ck}$ = Characteristic compressive strength at 28 days (specified strength),

$\sigma$ = Standard Deviation, as per **Table 6.6**; and

$X$ = A factor based on the grade of concrete, as per **Table 6.6**.

(5) Mix design done may be considered adequate for works undertaken within an year, provided there is no change in source and the quality of the materials.

(6) Standard Deviation

The standard deviation for each grade of concrete shall be calculated separately.

(a) Standard deviation based on test strength of sample

(i) *Number of test results of samples* — Total number of test strength of samples required to constitute an acceptable record for calculation of $\sigma$ shall be not less than 30. Attempts should be made to obtain the 30 samples, as early as possible, when a mix is used for the first time.

(ii) *In case of significant changes in concrete* — When significant changes are made in the production of concrete batches (for example, changes in the materials used, mix design, equipment or technical control), $\sigma$ shall be separately calculated for such batches of concrete.

(iii) *Standard deviation to be brought up to date* — Calculation of $\sigma$ shall be brought up to date after every change of mix design. The standard deviation shall be checked every month, subject to availability of a minimum 30 test results, to ensure that it is less than the value considered in mix design. If is found to be higher, necessary modification shall be done in the mix.

(b) Assumed standard deviation

Where sufficient test results for a particular grade of concrete are not available, the value of *Standard Deviation* $\sigma$ shall be assumed to be *5.0 MPa* for design of mix in the first instance. As soon as the results of samples are available, actual calculated standard deviation *(from at least 30 samples)* shall be used and the mix designed properly. But, when adequate past records for a similar grade exist and justify to the designer a value of standard deviation different from that shown in **Table 6.6**, it shall be permissible to use that value.

**Table 6.6:** Assumed Standard Deviation

| *Grade of Concrete* | *Characteristic Strength (MPa)* | *Assumed Standard Deviation (MPa)* | *X (MPa)* |
|---|---|---|---|
| M 10<br>M 15 | 10<br>15 | 3.5 | 5.0 |
| M 20 | 20 | 4.0 | 5.5 |
| M 30<br>M 40<br>M 50<br>M 60 | 30<br>40<br>50<br>60 | 5.0 | 6.5 |
| M70<br>M80 | 70<br>80 | 6.0 | 8.0 |
| M90<br>M100 | 90<br>100 | 8.0 | 10.0 |

**(a) Normal Concretes**

The *Design Mix Concretes* up to *M60* grade shall meet the following **additional** requirements:

**(b) High Strength Concretes**

The *Design Mix Concretes* higher than *M65* grade shall meet the following **additional** requirements:

**C61.3.2** *Design Mix Concrete*

Design Mix Concrete should be used for grades M25 and higher.

**61.3.3** *Nominal Mix Concrete*

Nominal mix concrete shall be *avoided*, in general. When constrained to use it, the use of *Nominal Mix Concrete* shall be restricted to works of *minor* nature, in which the *strength of concrete is not critical*. Nominal mix concrete shall be permitted only for making concretes of grades M20 or lower. *M20* concrete shall be used for RCC only, if considered acceptable from the point of view of *durability service life* (See **Clause 85** for requirements related to *Durability*).

The *mix proportions* of materials for *nominal mix concrete* shall be as specified in **Table 6.9**. The nominal mix specified therein shall be measured *by weight*; nominal mix shall not be measured *by volume*. Also:

(1) Cement content of the mix specified in **Table 6.7** for any nominal mix shall be proportionately increased, if the quantity of water in a mix has to be increased to overcome difficulties of placement and compaction, so that the *water content* as specified is not exceeded.

(2) Quantity of water required from *durability* point of view may be less than those given in **Table 6.7**. Suitable dose of water reducing admixture may be used to achieve required workability without increasing water content as given in **Table 6.7**. Alternately, Superplasticizers may be used to improve workability or reduce water content. But, in no case, *water content* shall exceed the values given in **Table 6.7**.

(3) The proportion of the *fine aggregates to coarse aggregates* shall be adjusted to from *upper limit* to *lower limit* progressively as the grading of fine aggregates becomes *finer* and the maximum size of coarse aggregate becomes *larger*. Graded coarse aggregate shall be used.

**Table 6.7**: Suggested ranges of values of workability of concrete for different applications

| *Grade of Concrete* | *Maximum Quantity of Dry Aggregates by Mass per 50 kg of Cement (taken as sum of Individual Masses of Fine and Coarse Aggregates)* | *Proportion of Fine Aggregates to Coarse Aggregates (by Mass)* | *Maximum Quantity of Water per 50 kg of Cement* |
|---|---|---|---|
| | *kg* | | *liters* |
| M 5 | 800 | Generally, 1:2, but subject to upper and lower limits of 1:1½ and 1:2½ | 60 |
| M 7.5 | 625 | | 45 |
| M 10 | 480 | | 34 |
| M 15 | 330 | | 32 |
| M 20 | 250 | | 30 |

**C61.3.3** *Nominal Mix Concrete*

The critical statement of the clause is the restriction on the use of *Nominal Mix Concrete* to works of minor nature, in which the strength of concrete is not critical. Also, specifying nominal mix is not a trivial task. The proportions will depend on ingredients available locally. The Engineer-in-Charge should prepare a few mixes and test them before prescribing the nominal mixes.

The designer and concrete manufacturer should understand clearly their respective areas of responsibility, as this is different from the *Design Mix Concrete*. In the Design Mix Concrete, the concrete manufacturer is responsible for ensuring that the mix as supplied meets the performance criteria and any mix limitations specified by the Designer. The designer's responsibility is to ensure that the specified criteria are adequate for the expected service conditions. In *Nominal Mix Concrete*, the Designer assumes the responsibility for the performance of the specified mix ingredients and proportions. In this case, the manufacturer is responsible only for ensuring that the materials actually used are as specified and in nominated proportions.

If the size of aggregate changes, adjustments should be made in the ratio of the weight of coarse aggregate and fine aggregate. In all cases, fine aggregates should conform to the grading of *Zone II* or *Zone III* of **IS 383**. Otherwise, further adjustments may become necessary. Nominal maximum size of aggregate recommended is 20 mm; nominal mix should not be used when aggregate size is larger. In view of these limitations and a number of other reasons, the clause prefers a *Design Mix Concrete* over *Nominal Mix Concrete*.

For underwater concreting, the ratio of quantity of coarse and fine aggregates should lie in the range 1.0–2.0.

### 61.4 Physical Properties

The salient physical properties of concrete shall be taken as specified hereunder.

### C61.4 Physical Properties

The properties mentioned in this clause are the ones used commonly in design and construction. For other properties, experimental data shall be relied up on from tests conducted on materials; when such data is unavailable, specialist literature may be referred to.

**61.4.1** *Modulus of Elasticity*

The short-term static *characteristic value* of *Modulus of Elasticity* $E_{ck}$ (in *MPa*) of concrete (to be used in the analysis and design of concrete structures) shall be taken as:

$$E_{ck} = \begin{cases} 10{,}000(f_{ck})^{0.3} & \text{Quartzite and Granite Aggregates} \\ 9{,}000(f_{ck})^{0.3} & \text{Limestone Aggregates} \\ 8{,}000(f_{ck})^{0.3} & \text{Basalt Aggregates} \\ 7{,}000(f_{ck})^{0.3} & \text{Sandstone Aggregates} \end{cases},$$

where $f_{ck}$ is the characteristic compressive strength (in *MPa*) of concrete.

*Partial Safety Factor* $\gamma_M$ for material or *Factor of Safety* $\kappa_M$ shall not be applied to $E_c$ given by the expression.

**C61.4.1** *Modulus of Elasticity*

The short-term static *Modulus of Elasticity* is influenced primarily by the elastic properties of the aggregate and to a lesser extent by:
(a) Type of cement,
(b) Mix proportions,
(c) Conditions of curing, and
(d) Age of concrete.
This *Modulus of Elasticity* is related normally to the compressive strength of concrete. Actual measured values may differ by 20% from values obtained from the above expression.

The modulus of elasticity is influenced primarily by the elastic properties of the aggregate (because coarse aggregate occupies about 70% of the volume of concrete) and to a lesser extent by the conditions of curing and age of the concrete, the mix proportions and the type of cement. Normally, it is expressed in terms of the compressive strength $f_{ck}$ of concrete. The short-term modulus of elasticity of a concrete is controlled by the moduli of elasticity of its ingredients. Approximate *characteristic* values for the Modulus of Elasticity $E_{ck}$ of concretes is obtained as the secant value between $f_c = f_{ck}/9$ and $f_c = f_{ck}/3$.

$E_{ck}$ of concrete provided is recommended for use in analysis and design of RCC structures, and for computing deflections of reinforced concrete flexural members. Irrespective of the limit state being considered during the analysis, the characteristic strength of concrete should be used in the expression for $E_{ck}$, and the factor $\gamma_M$ should not be introduced. Further, it should not be used for arriving at the modular ratio.

The expression for $E_{ck}$ gives the short-term Modulus of Elasticity of concrete for normal weight concretes. $E_{ck}$ is related to the density of concrete as well, but this factor has been ignored by the clause, because it deals with only with normal weights. It is a property that reflects the stiffness of the material and hence cannot be reduced with either *Partial Safety Factor* $\gamma_M$ or *Factor of Safety* $\kappa_M$ for material. The proposed values are much closer to the values proposed in the *ACI 318-14* **[Reference….]** and *EuroCode 2* **[Reference….]** for different grades of concrete (**Figure C6.2**).

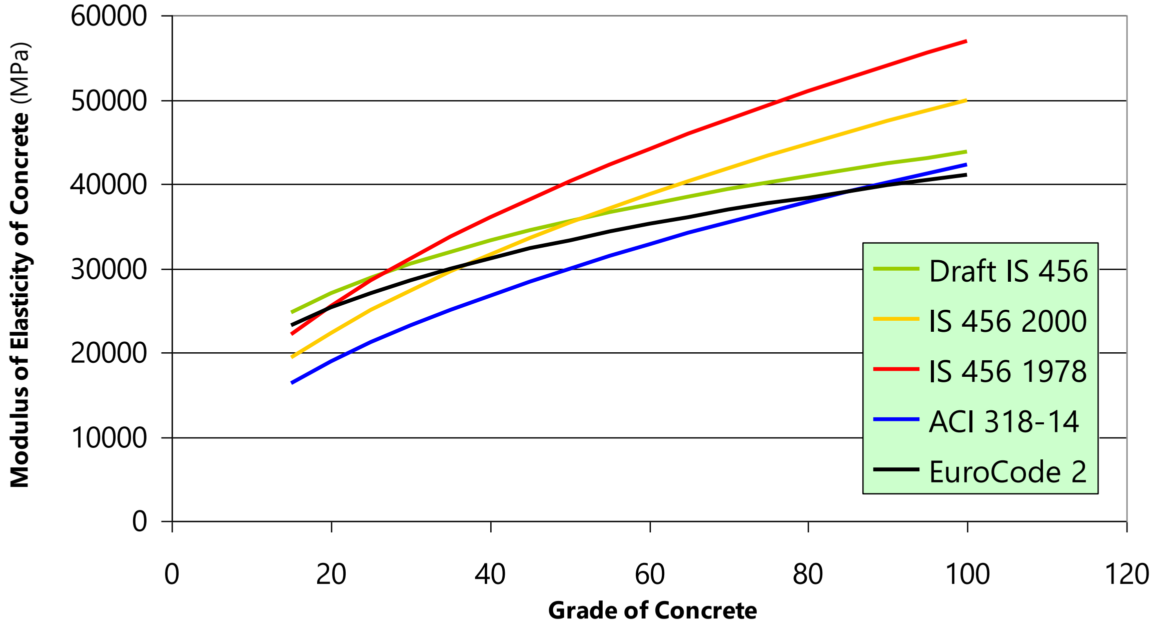



**Figure C6.2**: Modulus of Elasticity $E_c$ of concrete in Draft IS 456 2020 in comparison with those from ACI 318 – 14 and Euro Code 2

**61.4.2** *Poisson's Ratio*

The Poisson's Ratio $\nu$ of uncracked concrete with different aggregates shall be taken per **Table 6.8**.

**Table 6.8**: Poisson's Ratio $\nu$ of concrete

| *Type of Aggregate used in concrete* | *Poisson's Ratio* $\nu$ |
|---|---|
| Quartzite and Granite | 0.12 |
| Limestone | 0.15 |
| Basalt | 0.18 |
| Sandstone | 0.20 |

**C61.4.2** *Poisson's Ratio* $\nu$

The Poisson's Ratio $\nu$ is defined as:

$$\nu = -\frac{\varepsilon_{lateral}}{\varepsilon_{longitudinal}},$$

where $\varepsilon_{lateral}$ is the lateral strain and $\varepsilon_{longitudinal}$ the longitudinal strain (in the axial direction) in the member. It depends on nature of cement, the aggregate, the cement content, the relative humidity and the size of sections.

**61.4.3** *Coefficient of Thermal Expansion*

The Coefficient $\alpha$ of Thermal Expansion of concrete with different aggregates shall be taken per **Table 6.9**:

**Table 6.9**: Coefficient $\alpha$ of Thermal Expansion of concrete

| *Type of Aggregate used in concrete* | *Coefficient $\alpha$ of Thermal Expansion of Concrete* ($\times 10^{-5}$ / °C) |
|---|---|
| Quartzite | 1.20 – 1.30 |
| Sandstone | 0.90 – 1.20 |
| Basalt | 0.80 – 0.95 |
| Granite | 0.70 – 0.95 |
| Limestone | 0.60 – 0.90 |

**C61.4.3** *Coefficient of Thermal Expansion*

The Coefficient $\alpha$ of Thermal Expansion depends on nature of cement, the aggregate, the cement content, the relative humidity and the size of sections.

Data on thermal expansion of concrete is required to estimate the potential rise and distribution of temperature owing to hydration of cement in mass concrete structures. Occasionally, consideration of service load behaviour under a change in temperature may be warranted in special cases of reinforced concrete construction.

In general, the thermal characteristics of concrete are influenced greatly by characteristics of aggregate, which constitutes most of the volume of concrete. Even for concrete with same mix proportion, the thermal characteristics vary considerably depending on level of saturation and temperature. The value of coefficient of thermal expansion (per °C) is valid for 0°C to 150°C and for concretes containing aggregates from natural sources. As indicated in the Code, the value may vary between $0.6\times10^{-5}$ to $1.2\times10^{-5}$, the lowest being for calcareous aggregates and the highest for siliceous aggregates.

In the design of liquid storage structures, whenever development of cracks or over stressing of the concrete in tension cannot be avoided, **IS 3370** recommends, the concrete section should be suitably strengthened. In making the calculations either for ascertaining the expected expansion or contraction or for strengthening the concrete section, the coefficient of expansion of concrete shall be in accordance with the provisions given in **IS 456.**

$\alpha$ of concrete using common types of aggregates is shown in **Table 6.9**, and that of steel is $1.1–1.3\times10^{-5}$/°C. Hence, in situations and terrains where concrete is exposed to large variations of temperature, the difference in $\alpha$ of aggregate and of steel should not be more than $0.54\times10^{-5}$/°C. Hence, concrete in such situations should have $\alpha$ more than ($1.3\times10^{-5}$/°C – $0.54\times10^{-5}$/°C =) $0.77\times10^{-5}$/°C.

**61.4.4** *Creep Strain*

The creep co-efficient $\varphi_{t,t_0}(t)$ shall be defined as:

$$\varphi_{t,t_0}(t)=\frac{\varepsilon_{cc}(t)}{\varepsilon_{ci}(t_0)},$$

where

$\varepsilon_{cc}(t)$ = Creep strain at time $t>t_0$ (It does not include the *instantaneous elastic strain* in concrete at the time of loading),

$\varepsilon_{ci}(t_0)$ = Initial strain at loading, and

$t_0$ = Age of concrete at the time of loading.

Note: The coefficients given shall not be applicable to *light weight concretes*. The above estimates of $\varphi_{t,t_0}(t)$ shall be considered for *normal weight concretes* of grades M30–M90, if the compressive stress does not exceed $0.36f_{ck}$ at the age of loading

$$\cancel{\varphi_{t,t_0}(t)=\varphi_{bc}(t,t_0)+\varphi_{dc}(t,t_0)}$$

The Creep Coefficient $\varphi_{t,t_0}(t)$ in concrete members at time $t$ shall be determined depending on the level of creep expected in structures. For this purpose, structures shall be classified into three levels of creep, below:

(a) **Level 1:**

The following *members* and *structures* are placed under Level 1:

(1) Reinforced concrete beams, frames and slabs with spans up to 20 m, and cantilever beams up to 2.5 m span;
(2) Plain concrete footings with a reasonably uniform distribution of loads at their base;
(3) Reinforced concrete retaining walls with heights up to 30 m; and
(4) All structures:
  (i) Having Design Life up to 75 years,
  (ii) Made using concretes of grades between M20 and M60, or
  (iii) Of overall height less than 30 m.

(b) **Level 2:**

The following *members* and *structures* are placed under Level 2:

(1) Prestressed beams or slabs of spans up to 20 m, and cantilever beams up to 2.5 m span;
(2) Buildings of heights up to 100 m;
(3) Box-girder, cable-stayed or arch bridges with spans up to 80 m;
(4) Ordinary tanks, silos and pavements, and
(5) All structures having Design Life up to 75 years.

(c) **Level 3:**

The following *members* and *structures* are placed under Level 3:

(1) Prestressed, box girder, cable-stayed or arch bridges with span more than 80 m;
(2) Buildings of heights more than 100 m;
(3) Gravity, arch or buttress dams; cooling towers; large roof shells; nuclear containments and vessels; and large offshore structures; and
(4) All structures:
  (i) Having Design Life more than 75 years,

(ii) Made using concretes of grades above M60, or
(iii) Of overall height less than 30 m.

$\varphi_{t,t_0}(t)$ shall be estimated as given below for different levels of *creep*. When self-compacting concrete is used, $\varphi_{t,t_0}(t)$ shall be increased by 10% for *all levels* of creep.

(a) **Level 1:**

For structures sustaining level 1 creep, $\varphi_{t,t_0}(t)$ shall be taken as per **Table 6.10**.

**Table 6.10:** Simplified method of assessing *Creep Coefficient* $\varphi_{t,t_0}(t)$

| *Age at the time of loading* $t_0$ | *Creep Coefficient* $\varphi_{t,t_0}(t)$ |
|---|---|
| 7 days | 2.8 |
| 28 days | 2.1 |
| 365 days | 1.3 |

(b) **Level 2:**

For structures sustaining level 2 creep, $\varphi_{t,t_0}(t)$ shall be taken as per **Table 6.11**. The values specified therein are for *exposure categories C2, C3 and C4*; the values given in **Table 6.11** shall be multiplied by 1.2 for *exposure category C1*.

Values in **Table 6.13** correspond to 60% *relative humidity* and for *75 years design life*.

**Table 6.11:** Improved method of assessing *Creep Coefficient* $\varphi_{t,t_0}(t)$ considering size of Member and grade of concrete

| *Age at the time of loading* $t_0$ | *Creep Coefficient* $\varphi_{t,t_0}(t)$ *for different sizes of Members* | | |
|---|---|---|---|
| | *100 mm – 300 mm* | *300 mm* | *600 mm* |
| **Grades of Concrete M30 – M45** | | | |
| 1 day | 4.61 | 3.96 | 3.65 |
| 7 days | 3.22 | 2.76 | 2.55 |
| 28 days | 2.48 | 2.13 | 1.96 |
| 90 days | 1.98 | 1.70 | 1.57 |
| 365 days | 1.51 | 1.30 | 1.20 |
| **Grades of Concrete M50 – M65** | | | |
| 1 day | 3.28 | 2.86 | 2.66 |
| 7 days | 2.29 | 2.00 | 1.86 |
| 28 days | 1.76 | 1.54 | 1.43 |
| 90 days | 1.41 | 1.23 | 1.14 |
| 365 days | 1.07 | 0.94 | 0.87 |
| **Grades of Concrete M70 and higher** | | | |
| 1 day | 2.46 | 2.18 | 2.05 |
| 7 days | 1.72 | 1.52 | 1.43 |
| 28 days | 1.32 | 1.17 | 1.10 |
| 90 days | 1.06 | 0.94 | 0.88 |
| 365 days | 0.80 | 0.71 | 0.67 |

Where end results are not sensitive to precise values of *creep coefficients*, the values given in **Table 6.11** can be considered for design of structures with *normal weight concrete* of the said grades of concrete, subject to condition that the compressive stress does not exceed $0.36f_{ck}$ at the age of loading, and mean temperature of concrete is between 10°C and 20°C with seasonal variation between –20°C to 40°C. For temperature greater than 40°C, the creep coefficients given in **Table 6.11** may be increased by 10%, in the absence of accurate data.

(c) **Level 3:**

For structures sustaining level 2 creep, $\varphi_{t,t_0}(t)$ shall be estimated as:

$$\varphi_{t,t_0}(t) = \varphi_0 \beta_{t_0}(t),$$

where

$\varphi_0$ = *Notional Creep Coefficient* to which the creep co-efficient reaches logarithmically in 75 years, and

$\beta_{t_0}(t)$ = Coefficient describing development of creep with time

$$= \left[\frac{\log(t - t_0)}{4.437}\right]^{\min\left(\frac{RH}{75.1}\right)}$$

in which

$t$ = Age of concrete (in *days*) at the time when creep is being estimated, and

$t_0$ = Age of concrete (in *days*) at the time of loading.

The *Notional Creep Coefficient* $\varphi_0$ shall be estimated as:

$$\varphi_0 = \varphi_{RH} \beta(f_{cm}) \beta(t_0),$$

where

$\varphi_{RH}$ = Factor account for effect of *Relative Humidity* on *Notional Creep Coefficient*

$$= \begin{cases} 1 + \dfrac{1 - RH/100}{0.1\sqrt[3]{h_0}} & 45\,\text{MPa} \le f_{ck} \\ \left[1 + \left[\dfrac{1 - RH/100}{0.1\sqrt[3]{h_0}}\right]\alpha_1\right]\alpha_2 & 45\,\text{MPa} > f_{ck} \end{cases},$$

$\beta(f_{cm})$ = Factor accounting for effect of *concrete strength* on $\varphi_0$

$$= \frac{16.8}{\sqrt{f_{ck} + 8}}, \text{ and}$$

$\beta(t_0)$ = Factor accounting for effect of age of concrete at loading on $\varphi_0$

$$= \frac{1}{0.1 + t_0^{0.2}}, \text{ and}$$

in which

$RH$ = *Relative Humidity* of the ambient environment (in %),

$h_0$ = Notional size (in *mm*) of the member = $2A_c/u$,

$$\alpha_1 = \left(\frac{45}{f_{ck} + 8}\right)^{0.7}, \text{ and}$$

$$\alpha_2 = \left(\frac{45}{f_{ck} + 8}\right)^{0.2},$$

wherein

$A_c$ = *Cross-sectional Area* of the Member, and

$u$ = *Perimeter* of the Member in contact with the atmosphere.

**C61.4.4** *Creep Strain*

Creep is the continuous increase of the strain in concrete without any change in the applied stress. Creep depends on several factors, including mix of proportioning, environmental conditions (relative humidity), curing conditions, geometry of concrete member, stress in concrete, and loading history (age at loading and duration of loading) (**Figure C6.3**). As long as the stress in concrete does not exceed *one-third* of characteristic compressive strength (*i.e.,* $f_{ck}/3$), creep may be assumed to be proportional to the stress [Bazant and Wittmann, 1982].

In the estimation of deflection due to creep, the creep coefficient is considered explicitly. In other clauses, the effect of creep is allowed for through appropriate modifications to account for creep. Deflections owing to creep in beams and slabs shall be calculated as given hereunder. In *ordinary buildings*, when the slenderness ratios (*i.e.,* span-to-depth ratios) specified in _____ are complied with in beams and slabs, the effects of creep may be ignored in design of these members.

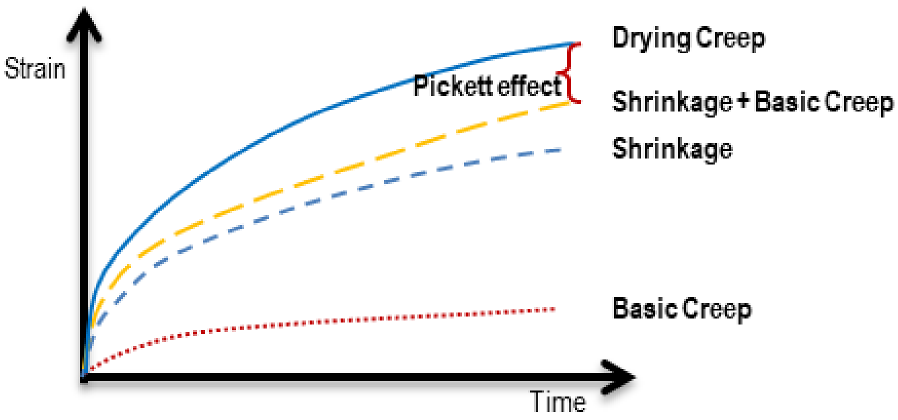
The effects of many factors that influence creep (*e.g.,* environmental condition, actual mix proportions of concrete, and proportion of permanent loads that may come on the structure) are not known with sufficient accuracy at the design stage. Therefore, grade of concrete and age of concrete at loading, which is a dominant factor and known in advance, is treated as a variable in the clause. A mean relative humidity of the location may be adequate to represent the ambient environmental condition that the structure is exposed to.

The expression for creep coefficient has been derived for 53 grade *Ordinary Portland Cement*. It is applicable for concrete strengths in the range 30–90 MPa, which is an improvement from the earlier standard.

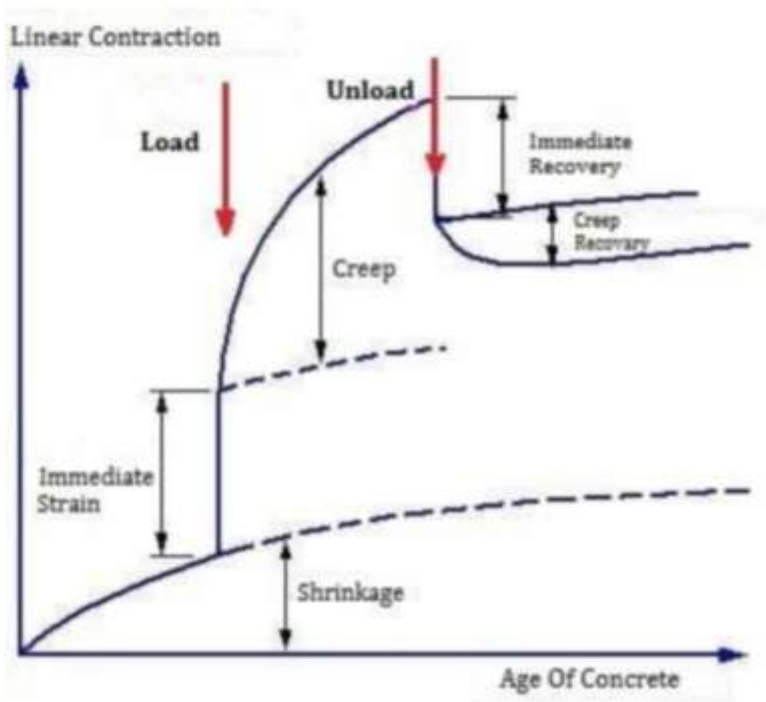
In the earlier version of this standard, the *Uniform Deformation Kinetics Approach* was adopted to represent the creep function. But, with high-strength concrete gaining popularity, the distinction between *basic creep* and *drying creep* is essential. In this version of the standard, a *product type approach* has been used to represent the two separate creep mechanisms, namely *basic creep* and *drying creep*. Basic creep depends only on the concrete mixture properties and the age of loading, but, drying creep depends on the size and shape of the specimen also, which is simplistically represented using the *notional size parameter* and the *environmental conditions* (reflecting relative humidity).

Tests have shown that a *logarithmic function* for basic creep and a *hyperbolic function* for drying creep represent the time-rate functions with reasonable accuracy [Troxell, 1958]. But, certain weakness which could become evident in certain conditions, *e.g.,* if a large member is loaded at an early age, a stress reversal can be observed under the loading conditions [Muller *et al*, 2013]. This limitation has been overcome in the present revision of the standard through the function form.

For creep sensitive structures (like tall buildings, long span bridges and nuclear containment structures), it may be appropriate to conduct creep tests on standard creep frames [ASTM C512, 2015] for a minimum period of one year, maintaining similar environmental conditions, using the concrete mixture, which is planned to be used for the project. The creep function may be appropriately deduced be including the frame interference effects [Shariff *et al*, 2020].




**(a)**




**(b)**

**Figure C6.3:** (a) Components of Creep in concrete, and (b) Time dependant behaviour of structure under creep

## 61.4.5 Shrinkage Strain

The total shrinkage of concrete depends upon the constituents of concrete, size of the member and environmental conditions. For a given humidity and temperature, the total shrinkage of concrete is most influenced by the total amount of water present in the concrete at the time of mixing and, to a lesser extent, by the cement content.

**61.4.5.1** The total shrinkage strain is composed of two components, the autogenous shrinkage strain and the drying shrinkage strain.

The value of the total shrinkage strain, $\varepsilon_{cs}$ is given by:

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$$

Where

$\varepsilon_{cs}$ = total shrinkage strain;

$\varepsilon_{cd}$ = drying shrinkage strain; and

$\varepsilon_{ca}$ = the autogenous shrinkage strain.

**61.4.5.2** The autogenous shrinkage strain develops during hardening of concrete; the major part develops in the early days after casting. Autogenous shrinkage is a linear function of concrete strength. It should be considered specifically when new concrete is cast against hardened concrete.

In the absence of accurate field/laboratory data, the following values may be considered in design:

| *Grade of Concrete* | *Autogenous Shrinkage* ( $\varepsilon_{ca} \times 10^6$ ) |
|---|---|
| M 30 | 35 |
| M 35 | 45 |
| M 45 | 65 |
| M 50 | 75 |
| M 60 | 95 |

**61.4.5.3** The drying shrinkage strain develops slowly, as it is a function of migration of the water through the hardened concrete.

The final value of the drying shrinkage strain, $\varepsilon_{cd,\infty}$ may be taken as equal to $k_h.\varepsilon_{cd}$ Values of $\varepsilon_{cd}$ may be taken from the table given below for guidance. These values are expected mean values, with a coefficient of variation of about 30 percent.

| $f_{ck}$ MPa | *Unrestrained Drying Shrinkage Values* ( $\varepsilon_{cd} \times 10^6$ ) *for Concrete with Portland cement, for Relative Humidity* | |
|---|---|---|
| | 50 percent | 80 percent |
| (1) | (2) | (3) |
| 25 | 535 | 300 |
| 50 | 420 | 240 |
| 75 | 330 | 190 |

NOTE – The values for the other designated grades may be obtained by interpolation.

$k_h$ is a coefficient depending on the notional size $h_0$, as given below:

| $h_0$ mm | $k_h$ |
|---|---|
| 100 | 1.0 |
| 200 | 0.85 |
| 300 | 0.75 |
| ≥ 500 | 0.70 |

**61.4.5.4** The development of autogenous shrinkage with time may be taken as:

$$\varepsilon_{ca}(t) = \beta_{as}(t).\varepsilon_{ca}$$

where

$$\beta_{as}(t) = 1 - \exp\left(-0.2\sqrt{t}\right) \quad \text{where, } t \text{ is in days.}$$

**61.4.5.5** The development of the drying shrinkage strain in time may be taken as:

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s).k_h.\varepsilon_{cd}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04\sqrt{h_0^3}}$$

where

$t$ = age of the concrete at the moment considered, in days;

$t_s$ = age of the concrete at the beginning of drying shrinkage, in days; normally this is at the end of curing; and

$h_0$ = notional size of the cross section, in mm

= $2A_c / u$, where $A_c$ is the concrete cross sectional area and $u$ is the perimeter of that part of the cross section which is exposed to drying.

**C61.4.5** *Shrinkage Strain*

Shrinkage of concrete is the shortening of concrete during the process of hardening, with or without the presence of external sustained loading. It depends on: (a) constituents of concrete, (b) size of member, and (c) environmental conditions. For a given humidity and temperature, the shrinkage of concrete is influenced most by the amount of water present in the concrete at the time of mixing, and by the cement content, though to a lesser extent.

The shrinkage strain $\varepsilon_{cs}$ is composed of *Autogenous Shrinkage Strain* $\varepsilon_{ca}$ and *Drying Shrinkage Strain* $\varepsilon_{cd}$. The former develops during hardening of concrete; a major part of it develops in the early days after casting owing to the self-desiccation property of concrete. $\varepsilon_{ca}$ is a linear function of concrete compressive strength $f_{ck}$. It should be considered specifically when *new concrete* is cast against *hardened concrete*.

Although, the shrinkage strain (similar to creep strain) is independent of the concrete strength, but rather a function of the mixture composition, $f_{ck}$ has been used to estimate the shrinkage strains, as it known a priori to the design engineer, and gives a reasonable approximation on the mixture properties. High-strength concrete, which has low water-binder ratio and high cementitious content, is becoming common in the construction of many structures. In such concretes, the effect of *Autogenous Shrinkage* cannot be ignored from the design calculations.

*Drying Shrinkage Strain* $\varepsilon_{cd}$ develops slowly, as water migrates out of the hardening concrete. Drying Shrinkage is higher in lean concretes, having high water-binder ratio and lesser cement concrete. The values given in **Table 6.13** and **Table 6.14** are expected mean values, with a coefficient of variation of ~30%.

In this version of the code, expressions for *Autogenous Shrinkage* strain and *Drying Shrinkage* strain are explicitly provided and the need for interpolation at different relative humidity values or notional size values has been eliminated. This helps the designer to arrive a more accurate value of the shrinkage strain. Also, if the structural response is significantly affected by the variations in shrinkage strain, then local tests are highly recommended conforming to **IS 1920-8 (Part 9)** using the same concrete type.

**61.4.6** *Strength*

The basic strengths of concrete shall be taken as specified hereunder, under *direct compression*, *direct tension* and *bending tension*. Implications of increase beyond 28 days in the strengths of concrete given hereunder shall be accounted for in design, when the same is detrimental or causes brittles actions to determine behaviour.

**(a) Compressive Strength** $f_{ck}$

The characteristic compressive strength $f_{ck}$ (in *MPa*) of concrete under *direct compression*, a basic quantity referred to in the standard, shall be taken as obtained by the procedure specified in **Clause 61.2**. The design should be based on *28-day* characteristic strength of concrete, unless there is evidence to justify a higher strength for a particular structure due to age.

When *Mean Compressive Strength* $f_{cm}$ is required to be adopted during construction, it shall be estimated as:

$$f_{cm} = Max\lfloor f_{ck} + 1.65\sigma; f_{ck} + X \rfloor,$$

where $\sigma$ is *standard deviation* and $X$ a constant specified in ______. In particular, when designing concrete mix, the *target strength* shall be taken as $f_{cm}$.

**(b) Direct Tensile Strength** $f_{ct}$

The *Direct Tensile Strength* $f_{ct}$ of concrete shall be obtained by carrying out the *Direct Tension Test* as specified in IS 5816. In the absence of test data, $f_{ct}$ can be approximated by:

$$f_{ct} = \begin{cases} 0.80 f_{ct,sp} \\ 0.54 f_{cr} \end{cases},$$

where $f_{ct,sp}$ is the *Split Tensile Strength* of concrete is obtained by carrying out the *Split Cylinder Test* as specified in **IS 5816**, and $f_{cr}$ the *Modulus of Rupture* of concrete under *bending tension* by carrying out the *Bending Tension Test* as specified in **IS 516**.

**(c) Split Tensile Strength** $f_{ct,sp}$

The *Split Tensile Strength* $f_{ct,sp}$ of concrete shall be obtained by carrying out the *Split Cylinder Test* as specified in **IS 5816**. In the absence of test data, the *design value* of $f_{ct}$ can be approximated by:

$$f_{ct,sp} = 0.36\sqrt{f_{ck}},$$

where $f_{ck}$ is the characteristic compressive strength (in *MPa*) of concrete.

When *Mean Split Tensile Strength* $f_{ct,sp,m}$ (in *MPa*) is required to be adopted during construction, it shall be estimated as:

$$f_{ct,sp,m} = Max\lfloor f_{ct,sp,m} + 1.65\sigma; f_{ct,sp,k} + 0.55 \rfloor,$$

where $\sigma$ is *standard deviation* (in *MPa*) and $X$ a constant (in *MPa*) specified in ______.

In the absence of data, the *mean value* of $f_{ct,sp,m}$, when required in construction, shall be taken as 1.4 times this characteristic value, *i.e.*, $0.50\sqrt{f_{ck}}$.

**(d) Modulus of Rupture** $f_{cr}$

The *Modulus of Rupture* $f_{cr}$ of concrete under *bending tension* shall be obtained by carrying out the *Bending Tension Test* as specified in **IS 516**. In the absence of test data, the *design value* of $f_{cr}$ can be approximated by:

$$f_{cr} = 0.5\sqrt{f_{ck}}\ ,$$

where $f_{ck}$ is the *characteristic* compressive strength (in *MPa*) of concrete.

When *Mean Flexural Tensile Strength* $f_{cr,m}$ (in *MPa*) is required to be adopted during construction, it shall be estimated as:

$$f_{cr,m} = Max\left[f_{cr} + 1.65\sigma ; f_{rk} + 0.55\right],$$

where $\sigma$ is *standard deviation* (in *MPa*) and $X$ a constant (in *MPa*) specified in ______.

In the absence of data, the *mean value* $f_{cr,m}$, when required in construction, shall be taken as 1.4 times this characteristic value, *i.e.*, $0.7\sqrt{f_{ck}}$ .

**(e) Increase in Strength with Age**

(1) The design of new structures shall be based *only on 28 days characteristic strength of concrete*, unless there is an evidence to justify a higher strength for a particular structure due to age. Such cases shall be governed by **61.4.6(e)(2)**.

(2) When increased strength is to be used, the rate of increase of compressive strength with age shall be based on actual investigations. The actual strength achievable (or achieved) at time other than 28 days strength, but not at more than 91 days in case of slow setting concretes, can be chosen to base the design and construction choices, if found more appropriate. This decision shall be based on achievement of early or delayed strength, and the age at which the first design load, apart from the self-weight, is expected to be resisted by the structure.

(3) For evaluation of *strength* or *load carrying capacity* and for retrofitting of existing structures, strength at ages other than 28 days may be used after ascertaining the actual strength, sustained load effect, state of cracking and fatigue effects, for which specialist literature may be referred.

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**C61.4.6** *Strength*

The three strength quantities specified herein are basic inputs to the design and construction processes. No *partial safety factor* $\gamma_M$ for *materials* is applied to arrive at these values.

**C(a) Compressive Strength** $f_{ck}$

The characteristic compressive strength $f_{ck}$ (in *MPa*) of concrete under *direct compression* shall be obtained from direct compression of 150mm cubes tested as per **IS 516**, and in keeping with the procedure for estimating characteristic value specified in **61**.

When tests show that strength of concrete at 28 days are lower than the specified strength, the construction may be approved based on projected strength, if that concrete is likely to attain strength required with continued hydration of cement by the time the full loads appear. This approval cannot be given, if:

(1) Increase in strength with age is already allowed for at the design stage, and

(2) Concrete is made with *High Alumina Cement*, because concretes with that cement tend to reach their potential strength much more quickly than other cements.

An expression for compressive strength $f(t)$ of concrete at any age $t$ (in days) is of the form:

$$f(t) = \left(\frac{t}{a + bt}\right) f_{ck},$$

where $a$ and $b$ are empirical constants taken as *4.7* and *0.833*, respectively.

### C(b) Direct Tensile Strength $f_{ct}$

There are three *tensile strengths* of concrete, namely: (i) the *direct (axial)* tensile strength, (ii) *flexural* tensile strength corresponding to the *Modulus of Rupture*, and (iii) the *split* tensile strength. The evaluation of direct tensile strength of concrete through is complex and requires special detailing of test specimens and test set-up. In the absence of such test data, the mean and characteristic values of *direct tensile strength* of concrete may be approximately computed as 90% of the corresponding values of *split tensile strengths*.

### C(c) Split Tensile Strength $f_{ct,sp}$

*Spilt tensile strength* of concrete $f_{ct,sp}$ is obtained by *splitting* a concrete cylinder by applying a compressive load along two diametrically opposite lines of its length. Split tensile strength is used for estimating the shear strength of beams with unreinforced webs, and axial tensile capacity of columns. The proposed mean values of *split tensile strengths* of concrete are compared with the estimates given in literature [ACI 318-14, 2014; Eurocode 2, 2004; Zain *et al*, 2004; Ahmad & Shah, 1985; Arioglu *et al*, 2006] (**Figure C6.4**).

In Appendix B, this value has to be used straightway without applying any reduction factor. The reason for not using a strength reduction factor may lie in the fact that deflection is a cumulative effect rather than a local one.

The expression given in the standard is not applicable to light weight concretes.

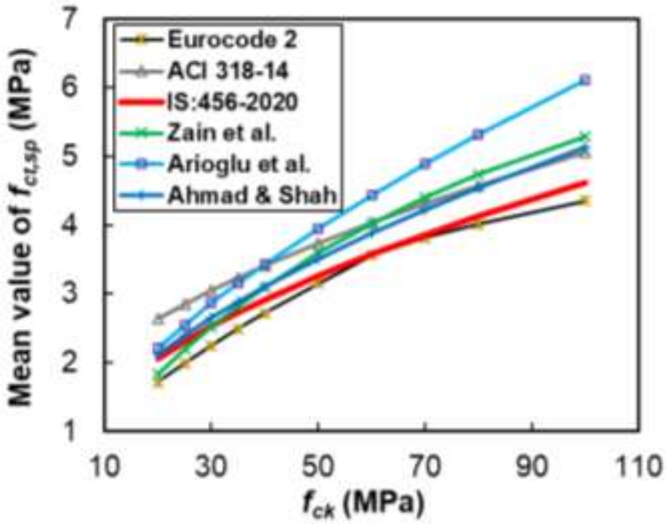


**Figure C6.4**: *Split Tensile Strength* values proposed in this standard compared with estimates proposed in literature

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**C(d) Modulus of Rupture** $f_{cr}$

The value of *Modulus of Rupture* of concrete is the bending tension capacity and is obtained from the *Bending Tension Test* on a flexural beam specimen under two-point loading. This is used in estimating the bending moment at first crack, which is used in many instances of design (including in the estimation of bending deflection of beams). The characteristic *flexural tensile strength* represents the value for which NOT more than 5% of test data are expected to fall less than this value. The *mean strength* value of $0.7\sqrt{f_{ck}}$ should be used to compute the *demand*, whereas the *characteristic value* of $0.5\sqrt{f_{ck}}$ should be used in the *design*. The proposed mean values of *Modulus of Rupture* of concrete are compared with the estimates given in literature [ACI 318-14, 2014; Eurocode 2, 2004; AS 3600, 1998] (**Figure C6.5**).

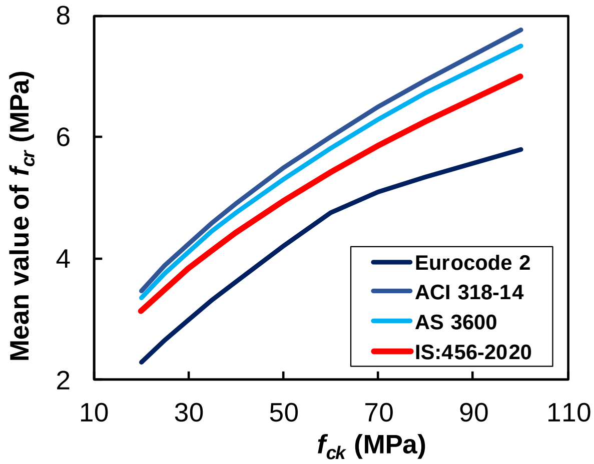


**Figure C6.5:** *Modulus of Rupture* values proposed in this standard compared with estimates proposed in literature

**C(e) Increase in Strength with Age**

~~Normally, there is an increase in compressive strength of concrete beyond 28 days; the quantum depends on many factors, including: (a) grade and type of cement; values specified correspond to concrete made with *Ordinary Portland Cement*; (b) water-cement ratio; (c) curing; (d) admixtures; and (e) environmental conditions. These age factors apply to design mix concrete only. For concretes of grades M30 and above, the rate of increase of compressive strength with age shall be based on actual investigations. Increase in strengths is observed in bending and shear in addition to direct compression. By not permitting to consider the increase in strength, the clause is taking a conservative view on this increase in strength with time.~~

**Wording need improvement**

**61.4.7** *Workability*

The concrete mix proportions chosen should be such that the concrete is of adequate workability for the placing conditions of the concrete and can properly be compacted with the means available. Suggested ranges of workability of concrete measured in ranges of workability of concrete measured in accordance with **IS 1199** are given in **Table 6.15**.

**Table 6.15**: Suggested ranges of values of workability of concrete for different applications

| *Placing Conditions* | *Degree of Workability* | *Slump* |
|---|---|---|
| (1) Blinding concrete<br>(2) Shallow sections<br>(3) Pavements using pavers | Very Low | Strict control is necessary.<br>Measurement of workability shall be determination of compacting factor; a value of compacting factor of 0.75 to 0.80 is suggested.<br>Slump shall not be relied upon (**IS 1199**). |
| Mass concrete<br>Lightly reinforced sections in slabs, beams, walls and columns<br>Floors<br>Hand placed pavements<br>Canal Lining<br>Strip footings | Low | 25 – 75 |
| Heavily reinforced sections in slabs, beams, walls and columns | Medium | 50 – 100 |
| Slip-form work<br>Pumped concrete | | 75 – 100 |
| Trench fill | High | 100 – 150 |
| Tremie concrete | Very High | Measurement of workability by determination of flow will be appropriate (as per **IS 9103**). |

**Note:**
(1) For most of the placing conditions, internal vibrators (needle vibrators) are suitable.
(2) The diameter of the needle shall be determined based on the density and spacing of reinforcement bars and thickness of sections.
(3) For Tremie concrete, vibrators are not required to be used.

**C61.4.7** *Workability*

Workability is used to cover a variety of characteristics such as cohesiveness, mobility, compatibility and finishability of concrete. Concrete design mix should be proportioned such that the required workability is achieved. The clause requires that the workability should be controlled by direct measurement of water and it should be checked at frequent intervals. There are different methods of measuring the workability of fresh concrete. Each of them measures only a particular aspect of it, and there is really no unique test which measures workability of concrete in its totality. Although new methods are being developed frequently, **IS 1199** envisages the following three methods.

**(a) Slump Test**

Out of the three methods envisaged for measuring workability, the slump test is perhaps the most widely used, primarily because of the simplicity of apparatus and the test procedure. Apart from some conclusion being drawn regarding the harshness or otherwise of the mix, slump test is essentially a measure of *consistency* or the *wetness* of the mix. The test is suitable only for concretes of medium to high workability, *i.e.,* slump 25 to 100 mm. For very stiff mixes having zero slump, the slump test does not indicate any difference in concrete of different workability. It has been pointed out that different concretes having same slump may have indeed different workability under the site conditions. But, when the uniformity between different batches of supposedly similar concretes under field conditions is to be measured, slump test has been found to be suitable.

**(b) Compacting Factor Test**

The compactability, *i.e.*, the amount of work needed to compact a given mass of concrete, is an important aspect of workability. Strictly speaking, compacting factor test measures workability in an indirect manner, *i.e.,* the amount of compaction achieved for a given amount of work. This test has been held to be more accurate than slump test, specially for concrete mixes of *medium* and *low* workability, *i.e.,* compacting factor of 0.9 to 0.8. Its use has been more popular in laboratory conditions. For concretes of very low workability, *i.e.,* compacting factor of 0.7 and below which cannot be fully compacted for comparison, this test is not suitable in the manner described in the test method. This test is sufficiently sensitive to enable differences in workability arising from the initial process of hydration of the cement. to be measured. Therefore, each time test should preferably be carried out at a constant time interval after mixing is completed.

A convenient time for releasing the concrete from the upper hopper of the compacting factor apparatus would be two minutes after completion of mixing.

(c) **Vee-Bee Test**

*Vee-Bee Test* is preferable for stiff concrete mixes having *low* or *very low* workability. Compared to the other two methods, the *Vee-Bee Test* has advantage that the concrete in the test receives a similar treatment as it would be in actual practice. In this method, *Vee-Bee Time* is the index determined as the time taken for the concrete surface to uniformly adhere to the glass disc or rider of the apparatus. This is judged visually and the difficulty of establishing the end point of the test may be a source of an error, especially if the time involved is short, say 2 to 5 seconds.

## 62. REINFORCING STEEL

Reinforcing steel used in reinforced and prestressed concrete structures shall comply with the requirements specified hereunder.

Further:

(1) All reinforcement shall be free from loose mill scales, loose rust and coats of paints, oil, mud or any other substances. Sand blasting is recommended to clean reinforcement.
(2) No reinforcement bar shall be coated with any material, *e.g., epoxy*. Any such coated bars shall not be used in concrete structures.
(3) No reinforcement bar shall be welded.

## C62. REINFORCING STEEL

For uniformity in the design estimates of strengths, values to be used in design are specified hereunder.

Also:

(1) Presence of deleterious material on the surface *destroys or reduces bond*. Hence, the surface shall be cleaned before use.

(2) Coating of reinforcement bars *weakens its bond characteristics* between the bars and adjoining concrete. The bond achieved in such cases can be radically different from that specified in this standard. For instance, investigations of concrete structures damaged either due to corrosion or earthquake shaking showed that snug contacts were not established to begin with between the epoxy coated bars and adjoining concretes, and concrete stayed delaminated from the epoxy layer.

Thus, the surface of the reinforcement should be free from any material that is likely to impair the bond or durability of steel. Loose mill scale and loose rust are removed in the normal course of handling before fixing the reinforcements. A coat of cement wash may be applied over the reinforcements, if they are to remain in the formwork for more than a few days, say a week, to prevent rusting. Also, mould oils or other formwork releasing agents shall be applied carefully in such a way that they are not smeared over the steel bars.

(3) Welding makes the material brittle in the vicinity of the weld, which is referred to as the *Heat Affected Zone*.

In exceptional cases, when welding of cold worked bars is permitted by the Design Engineer, the strength of such welded reinforcing bars shall be limited to that of Fe250 *mild steel* for purpose of design calculation. This is applicable, only when extra care and precautions are taken to prevent uncontrolled application of heat to bars.

**62.1 Grades**

The grade of the steel bars to be used at site shall be that given in the structural drawings. No change shall be made at site, unless it is approved specifically by the *structural designer* and *owner*.

The reinforcing steel bars used shall conform to **IS 1786**. The provisions of this standard shall be applicable to reinforcing steels of grades shown in **Table 6.19**.

**Table 6.19**: *Designated Grades* of steel permitted as reinforcing steel bars

| *S.No.* | *Grade* | *Specified Characteristic Yield Strength fy (MPa)* |
|---|---|---|
| 1 | Fe 250 | 250 |
| 2 | Fe 415 | 415 |
| 3 | Fe 500 | 500 |
| 4 | Fe 550 | 550 |

**C62.1 Grades**

Plain *Mild Steel* bars complying with **IS 432 (Part 1)** and HYSD bars (either cold-twisted and conforming to **IS 1786,** or hot-rolled and conforming to **IS 1139)** were widely used in the country till 2010s. The latter had lugs, ribs or deformations on the surface, that permitted larger the bond strength (at least 40% more) than plain mild steel bars of the same size. TMT bars of characteristic strength, *i.e.,* 0.2% proof stress, of 600 and 650 MPa are becoming available in the market and are included in **IS 1786**.

For certain special structures (like bins), the relevant Indian Standards indicate a preference for deformed bars. **IS 4995 (Part 2)** requires the use of deformed bars in bins and silos for avoiding large cracks and for fixing conveniently the horizontal bars that are facilitated smooth operation of sliding formwork in the construction of bins.

Hard-drawn steel wire fabrics (**IS 1566**) are occasionally used for floor slabs (hollow block, ribbed), for secondary reinforcement in developing fire resistance and in some precast concrete products (like pipes). The mesh size, weight and size of wires for square and oblong welded wire fabric should be agreed to between the purchaser and the manufacturer.

Rolled steel sections made from steel conforming to **IS 2062** are intended mainly to cover concrete-steel *composite construction*.

When two dissimilar materials are used together, the process of *corrosion* is accentuated.

**62.2 Physical Properties**

The salient physical properties of reinforcing steel bars shall be taken as specified hereunder.

**C62.2 Physical Properties**

The properties mentioned in this clause are the ones used commonly in design and construction. For other properties, experimental data shall be relied up on from tests conducted on materials; when such data is unavailable, specialist literature may be referred to.

**62.2.1** *Modulus of Elasticity*

The *Modulus of Elasticity* $E_s$ of reinforcing steel of all grades shall be taken as 20,000 *MPa*.

**C62.2.1** *Modulus of Elasticity*

The value of Modulus of Elasticity $E_s$ of reinforcing steel is independent of the types of the steels envisaged in this standard, for all practical purposes. The value of $E_s$ specified is the minimum encountered, but slightly higher values (say up to 205 GPa) are possible. This variation has negligible effects on design calculation. The specifications on acceptance criteria of steels do not specify the values for *Modulus of Elasticity*. Therefore, this clause gives only the information that should be used in design.

**62.2.2** *Poisson's Ratio*

The Poisson's Ratio $\nu$ of reinforcing steel of all grades shall be taken as 0.3.

**C62.2.2** *Poisson's Ratio*

The…

**62.2.3** *Coefficient of Thermal Expansion*

The Coefficient $\alpha$ of Thermal Expansion of reinforcing steel of all grades shall be taken as $1.2 \times 10^{-5}/^{\circ}C$.

**C62.2.3** *Coefficient of Thermal Expansion*

The *Coefficient* $\alpha$ of *Thermal Expansion* is found to be in the range $1.1–1.3 \times 10^{-5}/^{\circ}C$. The clause suggests the use of an average value of $1.2 \times 10^{-5}/^{\circ}C$.

**62.2.4** *Strength*

The *Yield Tensile Strength* of steels for which there is no clearly defined yield point, shall be taken as the 0.2% *Proof Stress*, which represents that strength results in a residual plastic strain of 0.0020 when unloaded from it. For steels having definite *yield points*, the stress corresponding to the *lower yield point* shall be considered as the *Yield (Tensile) Strength*. And, the *Ultimate (Tensile) Strength* of steels shall be taken as the maximum stress that the steel will resist before rupture.

Strength refers to *characteristic strength*. *Characteristic Yield Strength* $f_y$ of the reinforcing steel refers to the value below which not more than 5% of the test results are expected to fall. Similarly, the *Characteristic Ultimate Tensile Strength* $f_u$ also is defined similarly.

**Acceptance Criteria**

The following are the acceptance criteria:

(1) *Actual yield strength* (*i.e.,* 0.2% *Proof Strength*) of reinforcement steel bars shall not exceed 1.2 times the specified *Characteristic Yield Strength* $f_y$, and

(2) The ratio of the *actual Ultimate Tensile Strength* and *actual Yield Strength* of reinforcing steel shall be at least 1.15, but not more than 1.25.

**C62.2.4** *Strength*

The ratio of the *actual Ultimate Tensile Strength* and *actual Yield Strength* of reinforcing steel is observed to have large spread in the range 1.1 to 1.6. Such large variations

**62.2.5** *Elongation*

All reinforcing steel shall comply with the elongation requirements specified in **IS 1786**.

**C62.2.5** *Elongation*

The…

## 63. PRESTRESSING STEEL

Prestressing steel shall be used only as specified hereunder:

(1) Only as *bonded* tendons in *new concrete structures*, where it is required to resist applied loads,

(2) As *bonded* (as *internal* prestressing, if possible) or *unbonded* tendons (as *external* prestressing, but with due precautions) in *existing concrete structures*, where it is required to increase resistance of the structure, and

(3) As *unbonded* tendons, where it is required to resist effects of temporary loads during the *construction of new concrete structures* and it is removed after the said stage of construction.

## C63. PRESTRESSING STEEL

In new prestressed concrete structures, the clause requires the use of bonded tendons only; the use of *unbonded tendons* is prohibited. The following is the justification:

(1) There is no Indian Standard specifying permissible quality and strength of unbonded single strand tendons and components used in unbonded systems.

(2) No specifications are available for material and its minimum sizes and thickness to be used for various components (*e.g.,* PT coatings, PT sheathings, Anchorage coatings, connecting sleeves, end caps, and couplers) in unbonded systems.

(3) No test procedure is available in **IS 1343** to verify watertight encapsulation of unbonded prestressing steel and component. ACI has specific code for unbonded tendons/system which requires unbonded system to be encapsulated where the whole system is completely enclosed in a watertight covering from end to end, including prestressing steel, anchorages, sheathing, post-tensioning coating, sleeves, and an encapsulation cap over the strand tail at each end. Use of non-encapsulated system may result in corrosion of prestressing strand, anchorages and other components, resulting in complete loss of prestressing force. Even with encapsulated system, keeping the whole system watertight during execution is challenging.

(4) Generally, modifications made after construction (creating cut-outs, drillings, *etc.*) are not controlled. Any accidental damage to unbonded prestressing steel or anchorage due to such modifications may result in complete loss of prestressing force.

(5) International codes requires to limit contribution of unbonded prestressing steel in moment strength to 20 to 25% in plastic hinge region for SMRF. There are many other penalties specified for this system for seismic applications. Bonded system can be used for seismic application with some minimum requirements satisfied.

(6) Currently, **IS 1343** treats bonded and unbonded systems similarly, though separate requirements are available in international standards. Also, **IS 1343** does not specify many other requirements for unbonded systems, that are in force in international standards.

**63.1 Units**

The…

**C63.1 Units**

The…

**63.1.1** *Tendons*

Three types of *prestressing tendons* are permissible, namely:

(a) Plain or indented *wires*,
(b) Stress-relieved multi-ply *strands*, and
(c) High tensile steel *bars*.

These tendons shall conform to the provisions given hereunder, in addition to the standards specified in **Table 6.20**.

**Table 6.20:** *Types of Prestressing Steel*

| *Type* | *Full Name* | *BIS Standard* |
|---|---|---|
| **Wires** | *Plain* Cold Drawn Stress-relieved *Wires* | IS 1785 (Part 1) |
| | *Indented* Hard-Drawn Stress-Relieved *Wires* | IS 6003 |
| **Strands** | Stress relieved multi-ply strands of *Normal Relaxation* | IS 6006 |
| | Stress-relieved multi-ply strands of *Low Relaxation* | IS14268 |
| **Bars** | High Tensile Steel *Bars* | IS 2090 |

**(a) Wires**

The minimum elongation at fracture of the *Wires* shall be as specified in **Table 6.21**. Also:

(1) The elongation shall be measured over a gauge length of 200 mm.
(2) The 1,000 hour relaxation (tested at an initial stress of 0.7 times the *ultimate tensile strength (UTS)* at 20°C) shall not be more than 5% of 0.7 times the *UTS*.
(3) For acceptance of test results from a lot, the value calculated, as *arithmetic mean* minus *0.6 of the range of test results*, shall be more than the *Minimum Tensile Strength* and *Minimum Elongation at Fracture* specified specified in **Table 6.21**.
(4) *0.2% Proof Stress* shall not be less than *85% of Minimum Tensile Strength* specified in **Table 6.21**.
(5) The wires conforming to Indian Standards can be provided with protective coatings, like galvanizing or epoxy coating, carried out in specialized manufacturing units. But, if the manufacturing processes affect any of the *mechanical* and *physical properties*, such properties shall be considered in design calculations.

**Table 6.21:** Minimum elongation needed in *Hard Drawn Stress Relieved Wires*

| *Type* | *Diameter* | *Minimum Tensile Strength* | *Minimum Elongation at Fracture* |
|---|---|---|---|
| | *mm* | *MPa* | % |
| Plain Wires | 4 | 1715 | 3.0 |
| | 5 | 1570 | 4.0 |
| | 7 | 1470 | 4.0 |
| | 8 | 1375 | 4.0 |
| Indented Wires | 4 | 1715 | 3.0 |
| | 5 | 1570 | 4.0 |

**(b) Strands**

The minimum elongation at fracture of the *Strands* shall be as specified in **Table 6.22**. Also:

(1) The elongation shall be measured over a gauge length of *600 mm*. The elongation measured immediately before fracture of any of the constituent wires over the said gauge length, shall not be less than 3.5% of the gauge length.

(2) The 1,000 hour relaxation (tested at an initial stress of 0.7 times the *ultimate tensile strength (UTS)* at 20°C) shall not be more than 5% of 0.7 times the *UTS* in *normal relaxation* steel and 2.5% of 0.7 times the *UTS* in *low relaxation* steel.

(3) For acceptance, all samples tested from a batch shall meet requirement of minimum *Breaking Load* and *Proof Load* as specified in **Table 6.22**.

(4) The strands conforming to Indian Standards can be provided with protective coatings, like galvanizing or epoxy coating, carried out in specialized manufacturing units. But, if the manufacturing processes affect any of the *mechanical* and *physical properties*, such properties shall be considered in design calculations.

**Table 6.22**: *Minimum elongation*, *breaking load* and *proof load* needed in *Stress Relieved Strands*

| *Class* | *Designation* | *Nominal Area* | *Normal Relaxation* | | *Low Relaxation* | |
|---|---|---|---|---|---|---|
| | | | *Breaking Load* | *0.2% Proof Load* | *Breaking Load* | *0.2% Proof Load* |
| | | *mm²* | *kN* | *kN* | *kN* | *kN* |
| I | 11.1 mm 7 ply | 70.0 | 124.54 | 105.86 | 120.1 | 108.00 |
| | 12.7 mm 7 ply | 92.9 | 166.18 | 139.6 | 160.1 | 144.1 |
| | 15.2 mm 7 ply | 139.0 | 226.86 | 192.83 | 240.2 | 216.2 |
| II | 11.1 mm 7 ply | 74.2 | 137.89 | 117.21 | 137.9 | 124.1 |
| | 12.7 mm 7 ply | 98.8 | 183.71 | 156.11 | 183.7 | 165.3 |
| | 15.2 mm 7 ply | 140.0 | 261.44 | 222.23 | 260.7 | 234.6 |

**(c) Bars**

The minimum elongation at fracture of the *High Tensile Steel Bars* shall be as specified in **Table 6.23**. Also:

(1) The elongation shall be measured over a gauge length of $5.65\sqrt{A}$, where $A$ is the area of cross-section of steel bar. The elongation measured immediately before fracture shall not be less than 10% of the gauge length.

(2) The 1,000 hour relaxation (tested at an initial stress of 0.7 times the *Ultimate Tensile Strength (UTS)* at 20°C) shall not be more than *49 MPa*.

(3) For acceptance of test results from a lot, a value calculated as arithmetic mean minus 0.6 of the range of test results shall be more than the *minimum strength* and elongation as specified in **Table 6.23**.

**Table 6.23:** *Minimum Elongation*, *Specified Tensile Strength* and *0.2% Proof Strength* needed in *High Tensile Steel Bars*

| *Diameter* | *Minimum Elongation* | *Minimum Specified Tensile Strength* | *Minimum 0.2% Proof Strength* |
|---|---|---|---|
| *mm* | % | *MPa* | % |
| 10, 12, 16, 20, 22, 25, 28, 32 | ??? | 980 MPa | 85% of specified tensile strength |

**C63.1.1** *Tendons*

The…

**C(a) Wires**

The…

**C(b) Strands**

The…

**C(c) Bars**

The…

**63.1.2** *Sheathing Ducts and Joints*

The *sheathing ducts* shall:

(1) Be made of either of *Mild Steel* (conforming to ____) or *High Density Poly-Eurythane* (conforming to ____), having a test certificate furnished by the manufacturer;
(2) Be in as long lengths as practical from handling and transportation considerations without getting damaged;
(3) Be watertight *at its internal joints* when bent to the minimum radius of bending required;
(4) Conform to the requirements specified in ____ and ____, and. The tests specified in ____ shall be performed as part of additional acceptance tests for prestressing systems employing corrugated HDPE sheathing ducts and are not meant for routine site testing purposes and
(3) Have *smooth internal surfaces*.

The *joints in sheathing ducts* shall:

(1) Be joined by any of the following methods, subject to satisfactory pressure tests, before prestressing:
   (a) *Corrugated Threaded Sleeve Couplers*, which can be tightly screwed to the outside of the sheathing ducts;
   (b) *Welding* using electric roaster machine or mirror machine; and
   (c) *Heat Shrink Couplers*; and
(2) Be *leak-tight* against water pressure of 0.05 MPa for 5 minutes as per test procedure given in *Appendix*.

**(a) Metallic Ducts**

Unless specified otherwise, the material of *metal sheathing* shall be:

(1) *Cold rolled cold annealed* (CRCA) mild steel intended for mechanical treatment and surface refining, but not for quench hardening or tempering;
(2) *Clean* and *free from rust*, and normally of bright metal finish;
(3) *Galvanized* or used with lead coated mild steel strips when used in aggressive environment; and
(4) Of *thickness* not be less than 0.3mm, 0.4mm and 0.5mm for sheathing ducts having internal diameter up to 50mm, 75mm and 90 mm, respectively; for ducts of larger diameters, the thickness shall be as recommended by prestressing system supplier.

**(b) Corrugated High Density Poly-Eurythane Ducts**

Unless specified otherwise, the material of *HDPE sheathing* shall:

(1) Be made of High density polyethylene with more than 2% carbon black to provide resistance to ultraviolet degradation;
(2) Of manufactured wall thickness of at least 2.0 mm, 2.5 mm, 3 mm, and 4 mm for *circular ducts* of internal diameter 50 mm, 85 mm, 100 mm and 125 mm, respectively, and of at least 2.0 mm for *flat ducts* of any size. Linear interpolation may be done for any intermediate values.
(3) Have tolerance for duct diameter shall be larger of ±1% and ± 1 mm, and for wall thickness -0/+0.5 mm;
(4) Corrugated on both sides for internally bonded tendons; and
(5) Transmit 40% of ultimate tendon strength from the tendon to the surrounding concrete over a length not greater than 16 times *duct diameter*.

**(c) Tests on Mild Steel and Corrugated HDPE Sheathing Ducts**

The following tests shall be performed on Ducts to ascertain their suitability:

**(i) Mild Steel Sheathing Ducts**

All tests specified below shall be carried out on the same sample and in the order given below. At least 3 samples shall be tested for one lot of supply (not exceeding 7,000m in length).

*Workability Test –*

A test sample 1,100 mm long shall be soldered to a fixed base plate with a soft solder (**Figure 6.5**). Then, it shall be bent to a radius of 1,800 mm alternately on either side to complete 3 cycles. Thereafter, the sealing joints shall be visually inspected to verify that no failure or opening has taken place.

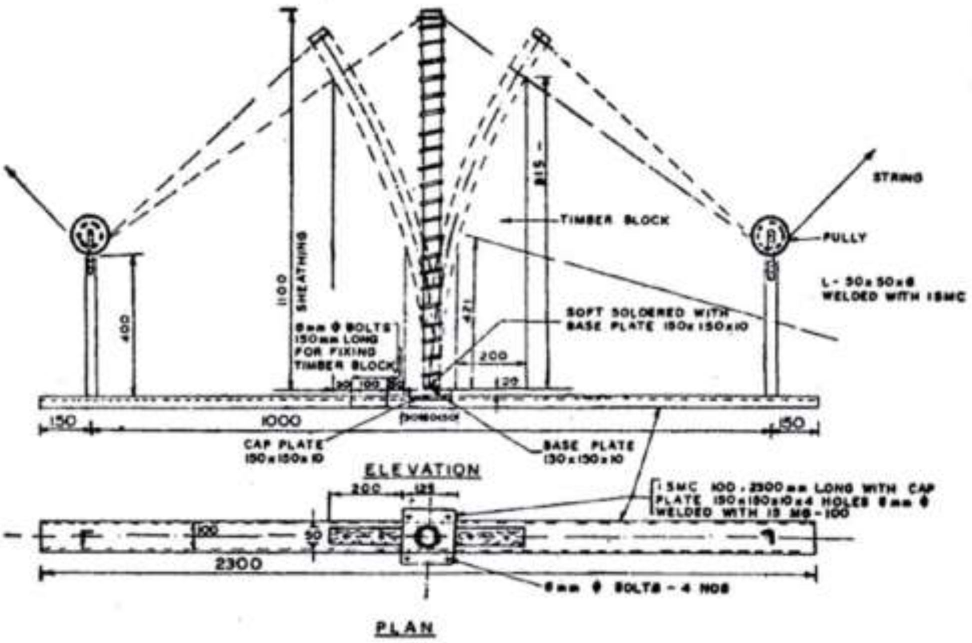


**Figure 6.5**: Test set-up of *Workability Test* of a MS Sheathing Duct

*Transverse Load Rating Test –*

A test sample (500 mm long) shall be placed on a horizontal support, so that the sample is supported at all points of outward corrugations (**Figure 6.6**), and loaded gradually in increments (as per **Table 6.24**) at the centre of the supported portion through a circular contact surface of 12 mm diameter. Couplers shall be placed so that the load is applied approximately at the centre of two corrugations (**Figure 6.6**).

The sample is said to be acceptable, if the permanent deformation is less than 5% of the original diameter.

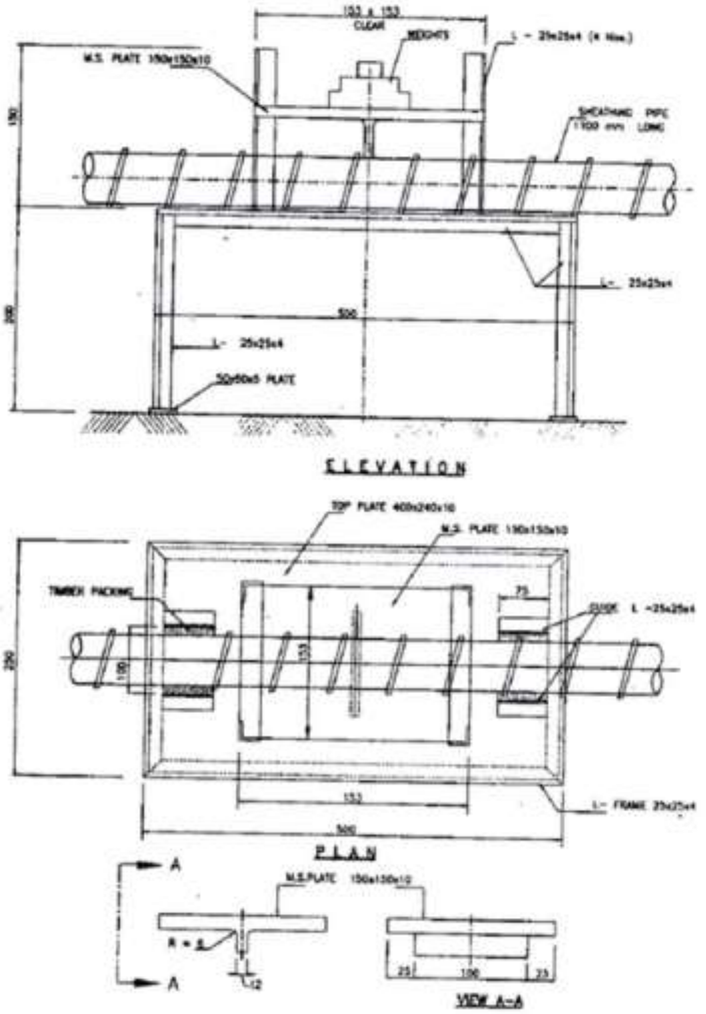


**Figure 6.6:** Transverse Load Test Set-up

**Table 6.24:** Load to be applied *transversely* to Test the Capacity of *Ducts*

| *Diameter* (mm) | *Transverse Load* (kN) |
|---|---|
| 25≤ φ <35 | 250 |
| 35≤ φ <45 | 400 |
| 45≤ φ <55 | 500 |
| 55≤ φ <65 | 600 |
| 65≤ φ <75 | 700 |
| 75≤ φ <85 | 800 |
| 85≤ φ <90 | 1,000 |

*Tension Load Test –*

A test sample (1,100 mm long) shall be hung from a vertical support, so that the sample is supported at all points of outward corrugations (**Figure 6.7**). The hollow core is filled with a wooden circular piece having a diameter of 95% of the inner diameter of the sample to maintain circular profile during test loading (**Figure 6.7**). And, the duct is loaded gradually in increments (as per **Table 6.25**) at the bottom end. Couplers shall be screwed on the duct (**Figure 6.7**).

The sample is said to be acceptable, if there is no deformation of the joints or slippage of couplers.

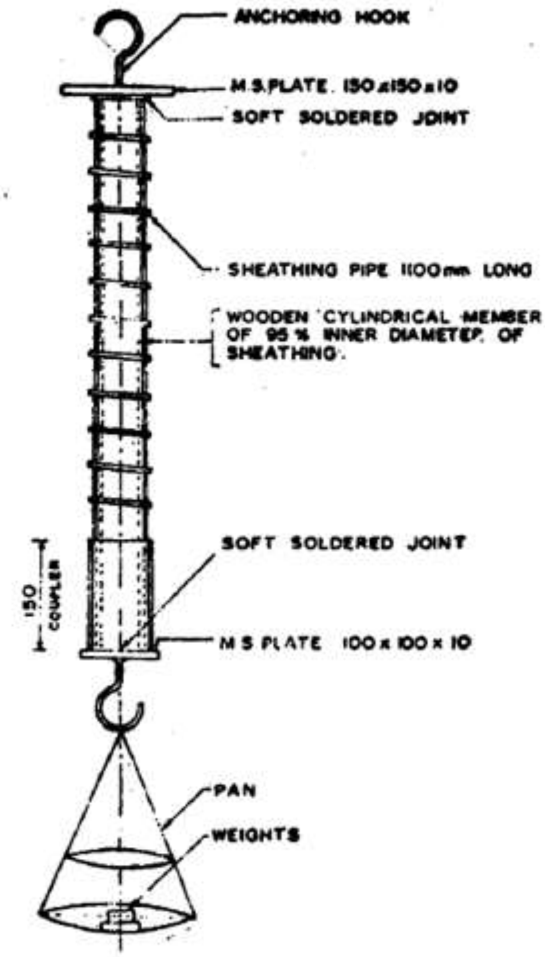


**Figure 6.7:** Axial Load Test Set-up

**Table 6.25:** Load to be applied *axially* to Test the Capacity of *Ducts*

| *Diameter* (mm) | *Axial Load* (kN) |
|---|---|
| 25≤ φ <35 | 300 |
| 35≤ φ <45 | 500 |
| 45≤ φ <55 | 800 |
| 55≤ φ <65 | 1,100 |
| 65≤ φ <75 | 1,400 |
| 75≤ φ <85 | 1,600 |
| 85≤ φ <90 | 1,800 |

*Water Loss Test* –

A sample (1,100 mm long) shall be sealed at one end, the other end is connected to a system capable of applying a pressure of 0.05 MPa, and the sample is filled with water (**Figure 6.8**). The water pressure is maintained at that pressure (using a *hand pump and pressure gauge*, or a *stand pipe system*) (**Figure 6.8**).

The sample is said to be acceptable, if the water loss does not exceed 1.5% of the volume of the sample.

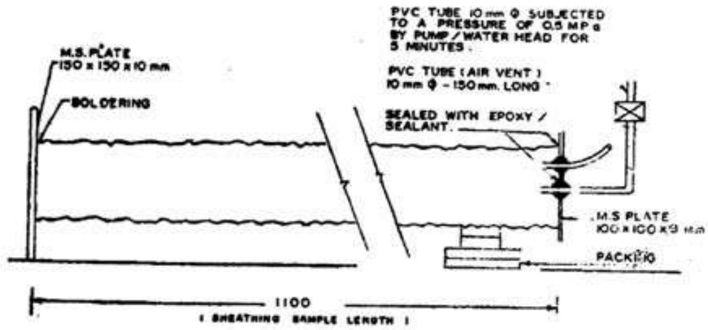


**Figure 6.8:** Water Loss Test Set-up

The *Actual Volume* $V_p$ of water filled in the sample is estimated as:

$$V_p = V_a - V_b,$$

where

$V_a$ = Pre-measured quantity of water in measuring cylinder before filling the sample, and

$V_b$ = Balance quantity of water left in the cylinder after completely filling of the test sample.

The *Relative Profile Volume* $\rho_v$ shall be estimated as:

$$\rho_v = V_p - \frac{\left(\frac{\pi\phi^2 L}{4}\right)}{\pi\phi}$$

where

$L$ = Length of duct specimen, and

$\phi$ = Internal nominal diameter.

*Specification for Sheathing Duct Joints –*

The *sheathing ducts* shall be of the spiral corrugated type. For major projects, the sheathing ducts shall be manufactured preferably at the project site utilizing appropriate machines. With such an arrangement, long lengths of sheathing ducts may be used with consequent reduction in the number of joints and couplers. Where sheathing duct joints are unavoidable, such joints shall be made cement slurry tight by the use of corrugated threaded sleeve couplers which can be tightly screwed on to the outer side of the sheathing ducts. A *heat-shrink coupler* also could be used, if suitable.

The *Sleeve Coupler* (**Figure 6.9**) shall not have a length of the coupler less than *150mm*, but should be increased up to 200 mm wherever practicable. The joints between the ends of the coupler and the duct shall be sealed with adhesive sealing tape to prevent penetration of cement slurry during concreting. The couplers of adjacent ducts shall be staggered, wherever practicable. As far as possible, couplers shall not be located in curved zones. The corrugated sleeve couplers are being manufactured conveniently using the sheath making machine with the next higher size of die set.

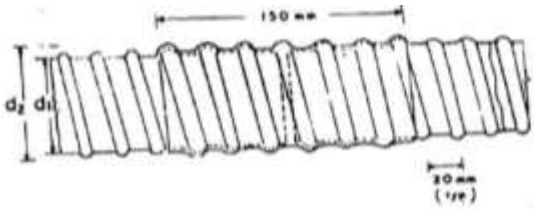


**Figure 6.9:** Typical details of *sleeve coupler*

The *Heat-Shrink Coupler* (**Figure 6.10**) is supplied in the form of bandage rolls, which can be used for all diameters of sheathing ducts. The bandage is coated on the underside with a heat sensitive adhesive so that after heating the bandage material shrinks in to the sheathing and ensures formation of a leak proof joint, without the need for extra taping or support in the form of corrugated sleeve couplers. A *soft gas* flame is used to heat the coupler.

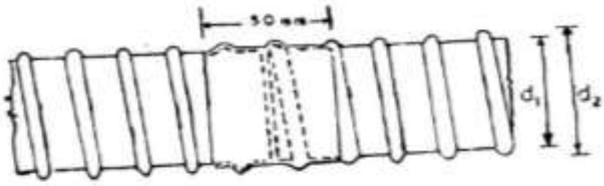


**Figure 6.10:** Typical details of *heat shrink coupler*

**(ii) Corrugated HDPE Sheathing Ducts**

A summary of all the requirements, methods of verification, and acceptance criteria for the polymer duct system together with the components and material is given in **Table 6.25**.

**C63.1.2** *Sheathing Ducts and Joints*

*Transverse Load Rating Test –*

The test ensures that stiffness of the sheathing is sufficient to prevent permanent distortion during site handling.

**63.1.3** *Mechanical Couplers*

Fixed or movable type *Mechanical Couplers* shall:

(1) Be placed to not affect adversely the load carrying capacity of the member;
(2) Meet the specifications laid down by the manufacturer for minimum concrete cover over couplers and reinforcement for bursting and spalling, but subjected to acceptance tests similar to those specified in **Clause _____** for *anchorages*;
(3) Be located away from intermediate supports;
(4) NOT be used in more than on 50% of the tendons at any cross-section;
(5) Be so spaced such that the distance between any two successive sections at which cables are coupled is not closer than 1.5m in structural members whose depth is less than 2.0m, but not closer than 3.0m when depth is more than 2.0m;
(6) Meet the requirements of strength of individual anchorages as specified in **6.4.2.6**; and
(7) Be able to transfer full force of tendon from one to another.

Also, the anchorage and stressing of *second tendon* shall not disturb the anchorage of the first tendon, when *fixed couplers* are used.

**C63.1.3** *Mechanical Couplers*

Mechanical couplers of *fixed or movable type* are devices in which individual lengths of tendons are anchored in two collinear directions to form one continuous tendon.

For established systems, the client/owner may ask for fresh tests to verify the suitability of the system.

**63.1.4** *Anchorages*

**(a) Pre-tensioned Systems**

*Anchorages* shall:

(1) Enable the *full design strength* to be developed in the tendons when used with appropriate lengths in pre-tensioned tendons, and have sufficient (**Figure 6.11**):

(a) *Transmission Length* $L_{pt}$ over which the prestressing force $P_0$ is transmitted fully to concrete,

(b) *Dispersion Length* $L_{disp}$ over which the prestressing force $P_0$ is dispersed gradually to concrete, as per:

$$L_{disp} = \sqrt{L_{pt}^2 + d^2}$$ , and

(c) *Anchorage Length* $L_{bpd}$ over which the design tendon force $P_{pd}$ is developed in concrete.

(2) Meet the requirements specified in **Clause _____**,

(3) Not be permitted in top surface of the deck, and

(4) Be introduced only after examining their consequences, if needed *temporarily* only during construction.

(5) When tendons are anchored:

(3) at a construction joint or within a concrete member (whether on an external rib, within a pocket or entirely inside the member), the minimum residual compressive stress shall be checked to be at least 3 MPa in the direction of the anchored prestressing force (under LC1 in Chapter 5). If the minimum residual stress is lesser, reinforcement shall be provided to cater to the local tension developed near the anchor beyond the tendon that is terminated. This check is not required, if the tendon is coupled at the anchorage considered.

(4) in the deck slab and soffit slab, the slab shall be thickened locally or provided with blisters so that the minimum concrete cover to anchorage is 200 mm.

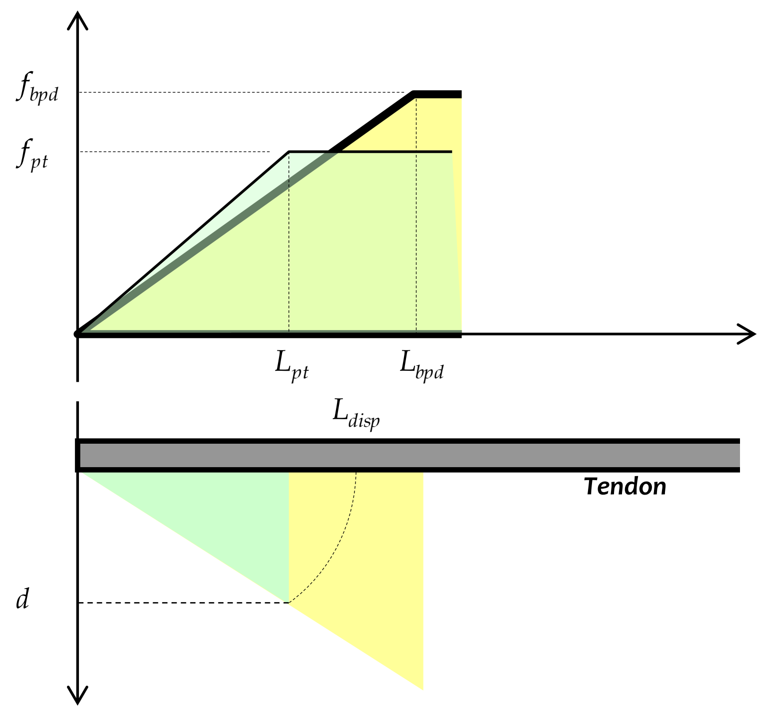


**Figure 6.11**: Length Parameters in Transfer of Prestress in *Pre-tensioned* Elements

(2) *Transfer of Prestress*:

(i) The prestress shall be assumed to be transferred to concrete by a *constant bond stress* $f_{bpt}$, given by:

$f_{bpt} = \eta_t \eta_b f_{ctd}$, and

where

$\eta_t$ = Coefficient accounting for type of tendon and bond with concrete

$$= \begin{cases} 2.7 & \text{Indented Wires} \\ 3.0 & \text{3 and 7 wire strands} \\ \textit{From Actual Tests} & \text{Others} \end{cases}$$

$\eta_b$ = Coefficient accounting for quality of bond with concrete

$$= \begin{cases} 1.0 & \text{Good bond conditions} \\ 0.7 & \text{Others} \end{cases}$$

$f_{ctd}(t)$ = Design tensile stress at time of release time $t$

$$= 0.7\left(\frac{0.5\sqrt{f_{ck}}}{\gamma_M}\right),$$

(ii) The tendons shall be provided with a *design transmission length* $L_{pd}$, given by:

$$L_{pd} = \begin{cases} 0.8L_{bpt} & \text{Local stress at release} \\ 1.2L_{bpt} & \text{Higher for Limit State of Safety (shear, anchorage, } etc.) \end{cases}$$

where

$L_{bpt}$ = Basic transmission length = $\alpha_1 \alpha_2 \varphi\left(\frac{f_{pm0}}{f_{pmt}}\right)$,

$$\alpha_1 = \begin{cases} 1.00 & \textit{Gradual Release} \\ 1.25 & \textit{Sudden Release} \end{cases},$$

$$\alpha_2 = \begin{cases} 0.25 & \textit{Tendons with circular crcross-section} \\ 0.19 & \textit{3-and 7-wire strands} \end{cases},$$

$\phi$ = Nominal diameter of tendon, and

$f_{pm0}$ = Tendon stress just after release.

(iii) The concrete stress shall be assumed to be linearly distributed outside the dispersion length $L_{disp}$, given by:

$L_{disp} = \sqrt{L_{pd}^2 + d^2}$, and

(iv) The concrete stress may be taken different from the above, if adequately justified and if the transmission length is modified accordingly.

(3) *Anchorage of Tensile Force for the Ultimate Limit State*:

(i) The anchorage of tendons shall be checked at sections where the tensile stress in concrete exceeds $0.5\sqrt{f_{ck}}$ *MPa*. The tendon force shall be calculated at a cracked section, including the effect of shear. When the tensile stress in concrete is less than $0.5\sqrt{f_{ck}}$ *MPa*, no anchorage check need be performed.

(ii) The bond strength for anchorage in the ultimate limit state is:

$f_{bpd} = \eta_t \eta_b f_{ctd}$, and

where

$\eta_t$ = Coefficient accounting for type of tendon and bond with concrete

$$= \begin{cases} 1.4 & IndentedWires \\ 1.2 & 7wirestrands \\ FromActualTests & Others \end{cases}$$

$\eta_b$ = Coefficient accounting for quality of bond with concrete

$$= \begin{cases} 1.0 & Goodbondconditions \\ 0.7 & others \end{cases}$$

$f_{ctd}(t)$ = Design tensile stress at time of release time $t$

$$= 0.7\left(\frac{0.5\sqrt{f_{ck}}}{\gamma_M}\right),$$

(iii) The design tensile strength of concrete shall be limited to *3MPa* in concretes of higher strengths.

(iv) The total anchorage length $L_{bpd}$ for anchoring a tendon carrying a stress $f_{pd}$ is (**Figure 6.12**):

$$L_{bpd} = L_{pt2} + \alpha_2\phi\left(\frac{f_{pd} - f_{pd\infty}}{f_{bpd}}\right),$$

where

$L_{pt2}$ = Upper value of design transmission length $L_{pd}$ (= $1.2L_{bpt}$),

$f_{pd}$ = Stress in tendon given by (i) above, and

$f_{pd\infty}$ = Stress in tendon after all losses.

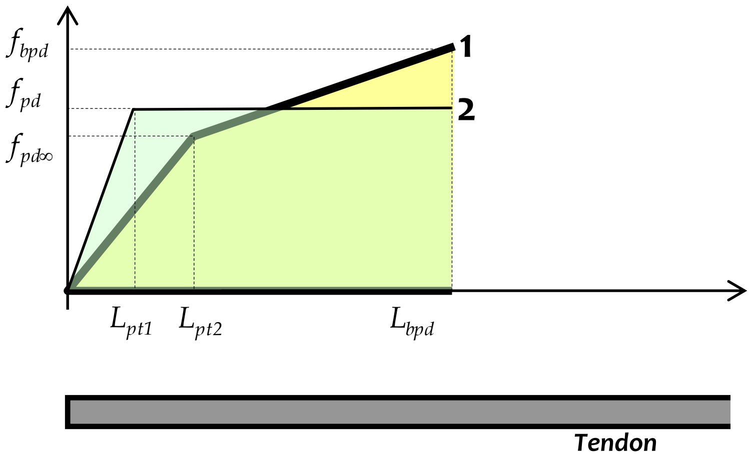


**Figure 6.12**: *Stresses in Anchorage Zone* of Pre-Tensioned Members: (1) at release of Tendons, and (2) at Ultimate Limit State

(v) When reinforcing and *pre-tensioned* steels are used together within the same zone of concrete, the anchorage capacities of each shall be calculated separately and added for design verification.

(4) *Deviators*:

(i) A deviator shall satisfy the following requirements:

(a) Withstand both longitudinal and transverse forces that the tendon applies to it and transmit these forces to the structure; and

(b) Ensure that the radius of curvature of the prestressing tendon does not cause any overstressing or damage to it.

(ii) In a deviation zone,

(a) The tubes forming the sheaths shall be able to sustain the radial pressure and longitudinal movement of the prestressing tendon, without damage and without impairing its proper functioning; and

(b) The radius of curvature of the tendon shall not be less than 40 times the diameter of wire/strand.

(c) Designed angle of deviation of tendon shall be limited to 0.01 radians when no deviator is used. But, the forces developed by the change of angle shall be considered in design.

**(b) Post-tensioned Systems**

The anchorages shall of one of the following types:

(1) Anchorages (partially or fully embedded in concrete) in which the prestressing force of tendons is transferred within the body of the prestressed element by *combination of bearing, friction and wedge action*; and

(2) Anchorages (externally mounted) in which the prestressing force of tendons is transferred to concrete through an *externally mounted bearing plate*.

The anchorage shall meet the following requirements:

(1) The anchorage may consist of any device patented or otherwise, which complies with the requirements laid down in **6.2.2.1.7.1** to **6.2.2.1.7.7**. Proprietary anchorages shall be handled and used strictly in accordance with the manufacturer's instructions and recommendations.

(2) The anchorage shall be capable of holding (with nominal slip) the prestressing tendon carrying a load corresponding to average of the *design* initial prestressing load and the *ultimate* strength of the prestressing tendon.

(3) The anchorage shall transfer effectively and distribute, as evenly as possible, the entire force from the prestressing tendon to the concrete without inducing undesirable secondary or local stresses.

(4) The anchorage shall be safe and secure against *static load*, *dynamic load* and *impact load*.

(5) The anchorage shall have provision for introducing a suitable protective medium, such as *cement grout*, for the protection of the prestressing steel, unless alternate arrangements are made.

(6) The anchorage shall be capable of:

(a) Holding and transferring a force not less than 95% of the *actual mean tensile ultimate strength* of the tendon, without failure of any part of the anchorage-tendon assembly.

(b) Withstanding not less than 20 lakh cycles of fatigue load less than 60% of nominal UTS of tendons it is expected to hold, without suffering more than 5% breakage of wires or strands at the load frequency of not more than 500 cycles per minute.

**C63.1.4** *Anchorages for Post-Tensioning Systems*

**(a) Pre-tensioned Systems**

(3)(iii) Concrete becomes more brittle with increase in its strength, and hence the limit is specified on the tensile strength.

**(b) Post-tensioned Systems**

The anchorage system comprises the *anchorage*, and an *arrangement of tendons & reinforcement* designed to act with the anchorage. The types of anchorages mentioned are used normally in bridges.

**63.1.5** *Grout*

Unbonded tendons placed either in ducts *embedded in concrete* or located outside (external to) the member shall be protected from corrosion by suitable fillers. Grouting by cement, wax, and nuclear grade (low sulphur) grease are some of the options. For materials other than cement or such long-life permanent materials, arrangements shall be made for inspection and refilling or replacement of grouting materials.

Post-tensioned tendons shall be bonded to the concrete of prestressed member using a *grout*, whose properties comply with requirements specified in **Table 6.26**. Also, the grout placed in the duct shall meet the requirements specified in **Table 6.27**.

**Table 6.26**: Typical properties of *Post-Tensioned Grouts* in Steel Ducts
**(Some modification pending in this table)**

| *S.No.* | *Parameter* | *Value* | *Test Method* |
|---|---|---|---|
| 1 | *Efflux Time* (s) | | Marsh Cone Test |
| | (a) Immediately after mixing | 15 - 20 | |
| | (b) 30 minutes after mixing | 18 - 23 | |
| | (c) 180 minutes after mixing | 25 - 30 | |
| 2 | *Spread Diameter* (mm) | | Spread Test |
| | (a) immediately after mixing | 155 - 160 | |
| | (b) 30 minutes after mixing | 145 - 150 | |
| | (c) 180 minutes after mixing (mm) | 130 - 135 | |
| 3 | *Bleed Water Volume* (ml) | | Inclined Tube Bleed Test |
| | (a) Standard | 0 | |
| | (b) Wick-induced | 0 | |
| | (c) Pressure-induced | 0 | |
| | (d) Inclined tube | 0 | |
| 4 | *Volume of Soft Grout* (ml) | 0 | Visual Observation and Extraction by Scraping |
| 5 | *Setting Time* (hours) | | ?? |
| | (a) Initial | 7 – 10 | |
| | (b) Final | 15 – 17 | |
| 6 | *Change in length* (%) | | ?? |
| | (a) In sealed condition on 28 days | 0 – 0.016 | |
| | (b) 28 days | 0 – 0.030 | |
| 7 | *Cube Compressive Strength* (MPa) | | ?? |
| | (a) 3 days | > 20 | |
| | (b) 7 days | > 35 | |
| | (c) 28 days | > 40 | |

**Table 6.27**: Specifications for Grouts to be used in *Post-tensioned Systems*

| *Parameter* | *Acceptance criteria* | *Lab Tests* | *Field (Site) Tests* |
|---|---|---|---|
| *Efflux Time* $T_{e,0}$ (s) immediately after mixing | ≤ 25 | | ✓ |
| *Efflux time* $T_{e,30}$ (s) 30 minutes after mixing | $1.2T_{e,0} \geq T_{e,30} \geq 0.8T_{e,0}$<br>$T_{e\,30} \leq 25$ | ✓ | |
| *Efflux time* $T_{e,180}$ (s) 180 minutes after mixing | $1.4T_{e,0} \geq T_{e,180} \geq 0.8T_{e,0}$<br>$T_{e,180} \leq 25$ | ✓ | |
| *Spread diameter* $D_{s,0}$ (mm) immediately after mixing | ≥ 140 | | ✓ |
| *Spread diameter* $D_{s,30}$ (mm) 30 minutes after mixing | $1.2D_{s,0} \geq D_{s,30} \geq 0.8D_{s,0}$<br>$D_{s,30} \geq 140$ | ✓ | |
| *Spread diameter* $D_{s,180}$ (mm) 180 minutes after mixing | $1.4D_{s,0} \geq D_{s,180} \geq 0.6D_{s,0}$<br>$D_{s,180} \geq 140$ | ✓ | |
| *Standard bleed volume* $BV_{Standard}$ (%) | ≤ 0.10 | | ✓ |
| *Wick-induced bleed volume* $BV_{Wick}$ (%) | ≤ 0.30 | | ✓ |
| *Pressure-induced bleed volume* $BV_{Pressure,\,350}$ (%) | ≤ 0.10 | ✓ | |
| *Incline tube bleed volume* $BV_{Inclined}$ (%) | ≤ 0.30 | ✓ | |
| *Prototype tendon grouting bleed volume* $BV_{Prototype\ grouting}$ (%) | ≤ 0.30 | ✓ | |
| *Change in length* $\Delta L$ (%) | –0.02% to 0.02 on 28 days | ✓ | |
| | ≤ 0.01 after 1 day | ✓ | |
| *Volume of soft grout* $V_{softgrout}$ | 0 | ✓ | ✓ |

Factory made coated wires or strands embedded in *polyethylene ducts* with suitable fill also are acceptable. When such specialized materials and techniques of prestressing are employed, manufacturer's recommendations shall be complied with.

Grout is a homogenous mixture of cement and water. It may contain chemical admixtures which modify the properties of grout in its fluid state. These recommendations cover the cement grouting of post tensioned tendons of prestressed concrete members.

**(a) Constituent Materials**

*Cement Grouts* shall be made with *cement*, *water* and *admixtures*, whose specifications are as hereunder:

(1) *Water*: Only clean potable water free from impurities conforming to **Clause ___** shall be used. No water from sea, creeks or effluents shall be used.
(2) *Cement*: The same cement as used in construction of prestressed elements, shall be used in the preparation of the grout.
(3) *Admixtures*: Acceptable admixtures conforming to **IS 9103** may be used, if tests show that their use improves the properties of grout, *i.e.,* increases *fluidity*, reduces *bleeding*, *entrains air* and expands the grout. Admixtures shall not contain *chlorides*, *nitrates*, *sulphides*, *sulphites* or any chemical that is likely to damage the steel or the grout. When an expanding agent is used, the total unrestrained expansion shall not exceed 10%. Aluminum powder shall not be used as an expanding agent.
(3) *Sand*: It is not recommended to use sand for grouting of prestressing tendons.

**(b) Use of Grout Colloidal Mixer**

The grout shall be continuously mixed in a colloidal mixer with a minimum speed of 1000 RPM, and with a travel discharge not exceeding 15 *m/s*.

**(c) Properties of Grout**

Before using any grout, the required tests shall be conducted in advance to ascertain the mix of the grout. The properties of the mix shall be tested periodically during the construction of a prestressed member, whose specifications are specified hereunder:

(1) *Water-Cement Ratio*: It should be as low as possible, consistent with the required workability. This ratio shall not exceed 0.45.
(2) *Deleterious Materials*: No chloride or sulphate shall be added separately to the grout. When constituent materials of the grout contain *chlorides*, *sulphates* or *sulphites*, their proportion by weight shall not exceed the following limits:
    (a) Chlorides ($CI^-$) : 0.1%
    (b) Sulphate ($SO_3$) : 4.0%
    (c) Sulphide-ions ($S_2^-$) : 0.01%
(3) *Temperature*: The temperature of the grout (after accounting for the ambient temperature of the structure) shall not exceed 25°C.
(4) *Compressive Strength*: The compressive strength from 100 mm cubes of the grout shall not be less than 17 MPa at 7 days. The cubes shall be cured in a moist atmosphere for the first 24 hours and subsequently in water.
(5) *Setting Times*: The *initial setting time* of grout shall be more than three hours and less than 12 hours. And, the *final setting time* shall not be less than 24 hours.
(6) *Bleeding*: Bleeding in a grout kept at rest for three hours shall not exceed 0.3% of the initial volume of grout, as ascertained from the **____ Test** specified in **IS_____**.
(7) *Volume Change*: The volume change of grout kept at rest for 24 hours shall be within the range –0.5% and 5.0% of the original volume, , as ascertained from the **____ Test** specified in **IS_____**.
(8) *Fluidity*: Fluidity shall be in the range _____, as ascertained from the *Standard Flow Cone Test* specified in **IS_____**.

**(d) Tests for Ascertaining Properties of Grout**

The test set-ups of two tests employed in the evaluation of properties of the grout shall be taken shown in **Figures 6.13 and 6.14**.

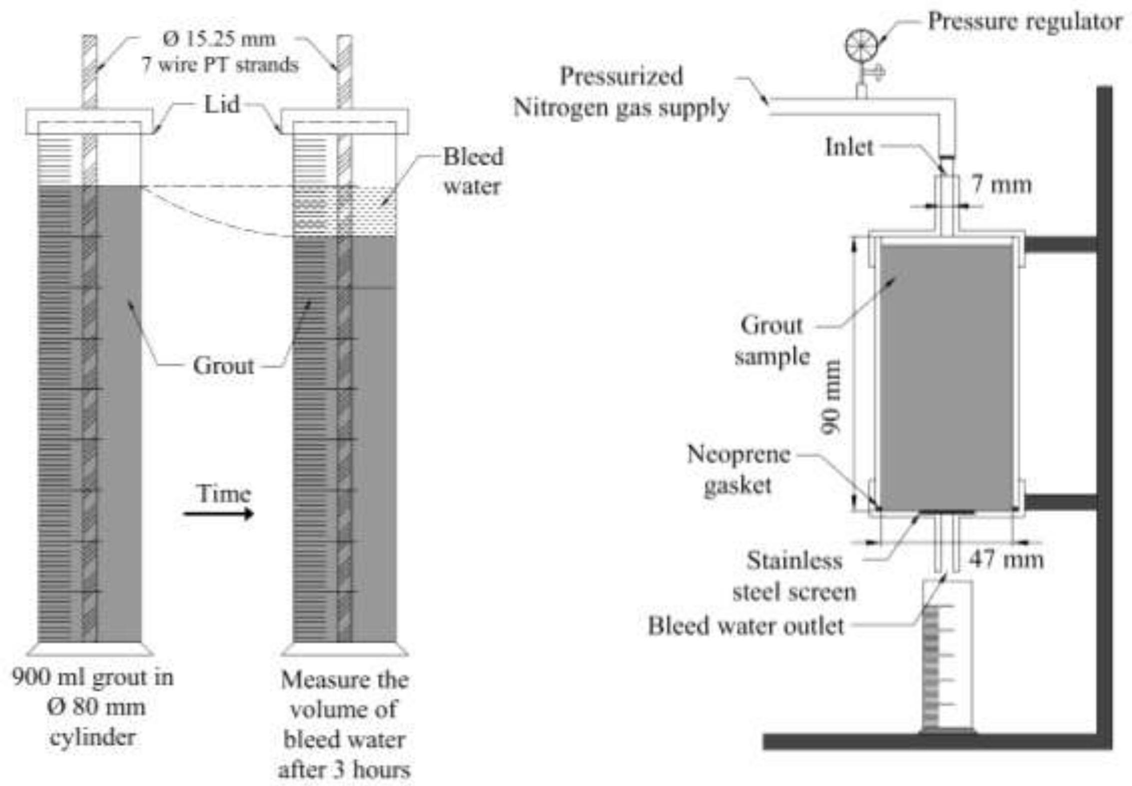


**Figure 6.13**: Schematic representation of the test methods to measure the bleed water: (a) Wick-induced bleed test, and (b) Pressure-induced bleed test

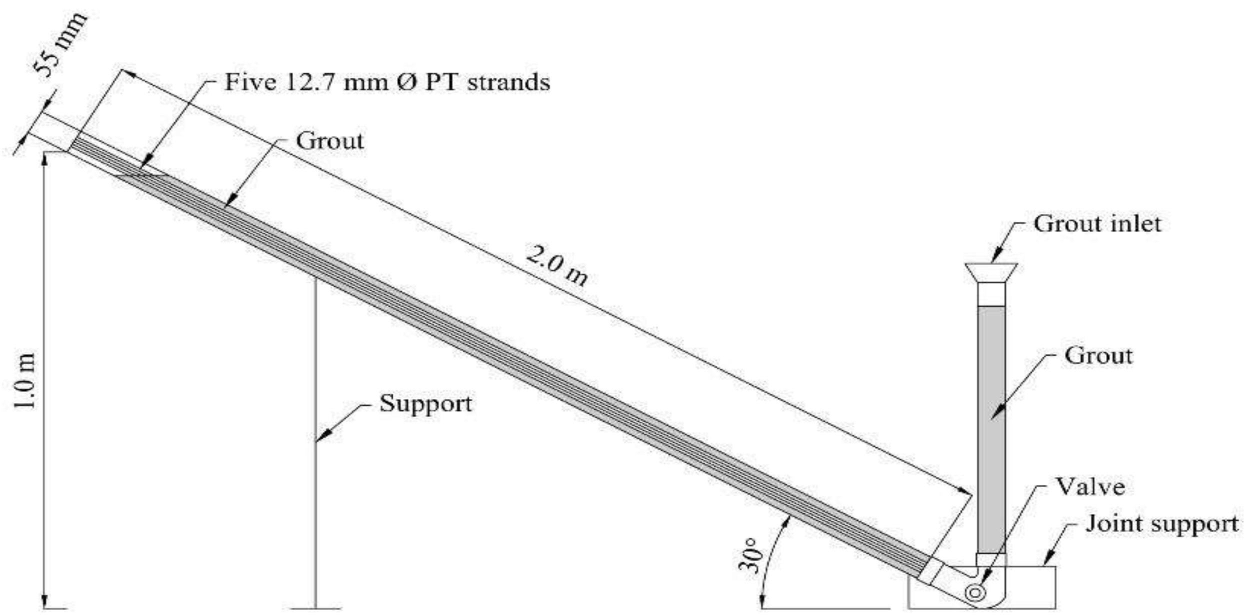


**Figure 6.14**: Schematic representation of the inclined tube-bleed test setup

**C63.1.5** *Grout*

The purpose of grouting is to develop bond between the prestressing steel and the surrounding structural concrete, and to provide permanent protection to the *post-tensioned steel* against corrosion. The grout ensures encasement of steel in an alkaline environment for corrosion protection, and by filling the duct space it prevents water collection and freezing. Thus, it protects prestressing steel from corroding by filling the ducts fully, without leaving any entrapped air or water pockets, voids created by evaporation of excess water in the grout and bleeding. A critical feature of grout is that it is pumpable for the entire duration when it is being injected into the duct.

**(a) Constituent Materials**

Aluminum powder is not recommended to be used as an expanding agent, because its long-term effects are not known.

**(b) Use of Grout Colloidal Mixer**

It is essential that the grout is maintained in a homogenous state and of uniform consistency so that there is no separation of cement during the entire grouting process. Hence, a colloidal mixer is recommended.

**(c) Properties of Grout**

*Bleeding* is the separation of free water from the grout mix. It includes the filtering effect of strands where the cavities between the wires constituting the strand, block cement particles and permit water under pressure to move ahead of the grout in the direction of general flow of grout. The bleeding shall be sufficiently low to prevent excessive segregation and sediment of the grout material.

*Fluidity* of grout changes from time of mixing to time of setting in the ducts. The requirement given above are for general guidance and may be modified as per the specific application, depending upon the total temperature, length of tendons, head of pumping, requirement of simultaneous grouting of closely spaced tendons etc. provided that other specifications and functions are satisfied.

**(d) Tests for Ascertaining Properties of Grout**

The …

**63.2 Grades**

The grades of concrete used in prestressed concrete structures shall:

(1) Not less than M30, and

(2) Not more than M60.

**C63.2 Grades**

The lower limit is specified based on *durability* considerations, and the upper limit based on the fact that basic experiments to examine structural *serviceability*, *stiffness*, *strength* and *deformability*, have not been performed using higher grades of concrete on *prestressed members* (beams, slabs, columns and walls), *prestressed structural sub-assemblages* and *prestressed structural systems*.

**63.3 Physical Properties**

The salient physical properties of prestressing steel tendons shall be taken as specified hereunder.

**C63.3 Physical Properties**

The properties mentioned in this clause are the ones used commonly in design and construction. For other properties, experimental data shall be relied up on from tests conducted on materials; when such data is unavailable, specialist literature may be referred to.

**63.3.1** *Modulus of Elasticity*

The *Modulus of Elasticity* $E_p$ of prestressing steel shall be taken be taken as per manufacturer's test reports for the lot. The value of $E_p$ used in design shall conform to the manufacturer's test value, subject to it complying with the acceptance criteria. The *maximum* and *minimum* value of modulus of elasticity obtained from acceptance tests shall be within a range of 2.5% of the *maximum value*.

When values are not available from the manufacturer, tests shall be performed to ascertain the same. In general, the values shall be around the values indicated hereunder:

$$E_p = \begin{cases} 210{,}000\,\text{MPa} & \text{for Plain Cold Drawn Wires} \\ 200{,}000\,\text{MPa} & \text{for High Tensile Steel Bars (rolled or heat treated)} \\ 195{,}000\,\text{MPa} & \text{for Strands} \end{cases}.$$

**C63.3.1** *Modulus of Elasticity*

The manufacturing process of heating and cooling makes the individual *wires* (which are of smaller diameter) stiffer than the individual *bars* (which are of larger diameter). A larger fraction of the cross-sectional area of *wires* hardens than of *bars*. Hence, *wires* have higher *Modulus of Elasticity* $E_p$ than *bars*. On the other hand, *strands* are made of a group of wires; the individual wires of the strand reduce in diameter when stretched in tension, and thereby tend to come closer in the transverse direction. This transverse movement makes *strands* more flexible than *bars* and *wires*, and hence have lower $E_p$.

**63.3.2** *Poisson's Ratio*

The Poisson's Ratio $\nu$ of prestressing steel of all grades shall be taken as 0.3.

**C63.3.2** *Poisson's Ratio*

The Poisson's Ratio $\nu$ of wires, bars and strands are taken to be the same.

**63.3.3** *Coefficient of Thermal Expansion*

The Coefficient $\alpha$ of Thermal Expansion of prestressing steel of all grades shall be taken as 1.1–1.3 $\times 10^{-5}/^{o}C$.

**C63.3.3** *Coefficient of Thermal Expansion*

The Coefficient $\alpha$ of Thermal Expansion is more for *wires* than *bars*.

**63.3.4** *Strength*

The tensile yield stress of steels for which there is no clearly defined yield point, shall be taken as the 0.2% *Proof Stress*, which represents that strength results in a residual plastic strain of 0.0020 when unloaded from it.

Strength refers to *characteristic strength* $f_y$, and shall mean that the value of 0.2% *Proof Stress* of the reinforcing steel wires, bars or strands below which not more than 5% of the test results are expected to fall.

**C63.3.4** *Strength*

The…

**63.3.5** *Stress Relaxation*

The *stress relaxation* shall be obtained from the *Manufacturers Test Certificates*, and verified independently through tests, if required. In absence of actual test data, the design value of *stress relaxation* up to 30 years shall be taken as three times the relaxation value at 1,000 hours, which is tested at an initial stress of 0.7 times the *Ultimate Tensile Strength (UTS)* at 20°C). For initial stress other than 0.7 times the UTS, the values given in **Table 6.28** shall be used. For test durations less than 1,000 hours, the maximum values of relaxation loss may be taken as per **Table 6.29**.

**Table 6.28:** *Stress relaxation* for different *Initial Stresses* (expressed as percent of initial stress tested at 1,000 hours at 20°C ± 2°C)

| *Initial Stress* | *Relaxation Loss (%)* | |
|---|---|---|
| | *Normal Relaxation Steel* | *Low Relaxation Steel* |
| $\leq 0.5 f_p$ | 0 | 0 |
| $0.6 f_p$ | 2.5 | 1.25 |
| $0.7 f_p$ | 5.0 | 2.50 |
| $0.8 f_p$ | 9.0 | 4.50 |

**Table 6.29:** *Stress relaxation* up to 1,000 hours **(**expressed as percent of initial stress tested at 1,000 hours at 20°C ± 2°C)

| *Time (Hours)* | | *1* | *5* | *20* | *100* | *200* | *500* | *1000* |
|---|---|---|---|---|---|---|---|---|
| % loss of the loss at 1000 hours | *Normal Relaxation Steel* | 34 | 44 | 55 | 70 | 78 | 90 | 100 |
| | *Low Relaxation Steel* | 37 | 47 | 57 | 72 | 79 | 09 | 100 |

The *stress relaxation* $\rho(t)$ at time t (hours) other than at 1,000 hours shall be estimated as:

$$\rho(t) = \rho_{1000}\left(\frac{t}{1000}\right)^k,$$

where

$\rho_{1000}$ = Stress Relaxation value at 1000 hours as per **Table 6.28**, and

$$k = \left(\frac{\rho_{1000}}{\rho_{100}}\right)$$

$$= \begin{cases} 0.155 & \text{Normal Relaxation Steel} \\ 0.143 & \text{Low Relaxation Steel} \end{cases}$$

In pre-tensioned members, stress relaxation shall be considered owing to temperature increase during their curing, which induced thermal strain in members. Hence, for capturing early age relaxation, arising from initial temperatures higher than 40°C (as in steam curing), an *equivalent time* $t_{eq}$ (hours) shall be added to the time $t$ (hours) after tensioning in the relaxation time to account for the effects of the heat treatment on the loss in prestress in the relaxation of the prestressing steel. $t_{eq}$ shall be estimated as:

$$t_{eq} = \frac{1.14^{(T_{\max}-20)}}{T_{\max}-20}\left[\sum_{i=1}^{n}\{(T_{\max}-20)\Delta t_i\}\right],$$

where

$T_{max}$ = Maximum temperature (in °C) during the heat treatment,
$\Delta t_i$ = Time interval for which the Temperature (in °C) is maintained, and
$n$ = Total number of time intervals considered.

**C63.3.5** *Stress Relaxation*

When constant strain is imposed over long time, steel tendons shed stress. This long-term effect should be accounted for in design. Stress relaxation is dependent on the *manufacturing process*. For wires and strands:

(a) *Normal Relaxation Steel*: Wires and strands made of *normal relaxation steel*, which conform to IS 1785 (Part 1), **IS 6003** and **IS 6006**, are required to not have stress relaxation more than 5% at 1,000 hours and 3.5% at 100 hours. Based on these limits, the value of $k$ is estimated as 0.155 for *normal relaxation steels*.

(b) *Low Relaxation Steel*: Wires and strands of *low relaxation steel*, which conform to **IS 14268**, are required to not have stress relaxation more than 2.5% at 1,000 hours and 1.8% at 100 hours. Based on these limits, the value of $k$ is estimated as 0.143 for *low relaxation steel*.

For bars and rods, **IS 2090** does not specify 100 hour relaxation loss value. Also, the 1,000 hour stress relaxation value is not provided. In this case, the $\rho_{1000}$ and $\rho_{100}$ values for steels of different strengths should be obtained from the manufacturer's data. For steels conforming to other national standards, these values should be taken from the respective standards.

**63.4 Untensioned Steel**

Requirements of cover and spacing between bars shall apply as specified in **Clauses 82.2 and 82.4**, and of provisions for assembly of reinforcement as specified in **Clause 83**.

**63.4 Untensioned Steel**

Please see Commentaries of **Clauses 82.2, 82.4 and 83**.

# 64. OTHERS

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### 64.1 FRP Bars

The provisions of this standard are not applicable for members reinforced with FRP bars.

# C64. OTHERS

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### 64.1 FRP Bars

Behaviour is different of concrete members constructed with reinforcing bars made of *other materials*, compared to that when concrete members constructed with reinforcing bars made of *steel*. For example, FRP bars do not have ductility. Hence, when they are used, the members are required to fail only with concrete crushing, and not with FRP bars fracturing. Thus, the philosophy of design itself is different of concrete members made of *FRP* bars as reinforcement from that of concrete members made of *steel* bars as reinforcement.

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