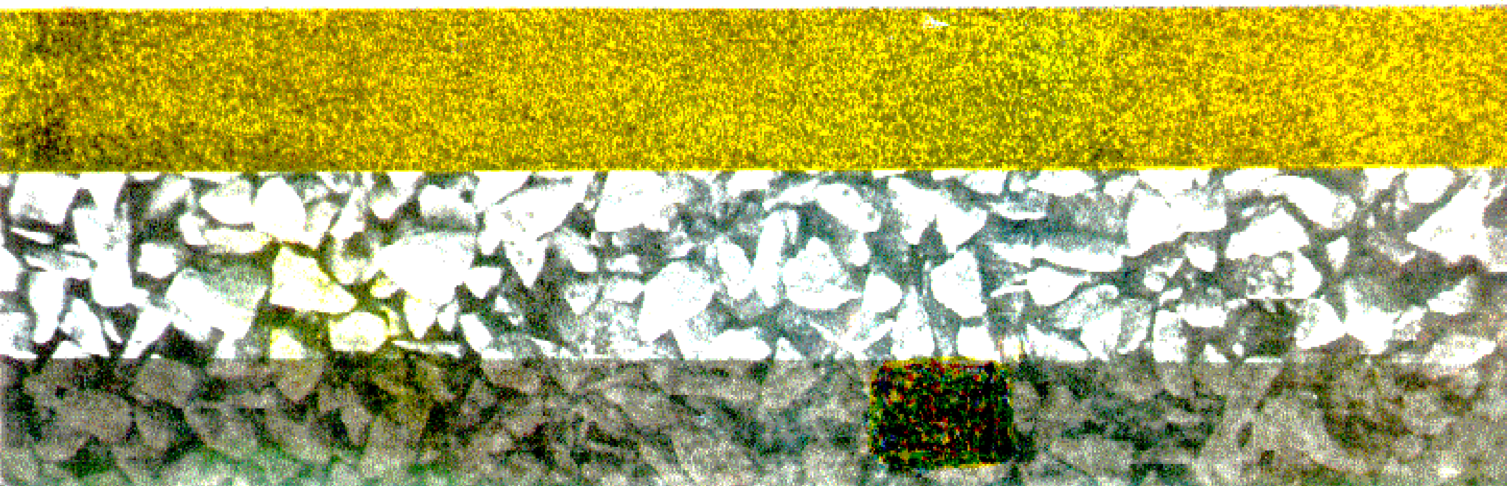
Handbook on Concrete Mixes

BASED ON INDIAN STANDARDS



HANDBOOK
ON
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BUREAU OF INDIAN STANDARDS
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FOREWORD

Users of various civil engineering codes have been feeling the need for explanatory handbooks and other compilations based on Indian Standards. The need has been further emphasized in view of the publication of the National Building Code of India 1970 and its implementation. In 1972, the Department of Science and Technology set up an Expert Group on Housing and Construction Technology under the chairmanship of Maj-Gen Harkirat Singh. This group carried out in-depth studies in various areas of civil engineering and construction practices. During the preparation of the Fifth Five Year Plan in 1975, the Group was assigned the task of producing a Science and Technology plan for research, development and extension work in the sector of housing and construction technology. One of the items of this plan was the production of design handbooks, explanatory handbooks and design aids based on the National Building Code and various Indian Standards and other activities in the promotion of National Building Code. The Expert Group gave high priority to this item and on the recommendation of the Department of Science and Technology the Planning Commission approved the following two projects which were assigned to the Indian Standards Institution:

a) Development programme on Code implementation for building and civil engineering construction, and

b) Typification for industrial buildings.

A Special Committee for Implementation of Science and Technology Projects (SCIP) consisting of experts connected with different aspects (*see* page v) was set up in 1974 to advise the ISI Directorate General for identifying and guiding the development of the work under the chairmanship of Maj-Gen Harkirat Singh, Retired Engineer-in-Chief, Army Headquarters and formerly Adviser (Construction), Planning Commission, Government of India. The Committee has so far identified subjects for several explanatory handbooks/compilations covering appropriate Indian Standards/Codes/Specifications which include the following:

Functional Requirements of Buildings
Functional Requirements of Industrial Buildings
Summaries of Indian Standards for Building Materials*
Building Construction Practices
Foundation of Buildings
Explanatory Handbook on Codes for Earthquake Engineering (IS : 1893-1975 and IS : 4326-1976)†
Design Aids for Reinforced Concrete to IS : 456-1978†
Explanatory Handbook on Mansonry Code†
Explanatory Handbook on Indian Standard Code for Plain and Reinforced Concrete (IS : 456-1978)*
Handbook on Concrete Mixes†

*Under print.
†Printed.

Concrete Reinforcement Detailing
Form Work
Timber Engineering
Steel Code (IS : 800)
Causes and Prevention of Cracks in Buildings*
Plumbing Services
Loading Code
Fire Safety
Prefabrication
Tall Buildings
Design of Industrial Steel Structures
Inspection of Different Items of Building Work
Bulk Storage Structures in Steel
Liquid Retaining Structures

This handbook which has been formulated under this project provides information on the factors that influence concrete mix design and discusses them in detail. Basic features of concrete mix design systems, classification and grade designation have been highlighted. Relevant Indian Standards on materials for concrete and their methods of testing, and other special literatures available on the subject have been taken into consideration in preparing this handbook. The handbook will be useful to designers of concrete structures, field engineers, quality control engineers and laboratories engaged in design, research and testing of concrete mixes.

Some of the important points to be kept in view in the use of this handbook are as follows:

a) This handbook is intended to provide general guidance on the design of concrete mixes. Problems of mix design of special and peculiar nature shall be dealt with on the merits of each case.

b) The mix design is in accordance with the strength requirements and acceptance criteria specified in 'IS : 456-1978 Code of practice for plain and reinforced concrete (*third revision*)'.

c) Wherever there is any dispute about the interpretation or opinion expressed in this handbook, the provisions of the relevant Indian Standards referred to and discussed shall apply. The provisions in this handbook particularly those relating to other literature should be considered as only supplementary and informative.

This handbook is based on the first draft prepared by the Cement Research Institute of India, New Delhi. The draft handbook was circulated for review to Andhra Pradesh Engineering Research Laboratories, Hyderabad; Central Research Station, ACC Research and Development Division, Thane; Central Road Research Institute, New Delhi; Irrigation and Power Research Institute, Amritsar; Hindustan Construction Company Limited, Bombay; Tarapore and Company, Madras; Gammon India Limited, Bombay; and the views received have been taken into consideration while finalizing the Handbook.

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INTRODUCTION

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SECTION 1 INTRODUCTION

1.1 Concrete Mix as a System — Concrete is by far the most widely-used man-made construction material and studies indicate that it will continue to be so in the years and decades to come[1]. Such versatility of concrete is due to the fact that from the common ingredients, namely, cement, aggregate and water (and sometimes admixtures), it is possible to tailor the properties of concrete so as to meet the demands of any particular situation. The advances in concrete technology has paved the way to make the best use of locally available materials by judicious mix proportioning and proper workmanship, so as to result in a concrete satisfying the performance requirements. While the properties of the constituent materials are important, the users are now interested in the concrete itself having desired properties. In the true sense, concrete is thus the real building material rather than the ingredients like cement and aggregates, which are only intermediate products. This concept of treating concrete as an entity is symbolized with the progress of ready-mixed concrete industry, where the consumer can specify the concrete of his needs without bothering about the ingredients; and further in pre-cast concrete industry where the consumer obtains the finished structural components satisfying the performance requirements.

This Handbook, therefore, treats concrete in its entity as a building material. The various aspects covered are the materials, mix proportioning, elements of workmanship (for example placing, compaction and curing), methods of testing and relevant statistical approach to quality control and special precautions needed in extreme-weather concreting. The discussion on these aspects centres around the appropriate provisions in the various Indian Standards which are relevant. However, certain other properties of hardened concrete, namely, elasticity, creep and shrinkage are not covered in this Handbook.

While considering the concrete mixes as a material in itself, note has to be taken of the actions and interactions of its constituents on the characteristics of the end product, which may often give rise to conflicting demands. For example, the requirements of workability demand that water content in the mix should be more, whereas the requirements of compressive strength depend upon lower water-cement ratio and, therefore, the water content be kept as low as practicable. In this context a concrete mix forms a 'system'. Concrete mixes are also characterised by the fact that, unlike the other common structural materials like steel, these are mostly manufactured at site; the inherent variability of their properties and need for proper quality control, therefore, become important considerations.

1.2 Classification of Concrete Mixes — Concrete mixes are classified in a number of ways, often depending upon the type of specifications, which are broadly of two types; the 'prescriptive' specifications where the proportions of the ingredients and their characteristics (namely, type of cement, maximum size of aggregate, etc) are specified, with the hope that adherence to such prescriptive specification will result in satisfactory performance. Alternately, a 'performance' oriented specification can be used wherein the requirements of the desirable properties of concrete are specified (example — strength, workability or any other property). Concrete is accepted on the basis of these requirements being satisfied, and the choice of materials and mix proportions is with the producer.

Based on the above considerations, concrete can be classified either as 'nominal mix' concrete or 'designed mix' concrete as has been specified in IS : 456-1978[2]. British practices go a step further to specify 'standard' mixes[3] or 'prescribed' mixes[4] which are elaboration of 'nominal' mixes, to cater for different ranges of workability and different aggregate characteristics, for the desired compressive strength.

IS : 456[5] had earlier classified concrete into 'controlled' concrete and 'ordinary' concrete, depending upon the levels of control exercised in the works and the method of proportioning concrete mixes. According to this, where the mix proportions were fixed by designing the concrete mixes with preliminary tests were called 'controlled concrete'; whereas 'ordinary concrete' was one where 'nominal' concrete mixes were adopted. This might have inadvertently led to a feeling that no quality control was necessary in case of nominal mixes. However, realizing that mix proportioning is only one aspect of quality control of concrete and that quality control really encompasses many other aspects like choice of appropriate concrete materials after proper tests, proper workmanship in batching, mixing, transportation, placing, compaction and curing, coupled with necessary checks and tests for quality acceptance and quality control, the present concrete code IS : 456-1978[2] makes a significant departure, in that there is nothing like 'uncontrolled' concrete; only the degree of control varies, from 'very good' to 'poor' or no control.

Concrete can be classified in many other ways in special situations; by its density (for example, light weight, normal weight or heavy weight concrete), workability (for example, flowing or pumpable concretes) or its durability in specific environments (for example, sulphate-resisting concrete or its resistance to fire).

1.2.1 GRADES OF CONCRETE — Among the many properties of concrete, its compressive strength is considered to be the most important and has been held as an index of its overall quality. Many other engineering properties of concrete appear to be generally related to its compressive strength. Concrete is, therefore, mostly graded according to its compressive strength. The various grades of concrete as stipulated in IS : 456-1978[2] and IS : 1343-1980[6] are extracted in Table 1. Out of these, two grades, namely, M 5 and M 7.5, are to be used for lean concrete bases and simple foundations for masonry walls. Grades of concrete lower than M 15 are not to be used in reinforced concrete works and grades of concrete lower than M 30 are not to be used for pre-stressed concrete works. Similar grading of concrete on the basis of 28 days characteristic strength has also been adopted by ISO[7] and most of the other codes of practices.

TABLE 1 GRADES OF CONCRETE

(*Clause* 1.2.1)

GRADE DESIGNATION	SPECIFIED CHARACTERISTIC COMPRESSIVE STRENGTH AT 28 DAYS (N/mm²)
M 5	5
M 7.5	7.5
M 10	10
M 15	15
M 20	20
M 25	25
M 30	30
M 35	35
M 40	40
M 45	45
M 50	50
M 55	55
M 60	60

NOTE 1 — In the designation of a concrete mix, letter M refers to the mix and the number to the specified characteristic compressive strength of 15-cm cube at 28 days, expressed in N/mm^2.

NOTE 2 — M 5 and M 7.5 grades of concrete may be used for lean concrete bases and simple foundations for masonry walls. These mixes need not be designed.

NOTE 3 — Grades of concrete lower than M 15 shall not be used in reinforced concrete.

NOTE 4 — Grades of concrete lower than M 30 shall not be used in prestressed concrete.

REFERENCES

1. BURKS (S D). Will concrete be the leading building material of the future? *Pr J. Amer Concr Inst.* 68, 5; 1971; 321-26.
2. IS : 456-1978 Code of practice for plain and reinforced concrete (*third revision*).
3. CP : 116 : Part 2 : 1969 Code of practice for the structural use of precast concrete. British Standards Institution.
4. CP : 110 : Part 1 : 1972 Code of practice for the structural use of concrete. British Standards Institution.
5. IS : 456-1964 Code of practice for plain and reinforced concrete (*second revision*).
6. IS : 1343-1980 Code of practice for prestressed concrete (*first revision*).
7. ISO 3893-1977 Concrete classification by compressive strength. International Organization for Standardization.

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SECTION 2

CONCRETE MAKING MATERIALS

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SECTION 2 CONCRETE MAKING MATERIALS

2.0 General — The common ingredients of concrete are cement, coarse and fine aggregates and water. A fourth ingredient called 'admixtures' is used to modify certain specific properties of the concrete mix in fresh and hardened states. By judicious use of available materials for concrete making and their proportioning, concrete mixes are produced to have the desired properties in the fresh and hardened states, as the situation demands. In this section the physical and chemical properties of the concrete making materials which influence the properties and performance of concrete mixes are discussed.

2.1 Cements — Cement is by far the most important constituent of concrete, in that it forms the binding medium for the discrete ingredients. Made out of naturally occurring raw materials and sometimes blended or interground with industrial wastes, cements come in various types and chemical compositions. For general concrete constructions, IS : 456-1978[1] permits the use of the following types of cement, subject to the approval of the engineer-in-charge:

a) Ordinary or low heat Portland cement conforming to IS : 269-1976[2].

b) Rapid hardening Portland cement conforming to IS : 8041-1978[3].

c) Portland slag cement conforming to IS : 455-1976[4].

d) Portland pozzolana cement conforming to IS : 1489-1976[5].

e) High strength ordinary Portland cement conforming to IS : 8112-1976[6], and

f) Hydrophobic cement conforming to IS : 8043-1978[7].

In addition to these, high alumina cement conforming to IS : 6452-1972[8] and supersulphated cement conforming to IS : 6909-1973[9] can be used under special circumstances with necessary precautions.

For prestressed concrete construction, IS : 1343-1980[10] permits the use of ordinary Portland cement, rapid hardening Portland cement, high strength ordinary Portland cement and Portland slag cement conforming to the respective Indian Standards mentioned above. In case of Portland slag cement, the slag content should not be more than 50 percent.

Among the various types, ordinary Portland cement conforming to IS : 269-1976[2] is perhaps the most common. Further discussions relating to composition and hydration of cements in this clause and in *2.1.1* generally pertain to ordinary Portland cement conforming to IS : 269-1976[2]. Ordinary Portland cement is obtained by intimately mixing together a calcarious material such as limestone or chalk, and an argillaceous material (that is, silica, alumina and iron oxide bearing material), for example, clay or shale, burning them at a clinkering temperature of 1 400 to 1 450°C and grinding the resulting clinker with gypsum. Since the raw materials consist mainly of lime, silica, alumina and iron oxide, these form the major elements in cement also. Depending upon the wide variety of raw materials used in manufacture of cements, the typical ranges of these elements in ordinary Portland cement may be expressed as below:

	Percent
SiO_2	19 - 24
Al_2O_3	3 - 6
Fe_2O_3	1 - 4
CaO	59 - 64
MgO	0.5 - 4

NOTE — It may, however, be noted that the maximum permissible limit of MgO content in ordinary Portland cement is 6 percent (*see* Table 4).

The compounds of these oxides present in the raw materials, however, interact with each other and form a series of more complex products during clinkering. The stage of chemical equilibrium reached during clinkering in the kiln may be disturbed somewhat during cooling. Assuming that cement has the same equilibrium as existing at the clinkering temperature, the basic compound composition of Portland cement along with their ranges may be as shown in Table 2.

The role of each of these compounds in the properties of cement has been studied in length. As indicated in Table 2, their relative proportions in the cement may vary and indeed, the differences in the various types of ordinary Portland cement are really due to the differences in the proportions of these major compounds and fineness. The two silicates (C_3S and C_2S) which together constitute about 70 to 75 percent of the cement, are more important from the considerations of strength giving properties. Upon hydration, that is, reaction with water, both C_3S and C_2S perhaps result in the same product — called calcium silicate hydrate having approximate composition $C_3S_2H_3$ and calcium hydroxide[11]. Because of the similarity of their structures with that of a naturally occurring mineral, the hydrates are called 'tobermorite'. From approximate stoichiometric calculations, C_3S and C_2S need approximately 24 and 21 percent water by weight, respectively, for chemical reaction but C_3S liberates nearly three times calcium hydroxide compared to hydration of C_2S.

The reaction of C_3A with water is very quick and may lead to immediate stiffening of the paste — a phenomenon known as 'flash set'. The role of gypsum added in the manufacture of cement is to prevent such fast reaction. The reaction of gypsum and C_3A with water first gives rise to an insoluble compound called calcium sulphoaluminate (ettringite). But eventually the final product of hydration is possibly cubic crystal of tricalcium aluminate hydrate (C_3AH_6). Approximate stoichiometric calculation shows that C_3A reacts with 40 percent of water by weight, which is more than that required for silicates; however, since the amount of C_3A in cement is comparatively small, the net water required for the hydration of cement is not substantially affected. The products of hydration of C_4AF phase is not so well known. Neville[11] states 'C_4AF is believed to hydrate into tricalcium aluminate hydrate and an amorphous phase, probably $CaO.Fe_2O_3$ aqueous. It is possible also that some Fe_2O_3 is present in solid solution in the tricalcium aluminate hydrate'.

The role of these four major compounds on the properties of cement can be summarised by the kinetics of reaction, development of strength and evolution of heat of hydration of these individual compounds. The state of art can be summarised by Fig. 1[11] and 2[11], and Table 3[12]. From these it will be clear that C_3A and C_4AF are the earliest to hydrate but their direct individual contribution to overall strength development of the cement is perhaps less significant than the silicates. In addition, C_3A phase is responsible for the highest heat evolution both during the initial period as well as in the long run. Among the silicates, C_3S has faster rate of reaction accompanied by greater heat evolution and larger contribution to the initial strength than C_2S phase; however, it is likely that both C_3S and C_2S phases contribute equally to the long term strength of cement.

Apart from the chemical composition, fineness of cement contributes to the kinetics of reaction and initial rate of gain of strength. Generally greater the fineness, greater is the rate of development of strength during the initial period (*see* Fig. 3[13]) and larger is the heat evolution. This is possible because greater fineness enables a larger surface of cement to come in contact with water during the initial period, although the long term effect may not be different. In addition, the particle sizes also influence the hydration and strength at various ages. Particles below 5 micron hydrate within 1 to 2 days and the hydration of 10-25 micron sizes may commence after 7 days.

The different 'types' of cement are made by the adjustment in the relative proportion of chemical compounds and the fineness to suit the particular requirement. A summary of the requirements for physical characteristics and chemical composition of different Indian cements[14] is reproduced in Table 4. It will be seen that whenever a higher rate of initial strength gain is required, this is achieved by grinding the cement to greater fineness and the cement composition perhaps being richer in C_3S and C_3A phases, but it may give rise to more heat of hydration. Contrary to this, the low heat cements would be required to be ground to a lower fineness and would have lower percentage of C_3A and greater percentage of C_2S. The characteristics of different types of cement may be summarised as in Fig. 4[15] and 5[15] which are qualitatively applicable to Indian conditions. From Fig. 5 it is apparent that irrespective of differences in the initial strength, concretes made with different cements tend to have the same long term strength.

2.1.1 HYDRATION OF CEMENTS — The physical properties of concrete depend to a large extent on the extent of hydration of cement and the resultant microstructure of hydrated cement. While the hydration products of individual compounds were described above, it must be realized that the hydration of cement is the collective hydration of each of the compounds present therein and there is no selective hydration of any of the compounds. Nevertheless, the microstructure of hydrated cement is more or less similar to those of the silicate phases. Upon contact with water, the hydration of cement proceeds both inward and outward in the sense that hydration products get deposited on the outer periphery and the nucleus of unhydrated cement inside gets gradually diminished in volume[16]. At any stage of hydration the cement paste (that is, cement + water) consists of the product of hydration (which is called 'gel', because of the large surface area), the remnant of unreacted cement, $Ca(OH)_2$ and water, besides some other minor compounds. Hexagonal prismatic crystals of ettringite are formed first on the tricalcium aluminate phases. Crystals of calcium hydroxide form about four hours after mixing. Thin acicular particles of calcium silicate hydrate start protruding from the surface of cement grains after two hours[17]. In matured pastes, particles of calcium silicate hydrate form an interlocking network and owing to the similarity with the naturally occurring mineral, tobermorite is called 'tobermorite gel'. This gel is poorly crystalline, almost amorphous and appears as randomly oriented layers of thin sheets or buckled ribbon[18]. The thickness of primary 'gel' particles is estimated to be 3.0×10^{-9} to 4.0×10^{-9}m[19].

The products of hydration as described above form a random three dimensional network gradually filling the space originally occupied by water. Accordingly, the hardened cement paste has a porous structure, the pore sizes varying from very small (4×10^{-10}m) to much larger, and are called 'gel' pores and 'capillary' pores[20]. The water present in these pores are held with different degrees of affinity and the pore system inside the hardened cement paste may or may not be continuous. As hydration proceeds, the deposit of hydration products on the original cement grain makes the diffusion of water to the unhydrated nucleus more and more difficult, so the rate of hydration decreases with time.

Each gram of cement of average composition needs about 0.253 g of water for chemical reaction[21]. In addition, a characteristic amount of water is needed to fill the gel pores. The total amount of water thus needed for chemical reactions and to fill the gel pores is about 42 percent[11]. Since hydration can proceed only when the gel pores are saturated, it has often been mistakenly held that water-cement ratio less than 0.40 or so should not be permitted in concretes. However, it must be emphasised that even in presence of excess water, complete hydration of cement never takes place because of the decreasing porosity of the hydration products, nor is it necessary that cement should be fully hydrated[21]. In fact, water-cement ratio less than 0.40 is quite common in structural concretes, more so in high strength concretes.

In concretes, the hardened cement paste is thus a porous ensemble; the concentration of solid products of hydration in the total space available (that is original water + hydrated cement) is an index of porosity. Like any other porous solid, the compressive strength of cement pastes (or concretes) is related to the parameter gel-space ratio[20] or hydrate-space ratio[22] (*see* Fig. 6). The water-cement ratio, which is held as the most important parameter governing compressive strength (*see 3.2*), is really an expression of the concentration of hydration products in the total volume at a particular age for the resultant degree of hydration[22].

2.1.2 PORTLAND POZZOLANA AND SLAG CEMENTS — Among cements of different types, mention may be made of Portland pozzolana cement conforming to IS : 1489-1976[5] and Portland slag cement conforming to IS : 455-1976[4] because of increased production of these cements in the country mainly to offset the shortage of ordinary Portland cement.

Portland slag cement is manufactured by intergrinding ordinary Portland cement clinker with granulated slags obtained as a by-product from the manufacture of steel. The slags have more or less the same constituents as in ordinary Portland cement in varying proportions, depending upon the processes involved. Typical oxide compositions of Indian slags suitable for the manufacture of Portland slag cement are as follows[4]:

	Percent
SiO_2	27-32
Al_2O_3	17-31
FeO	0-1
CaO	30-40
MgO	0-17

The slag should, however, be in glassy form. IS : 455-1976[4] permits the proportion of slag to be in the range of 25 to 65 percent. The products of hydration of such Portland slag cement are believed to be similar to that of ordinary Portland cement; $Ca(OH)_2$ liberated by the hydration of ordinary Portland cement acts as an activator for the reaction of slag[12]. Since the hydration of slag component depends initially upon liberation of $Ca(OH)_2$, the rate of development of early strength may be somewhat slower (*see* Fig. 7[11]). However, for all engineering purposes, Portland slag cement may be held to be similar to ordinary Portland cement and the requirements of physical characteristics are also identical in both cases. Portland slag cements give lower heat of hydration and better sulphate resistance[11].

Portland pozzolana cement (*see* IS : 1489-1976[5]) is made by blending or intergrinding reactive pozzolana (for example, flyash, burnt clay, diatomaceous earth, etc) in proportions of 10 to 25 percent with ordinary Portland cement. Pozzolanas as such do not possess cementitious property in themselves but in combination with $Ca(OH)_2$ liberated from the hydration of ordinary Portland cement, give rise to cementitious products at room ambient temperature and the ultimate products of hydration in both cases are believed to be identical[23]. The requirements of 7-day strength of Portland pozzolana cement (*see* IS : 1489-1976[5]) are the same as that of ordinary Portland cement (21.5N/mm²). The use of Portland pozzolana cement is recommended in IS : 456-1978[1] as substitute for ordinary Portland cement for plain and reinforced concrete work in general building construction. In addition to 7-day compressive strength, IS : 1489-1976[5] specifies the minimum 28-day compressive strength of Portland pozzolana cement. However, for the reasons cited, the rate of development of early strength may be somewhat lower (*see* Fig. 8[24]) and concrete made with Portland pozzolana cement may need somewhat longer curing period under field conditions, delayed removal of formwork, etc. Portland pozzolana cement also has the advantage of lower heat of hydration and better sulphate resistance; in fact these properties led to its wide application in USA[25].

2.1.3 TESTS ON CEMENTS — The usual tests made on cement are: fineness, setting time, soundness, heat of hydration, compressive strength and chemical composition. All physical and chemical composition tests are carried out in accordance with the procedures described in IS : 4031-1968[26] and IS : 4032-1968[27].

The Blaines air permeability method is used for determining the fineness of cement. The method is based on the permeability to flow of air through a bed of the cement. The fineness is expressed as specific surface area per gram of cement.

The setting times are measured by Vicat apparatus, with different penetrating attachments. The term setting is used to describe the stiffening of the cement paste, and the terms 'initial set' and 'final set' are used to describe arbitrary chosen stages of setting.

The soundness of cement is determined in an accelerated manner by Le-Chatelier apparatus. This test detects unsoundness due to free lime only. Unsoundness due to magnesia present in the raw materials from which cement is manufactured can be determined by autoclave test. This test is sensitive to both free magnesia and free lime[11]. In this test high pressure steam accelerates the hydration of both magnesia and lime. The results of the autoclave test are affected by, in addition to the compounds causing expansion, the C_3A content. The test gives thus no more than a broad indication of the long-term expansion expected in service.

The heat of hydration is the amount of heat in calories per gram of unhydrated cement, evolved upon complete hydration at a given temperature. The method of determining the heat of hydration is by measuring the heats of solution of unhydrated and hydrated cement in a mixture of nitric and hydrofluoric acids: the difference between the two values gives the heat of hydration. The heat of hydration thus measured consists of the chemical heat of the reactions of hydration and the heat of absorption of water on the surface of the gel formed by the processes of hydration. The heat of hydration is required to be determined for low heat Portland cement, as specified in IS : 269-1976[2].

The compressive strength of cement is determined on 1:3 cement-sand mortar cube specimens with standard graded sand, cast and cured under controlled conditions of temperature and humidity. The water content is determined as $(\frac{P}{4} + 3)$ percent by weight of cement and sand, where P is percentage of water required for standard consistency. In most of the cases, it corresponds to a water-cement ratio of 0.37 to 0.42.

The chemical analysis is carried out to determine the oxide composition of cement. The percentages of main compound in cement (that is, C_3S, C_2S, C_3A and C_4AF) can be calculated from oxide composition using Bogue's equations[12], which is applicable to ordinary Portland cement only. In addition to the main compounds, two of the minor compounds are of interest. They are alkalis—Na_2O and K_2O. The insoluble residue determined by treating with hydrochloric acid, is a measure of impurities in ordinary Portland cement; largely arising from impurities in gypsum. The loss on ignition shows the extent of carbonation of free lime and hydration due to the exposure of cement to the atmosphere.

2.2 Aggregates — Aggregates which occupy nearly 70 to 75 percent volume of concrete are sometimes viewed as inert ingredients in more than one sense. However, it is now well recognised that physical, chemical and thermal properties of aggregates substantially influence the properties and performance of concrete. A list of properties of concrete which are influenced by the properties and characteristics of aggregates is given in Table 5[28]. Proper selection and use of aggregates are important considerations, both economically as well as technically. Aggregates are generally cheaper than cement and imparting greater volume stability and durability to concrete.

2.2.1 CLASSIFICATION OF AGGREGATES

2.2.1.1 GENERAL — General classification of aggregates can be on the basis of their sizes, geological origin, soundness in particular environments, unit weight or on many other similar considerations as the situation demands. In so far as the sizes are concerned, aggregates range from a few centimetres or more, down to a few microns. The maximum size of aggregate used in concrete may vary, but in each case the aggregate is to be so graded that particles of different size-fractions are incorporated in the mix in appropriate proportions. As per IS : 383-1970[29] fine aggregates are those, most of which pass through 4.75 mm IS sieve; aggregates, most of which are retained on 4.75 mm IS sieve are termed as 'coarse' aggregates. Sand is generally considered to have a lower size limit of about 0.07 mm. Materials between 0.06 mm and 0.002 mm are classified as silt, and still smaller particles are called clay. Sometimes combined aggregates are available in nature comprising different size-fractions of the above classification, which are known as 'all-in-aggregates'. In such cases they need not be separated into fine and coarse fractions but adjustments often become necessary to supplement the grading by the addition of respective size fractions which may be deficient in the total mass. Such 'all-in-aggregates' are generally not found suitable for making concrete of high quality. Aggregates comprising particles falling essentially within a narrow limit of size fractions are called 'single-size' aggregates.

2.2.1.2 GEOLOGICAL CLASSIFICATION OF NATURAL AGGREGATES — Aggregates for concrete are generally derived from natural sources which may have been naturally reduced to size (for example, gravel or shingle) or may be required to be crushed. As long as they conform to the requirements of IS : 383-1970[29] and concrete of satisfactory quality can be produced at an economical cost using them, both gravel/shingle or crushed natural aggregate can be used for general concrete construction. Aggregates can be manufactured from industrial products also, which are used for special purposes, for example, light weight concretes, concretes requiring better thermal insulating properties, etc. From the petrological stand-point, the natural aggregates, whether crushed or naturally reduced in size, can be divided into several groups of rocks having common characteristics. Natural rocks can be classified according to their mode of formation (for example ignious, sedimentary or metamorphic origin) and under each class they may be further sub-divided into groups having certain petrological characteristics in common. Such a classification adopted in IS : 383-1970[29] is reproduced in Table 6. Depending upon the minerals found in aggregates the mineralogical classification can also be made. However, such classifications are not very helpful in predicting the performance of the aggregates in concrete. This is so, because each rock will probably have a number of minerals present and even among the most abundant minerals in a particular aggregate it is difficult to classify one being universally desirable or otherwise.

2.2.2 PROPERTIES OF NATURAL AGGREGATES — As pointed out earlier, the properties and performance of concrete are dependent to a large extent on the characteristics and properties of aggregates themselves, and knowledge of the properties of aggregates is thus important. In the cases of marginal aggregates the record of performance of concretes made with them may be the best guide. However, tests in the laboratory as well as petrographic examinations are used in most general cases.

2.2.2.1 MECHANICAL PROPERTIES — The significance of the various tests for mechanical properties are discussed below:

a) *Tests on strength of aggregate* — The strength of aggregates in the conventional sense may appear to be not a criterion in so far as aggregates are generally of an order of magnitude stronger than the concretes made with them and a notional feeling that stronger aggregates are better may be sufficient. However, the localized stresses in an element of concrete may be much higher than the overall strength of concrete due to stress concentrations and in case of high strength concrete the mechanical strength of aggregates may itself become critical. Moreover, the mechanical strength of aggregates is important from the point of view of quarrying, stability in the mixer, better resistance to abrasion or attrition during subsequent service life of concrete. Quite often among aggregates of similar geological classifications one having higher mechanical strength has been found to be sounder in chemical environments. IS : 2386 (Part IV)-1963[30] prescribes the following three tests for testing the strength of aggregates:

1) Crushing strength,
2) Crushing value, and
3) Ten percent fines value.

The tests on crushing strength do not give very reproducible results but essentially measure the quality of the parent rock rather than those of the aggregates derived from it. This test may be useful for assessments of new sources of aggregates without proven records.

Among these three, crushing value test, which is performed on bulk aggregates is more popular and results are reproducible. IS : 383-1970[29] specifies limits of crushing value as 45 percent for aggregates used for concrete other than for wearing surfaces and 30 percent for concrete for wearing surfaces, such as runway, pavement and roads. For weaker aggregates with a crushing value of over 25 to 30 percent, crushing value, test is not so reliable in the sense that material crushed before the full load having been applied tends to get compacted thereby inhibiting crushing at a later stage and the intrinsic value may not be measured in such cases. For such situations 'ten percent fines value' test may be more reliable which measures the load required to produce 10 percent fines from 12.5 to 10 mm particles. There is not much data available correlating the 10 percent fines value with the crushing value as given in IS : 383-1970[29] which does not specify any limit for this test. BS : 882:1965[31] prescribes a minimum value of 10 tonnes for aggregates to be used in wearing surfaces and 5 tonnes when used in other concretes.

Another related aspect is the toughness of aggregates which is a measure of the resistance of the material to failure by impact. IS : 2386 (Part IV)-1963[30] prescribes a method for determining the impact value which is sometimes taken as an alternative to crushing value test. The results also in general correspond to each other and the requirements of IS : 383-1970[29] are similar to those for crushing value test. This is a convenient test which can be carried out in the site laboratory.

b) *Hardness and abrasion resistance* — In addition to crushing strength and toughness resistance, abrasion resistance is an important consideration specially for concretes exposed to wearing actions. Concretes made with aggregates having good abrasion resistance are necessary for making concrete which will be subjected to abrasion and attrition during service. More than that, the abrasion and attrition resistance of aggregates are also important to assess the likelihood of breakage during handling and stockpiling as well as during mixing in a mixer. Recent tests conducted by NRMCA[32] have reported certain fine aggregates degrading due to attrition during mixing, more so in prolonged mixing in case of ready-mix concrete, with consequent increase in the proportion of fines in the combined aggregates thereby lowering the workability. In these tests, the sand samples were considered otherwise 'satisfactory'. Some typical results as to the effect of prolonged mixing on fineness modulus of sand and gravel are reproduced in Fig. 9.

IS : 2386 (Part IV)-1963[30] recommends Los-Angeles test for the hardness and abrasion resistance of aggregates in addition to scratch test essentially for the detection of soft particles. The Los-Angeles attrition test combines test for attrition and abrasion and is quite popular. This test is found to be more representative of the actual performance expected of the aggregates and results can also be correlated with other mechanical properties of aggregates. IS : 383-1970[29] requires that a satisfactory aggregate should have Los-Angeles abrasion value of not more than 30 percent for aggregates used for wearing surfaces and 50 percent for aggregates used for non-wearing surfaces. Table 7[10] indicates the type of relationship that can be expected between various tests for different rock groups.

2.2.2.2 PARTICLE SHAPE AND TEXTURE — The external characteristics of mineral aggregates in terms of physical shape, texture and surface conditions significantly influence the mobility of the fresh concrete and the bond of aggregates with the mortar phase. Two relatively independent properties, sphericity and roundness define the particle shape. Sphericity is defined as a function of the ratio of the surface area of the particle to its volume whereas roundness measures the relative sharpness or angularity of the edges and corners of a particle. To avoid lengthy descriptions of the aggregate shape, IS : 383-1970[29] lists four groups of aggregates in terms of particle shape (*see* Table 8). Well rounded particles require less water and less paste volume for a given workability; nevertheless, crushed or uncrushed rounded gravels generally tend to have a stronger aggregate-mortar bond and result in substantially the same compressive strength for a given cement content. The unit water content could be reduced by 5 to 10 percent and sand content by 3 to 5 percent by the use of rounded gravel. Use of crushed aggregates, on the other hand, may result in 10 to 20 percent higher compressive strength. For water-cement ratios below 0.4, the use of crushed rock aggregate has resulted in strengths up to 38 percent higher than when gravel is used[11]. Elongated and flaky particles, having a high ratio of surface area to volume, lower the workability of the mix and can also affect adversely the durability of concrete since they tend to be oriented in one plane with water and air voids underneath. A flakiness index not greater than 25 percent is suggested for coarse aggregates.

Surface texture is the measure of polish or dullness, smoothness or roughness and the type of roughness of the aggregates. IS : 383-1970[29] classifies surface characteristics of the aggregate into five headings or groups (*see* Table 9). The grouping is broad and it does not purport to be a precise petrographical classification, but is based upon a visual examination of hand specimens. Rough porous texture is preferred to a smooth surface; the former can increase bond of cement paste by 1.7 times and leads to 20 percent more flexural and compressive strength in concrete.

The shape and texture of fine aggregate significantly affect the water requirement of the mix. As a typical example, Fig. 10[11] shows the influence of void content (indirect expression of shape and texture of fine aggregate) of sand in a loose condition on the mixing water requirement of concrete.

2.2.2.3 POROSITY AND ABSORPTION — Porosity, permeability and absorption of aggregate influence the bond between aggregate and cement paste, the durability of concrete with regard to the aggressive chemical agencies, resistance to abrasion of concrete, and freezing and thawing; out of these, resistance to freezing and thawing is not an important consideration in the conditions prevailing in India, unlike the colder climates of western countries. The porosity of some common rocks is given in Table 10[11].

The water absorption properties of aggregates are important in the sense that depending upon the condition in which the aggregates are used that is, saturated, surface dry, dry or bone dry. Porous may become reservoir of free moisture inside the aggregates. Afterwards this moisture may be available for hydration or may actually extract some water used for mixing and the entire water may not be available for perfect workability and subsequent hydration of cement. The absorption of water by aggregate is determined by measuring the increase in weight of an oven-dried sample after immersion for 24 hours. However, the absorption of water from the mix by dry aggregates is somewhat proved as the paste or mortar paste surrounds the surface and indeed the absorption may not proceed to the full and may come to an end within first 10 or 20 minutes. Under such circumstances the water absorption of aggregates in the first 10 or 20 minutes is sometimes more meaningful than the full absorption capacity determined in 24 hours. Some typical ranges of values of different rocks are given in Table 11[33]. Since the outer layer of the gravel particles can be more porous and absorbent due to weathering, gravel generally absorbs more water than crushed rock of the same petrological character.

2.2.2.4 DELETERIOUS CONSTITUENTS — A number of materials may be considered undesirable as constituents in aggregates because of their intrinsic weakness, softness, fineness and other physical characteristics, the presence of which may affect the strength, workability and long-term performance of concrete. By way of their actions, these can be classified as falling into any one of the following categories:

a) Which are present as coating around the aggregates and may interfere with the bond characteristics.

b) Which are essentially fine particles and increase the total specific surface area of the aggregate thereby affecting the workabililty.

c) Which are themselves soft and friable, and can be considered as a weak inclusion in the composite, being potential side of stress concentration.

d) Which can affect the chemical reactions of hydration of cement.

IS : 383-1970[29] identifies iron pyrites, coal, mica, shale or similar laminated material, clay, alkali, soft fragments, sea shells, organic impurities, etc, as such undesirable ingredients in aggregates. It has to be remembered that each of these will have actions falling in one or more of the above categories; for example clay can be present as coating around the aggregate thereby reducing the bond. They can be present as fine particles which increase the water demand for a particular workability and they themselves are soft enclosures. It is difficult to precisely tell what proportion of each will definitely pose adverse effects on the properties of concrete as it depends upon the particle size and shape, the size of the concrete members, the distribution of such impurities in the aggregate and above all the exposure conditions. Nevertheless many codes of practices including IS : 383-1970[29] have set limits about the presence of deleterious constituents and have prescribed relevant methods of tests to determine the amount thereof. The related Table from IS : 383-1970[29] as to the limit of deleterious materials in aggregates is reproduced in Table 12. The salient points about deleterious materials are discussed below:

a) *Clay lumps, clay and silt* — Clay lumps could be of two types:

1) those which will get broken during

the mixing operations; and

2) those which survive mixing operation and will be present in the concrete.

The amount of clay lumps which can be handpicked is required to be limited to one percent of the clay. The proportion of clay which is likely to be broken down in mixing is included in the test for 'material passing 75 microns IS sieve' which also include the other fines in the aggregate like silt and fine dust. Another test prescribed in IS : 2386 (Part II)-1963[34] to determine specifically the proportion of clay, silt and fine dust is by sedimentation method. While IS : 383-1970[29] prescribes a limit of 3 percent for fine aggregates and in crushed coarse aggregates or natural coarse aggregates and, one percent for crushed rocks for the material passing 75 micron IS sieve, there is no corresponding limitation in the deleterious constituents detected by the sedimentation test. The deleterious constituents determined by these two test methods may or may not be the same. However, the British practice limits the proportion of clay silt and fine dust content in aggregates (as determined by the sedimentation method) to 3 percent by weight in sand and one percent by weight of crushed coarse aggregates.

The effect of such fine particles on the workability and strength of concrete may be appreciated by the fact that for every one percent clay in the fine aggregate, the compressive strength of concrete can decrease by 5 percent (*see* Fig. 11[35]).

b) *Lightweight and soft fragments* — IS : 383-1970[29] sets different limits for lightweight pieces and soft fragments in the aggregates (*see* Table 12) and gives two different test methods to determine the same; however, in practice a deleterious constituent may indeed be falling under both categories. Lightweight pieces are essentially coal and lignite and the method of determination is by gravity separation in a liquid of specific gravity 2.0. Coal and lignite may cause localised pitting and staining in concrete surfaces. In addition, some particles specially of softer variety may swell when come in contact with moisture and thereby weaken the concrete. Because of the staining where the appearance of the finished surface is a criterion, the permissible amount of coal and lignite is limited to 0.5 percent. It may be noted that all the particles having specific gravity less than 2.0 may not be coal and lignite alone, and may include other type of particles which may not have such deleterious effects on surface appearance. That is why some specifications [for example, ASTM C33[36] and IS : 2386 (Part II)-1963[34]] imply that particles having specific gravity less than 2.0 and of black and brownish-black colour are considered as coal and lignite.

The test (scratch-hardness) for soft fragments as prescribed in IS : 2386 (Part II)-1963[34] is essentially to detect materials which are so poorly bonded that the separate particles in the piece are easily detached from the mass. Soft inclusions, such as clay lumps, shale, wood and coal are included in the test for lightweight pieces if the specific gravity is lower than 2.0. Such soft small fragments when present in larger quantities (2 to 5 percent) may lower the compressive strength of concrete by being a weak spot in the composite and giving rise to stress concentration as discussed earlier. IS : 383-1970[29] specifies that soft fragments shall not be more than 3 percent in natural coarse aggregates whereas the shale content in natural fine aggregates is limited to one percent.

c) *Organic impurities* — Organic impurities like those resulting from products of decay of vegetable matter interfere with the chemical reactions of hydration of cement. Such organic impurities are more likely to be present in fine aggregates; those in coarse aggregates can be removed during washing. Not all the organic matter present in aggregates may be harmful and in most cases it is desirable to test the strength properties made with such aggregates (mostly fine aggregates) and compare with those made with aggregates known to be free from organic impurities. This test is, however, resorted to only when the organic impurities are detected by a colorimetric test as prescribed in IS : 2386 (Part II)-1963[34]. In this test the change in the colour of a 3 percent sodium hydroxide solution, in contact with the sample for 24 hours indicates the presence of organic matter; darker the colour, more the organic content. The colour of the solution is compared with a standard solution. This test is mainly a negative test which means that if there is no colour change, no organic matter is present but if the colour changes, the presence of organic matter is indicated. Another test prescribed in IS : 2386 (Part VI)-1963[37] for mortar making properties of fine aggregates is comparative test on the basis of strengths of mortar made with suspected sand and good sand as enumerated earlier.

d) *Mica in aggregates* — Mica which is often considered to be harmful for concrete may be present in almost all river sands although the proportion may vary from negligible to substantial amount. The mica being flaky and laminated in structure affects the strength and workability of concrete the way other flaky and laminated particles do. The effect on durability will mostly result from the unsatisfactory workability of the fresh concrete and leads to increased permeability; in addition Neville has contemplated that in the presence of active chemical agents produced during the hydration of cement, alteration of mica to other forms may result. Mica is generally found in two varieties : Muscovite, $KAl_2 (Si_3Al) O_{10} (OH_2)$, is potassium aluminium silicate which is colourless or has a silvery or pearly lustre. The second variety of mica biotite which is black brown or dark green in colour, is a complex silicate of potassium, magnesium, iron and aluminium $K_2(MgFe)_{16} — (SiAl)_8 O_{20} (OH_4)$. Of these two, the muscovite variety is now believed to be more harmful for concrete. To give an idea of their related influence if 1 to 2 percent muscovite mica brings down the strength of concrete by 15 percent the same reduction may be expected in the presence of 10 to 15 percent of biotite type mica in concrete sand. In most situations both the varieties are present together and if the muscovite variety is more prominent it poses more problems.

The amount of mica present in some of the Indian river sands is given in Table 13[38,39]. It may be seen that in many cases the mica content could be as high as 12 percent and the problem is often aggravated by the fact that the sand is of finer variety, the fineness modulus being as low as 0.6 or so in the case of sand from river Ganges.

IS : 383-1970[29] does not specify upper limit of mica in fine aggregates; however, a cautionary note is added stating that the presence of mica in the fine aggregate has been found to reduce considerably the compressive strength of concrete and further investigations are underway to determine the extent of the deleterious effect of mica. It is advisable, therefore, to investigate the mica content of fine aggregate and make suitable allowances for the possible reduction in the strength of concrete or mortar. IS : 383-1970[29], however, specifies tolerable limits of mica in terms of deleterious contents of alkali mica and coated grains taken together and these substances are limited to 2 percent by weight in sand. In the investigations done so far on the extent of deleterious effect of mica it has been found that under such situation mica content as low as even one percent has resulted in adverse effects on the properties of concrete and mortar, specially of muscovite variety. It has generally been seen that the workability[40,41] is adversely affected when the mica content is of the order of 5 percent or more; above 8 percent the mortar mixes may indeed become unworkable or the water content may become too high. The effect on workability and strength is generally more the leaner the mix; in one case with 2 percent mica content the compressive strength has been reported to go down by 11 to 19 percent at the age of 7 days and 24 to 30 percent at 28 days. With 12 percent mica content, the respective reductions were reported to be 40 to 50 percent at 7 days and 47 to 55 percent at 28 days. The tests on durability have been on the basis of abrasion resistance, percentage loss of weight of mortar specimens exposed to 10 alternate cycles of wetting in concentrated sodium sulphate solution and drying, and tests on coefficient of permeability. The results indeed are of relative nature. Nevertheless it can be concluded that mica content in excess of 6 percent may be considered to be deleterious for good abrasion resisting concrete. The coefficients of permeability for typical concrete mixes were found to increase about 10 to 20 times as the mica content increased from 2.5 percent to 8.7 percent.

From the foregoing discussions while it will be very difficult to set a limit for the mica content in sand in most of the cases, it would be necessary to carry out trial mixes with such sand and adjust the mix proportions so as to result in satisfactory compressive strength, adequate workability and permeability of concrete as the situation will demand. The beneficiation of sand involving removal or reduction in the mica content in sand by floatation, density separation or wind blowing or any other method may be considered if found to be more economical than adjustments in mix proportions or procurement of good sand from another source.

In so far as determination of mica content in the fine aggregate is concerned, there is no specific test in Indian Standards. However, two commonly adopted methods are based on the consideration that mica particles of the muscovite and biotite varieties differ from other deleterious constituents of sand, such as clay and silt, with respect to their specific gravity which ranges from 2.8 to 3.0. Mica particles also differ from the siliceous particles in sand in that they are more angular and flaky. These differences in physical properties can be taken advantage of, to separate out mica from sand, by adopting the following procedures[41].

1) *Floatation or density separation method* — In this method, an inert liquid medium consisting of a solution of potassium iodide in mercuric iodide having density within the range of 2.8 to 3.0 is used. When sand is immersed in this solution, the relatively lighter siliceous particles float and mica particles having comparatively higher density, settle at the bottom of the solution. The floating sand particles are removed and washed free of chemicals. The liquid medium is usable again.

2) *Wind-blowing method* — In this method, air is blown against a thin column of sand from a funnel into an improvized channel. This results in flaky mica particles being carried to a greater distance than the sand particles of corresponding size, and thus resulting in the separation of mica particles from sand. The actual distance to which the particles are blown depends upon the velocity of air. For finer fraction, a velocity of 19 km/hour and for coarser fraction, a velocity of 26 km/hour are recommended.

The floatation method for determination of mica content in sand is more accurate than the wind-blowing method, although it involves a costly liquid medium. The efficiency of wind-blowing method depends on the texture and shape of the mica grains being different from those of the siliceous grains composing sand.

e) *Salt contamination in sea dredged aggregates* — Aggregates dredged from sea are also used for making concrete, though to a very limited extent. Properties peculiar to sea-dredged aggregates are that the salt primarily sodium chloride (NaCl) and sea shell are present more frequently. Sea-dredged aggregates may also contain organic impurities due to sea-weed, dead fish, coal, oil and disposal at sea.

The salt content in the sea-dredged aggregates is directly proportional to the moisture content at the time of dredging but the final salt content depends upon the amount of washing and the type of water used for washing. Approval for the use of sea-dredged aggregates is not normally granted in the following situations[42]:

1) Where calcium chloride is also used;
2) Where the cement to be used is other than ordinary Portland cement or rapid-hardening Portland cement or sulphate resisting Portland cement; and
3) Where the concrete is to be prestressed or steam cured.

Where such approval is granted, the sodium chloride content of the fine and coarse aggregate shall not exceed, respectively, 0.10 percent and 0.03 percent by weight of dry aggregate. If either aggregate exceeds the limits, the total sodium chloride concentration from the aggregate shall not exceed 0.32 percent[42] by weight of the cement in the mix. In any case, the total amount of chloride and sulphate ions from all sources that is, aggregates, cement, water and admixtures, should not exceed the value specified in IS : 456-1978[1] and IS : 1343-1980[10].

Shell, that is calcium carbonate, in sands has no harmful effect and quite good concrete can be produced if the shell is present in quantities up to 20 percent. But the hollow or large flat shells have detrimental effect as regards to durability of concrete as they adversely affect the quality and permeability of the concrete. The shell content of the aggregates shall not exceed 2, 5, 15 percent, respectively, for 40, 20, 10 mm nominal sizes of coarse aggregate and 30 percent for fine aggregate[42].

2.2.2.5 SOUNDNESS OF AGGREGATES — Aggregate is said to be unsound when it produces excessive volume changes resulting in the deterioration of concrete under certain physical conditions, such as freezing and thawing, thermal changes at temperatures above freezing and alternate wetting and drying. Frost damage to concrete is distinct from the expansion as a result of chemical reactions between the aggregate particle and the alkalis in cement. Certain aggregates such as porous cherts, shales, some limestones particularly laminated limestones and some sandstones, are known to be susceptible to this frost damage. High absorption is a common characteristics of these rocks with a poor service-record, though many durable rocks may also exhibit high absorption. Thus, critical conditions for frost damage are water content and lack of drainage. These characteristics of high absorption and lack of drainage depend upon the pore characteristics of aggregate. It has been reported in the case of some aggregates that porosities in the region of +8 micron (as determined by mercury-intrusion porosity test) appear to separate the high from the low durability aggregates.

IS : 2386 (Part V)-1963[43] specifies a test for determining soundness of aggregates. This test popularly known as 'sulphate test' consists of subjecting a graded and weighed sample of aggregate to alternatively immersion in saturated solution of sodium sulphate or magnesium sulphate and oven drying and thus determining the weight loss after specified cycles of immersion and drying. The overall mechanism of this sulphate test is yet not fully understood but probably this test involves a combination of actions that is, pressure of crystal growth, effects of heating and cooling, wetting and drying and pressure development due to migration of solution through pores but this test in no way is considered to simulate exposure of concrete to freezing and thawing due to complexity of field exposure and does not provide a reliable indication of field performance. Consideration should always be given to the service record of the aggregate and this sulphate test serves only as a guide to the selection of aggregate. In a general sense, most aggregates with high soundness losses tend to have poor durability but there are numerous exceptions. It has been reported that few high soundness loss aggregates had excellent durability and soundness test failed to detect several aggregates of low durability[44]. Thus reliance should always be based on the actual performance from the service-record. Dolar-Mantuani[45] is of the opinion that the test does indicate weaknesses in aggregates. If the specification limits are used carefully together with sound engineering judgement, the test is certainly useful because of the relatively short time needed to perform it. IS : 383-1970[29] as a general guide restricts the average loss of weight after 5 cycles to 12 percent when tested with sodium sulphate and 18 percent when tested with magnesium sulphate.

2.2.2.6 ALKALI-AGGREGATE REACTION — This reaction takes place between the alkalis in the cement and the active siliceous constituents or carbonates of aggregates. Under most conditions, this reaction causes excessive expansion and cracking of concrete. These deleterious reactions have been encountered in many parts of the World and in all climatic zones. The reactions are:

a) Alkali-silica reaction, and
b) Alkali-carbonate reaction.

A typical example of the effects of alkali-silica reaction has been provided by the concrete of a military jetty in Cyprus[46], constructed in 1966. By 1972, widespread cracking and spalling of concrete were noticed and parts of the surface concrete crumbled and became friable, in some places to a depth up to 15 cm. Damage due to alkali-silica reaction had also been noticed in Tuscaloosa Lock and Dam, USA in 1952[47]. Dry dock in south Carolina also showed alkali-silica reaction cracking with quartz gravel aggregate in 1969. The reactive material was metamorphic quartz or metamorphic and highly weathered quartz. In the year 1965–66, Lachswehr Bridge of northern Germany had severe damage due to this reaction. The area of major concern in Germany is in two classes of reactive constituent in aggregates, opaline sandstone and reactive flint.

Certain dolomitic aggregates from Bahrain[48] have been found to react deleteriously with cement paste. The reaction has been noticed to be promoted by the presence of gypsum and excess hydroxyl in the mixture and by the marked porosity of the aggregate.

In 1956-57, expansive reactivity of concrete was noticed at Kingston[49], Ontario. A close look at culverts and bridges constructed only a few months earlier showed pattern of map cracking. Observations indicated that dolomitic limestone aggregate from local quarries was an essential ingredient of the affected concrete. By prior geological exploration of existing and potential quarry sites and subsequently testing the different rocks for alkali — carbonate reactivity, it is possible to select non-reactive aggregates. For constructing a major concrete highway, this procedure appeared to be realistic and economical considering the extra cost of using low alkali cement or bringing aggregates from outside.

Newlon and Sherwood[50] suggested the following measures to reduce expansion of concrete where it is essential to use alkali reactive carbonate aggregates, for economy:

a) If the aggregate is of a high degree of reactivity, dilution of reactive aggregates with a non-reactive one, reduction of cement alkalis or both, are necessary to eliminate cracking and reduce expansion significantly.
b) The limit of 0.60 percent alkali for low-alkali cement does not appear to be low enough to reduce the reaction with the highly reactive carbonate aggregates, even with aggregate dilution of 50 percent. To reduce the reaction to an acceptable level, the reduction of alkalis to 0.40 percent may be necessary. If reduction of alkalis to this limit is not possible, then corresponding greater dilution of aggregate is required.

The reactive forms of silica are opal (amorphous), chalcedony (crypto crystalline fibrous) and tridymite (crystalline). The chemical composition, physical character and the reactive minerals[33] are given in Table 14. The reaction minerals occurring in the reactive rocks are given in Table 15[33]. Rocks containing opal, chalcedony, chert, volcanic glass, crystobalite, tridymite or fused silica have shown to be reactive in many instances. As little as 0.5 percent of a defective aggregate is sufficient to cause considerable damage in concrete. The maximum expansion is produced when reactive aggregates make up about 4 percent of the total aggregates and this disadvantageous concentration is often referred to as the pessimum[46].

The actual reaction occurs between siliceous minerals in aggregates and the alkaline hydroxides derived from the alkalis (Na_2O and K_2O) in the cement. The result of reaction is alkali-silicate gel of 'unlimited swelling' type, and because the gel is confined by the surrounding cement paste, internal pressure causes cracking and disruption.

The carbonate in aggregates is generally argillaceous dolomitic limestone. A wide range of carbonate rocks have been reported as being potentially reactive, ranging from pure limestone to pure dolomite. The presence of clay minerals incorporated in the aggregate and the crystalline texture of the carbonate rocks influence the reaction. The reactive carbonate materials are confined to specific ranges of rock composition between calcitic dolomites and magnesium limestones. Dolomites and limestones containing excess Mg or Ca ions in their crystal structure over the ideal proportions are more likely to be reactive[46]. The dolomitic rock consists of substantial amounts of dolomite and calcite in the carbonate portion, with significant amounts of acid insoluble residue consisting largely of clay. Dolomitization reaction[47] is believed to be the alkali — carbonate reaction producing harmful expansion of concrete. Magnesium hydroxide, brucite [$Mg(OH)_2$] is formed by this reaction.

One method of determining the potential alkali-aggregate reactivity is by 'mortar bar' test as given in IS : 2386 (Part VII)-1963[51]. The method of test covers the determination of reactivity by measuring the expansion developed by the cement — aggregate combination in mortar bars during storage under prescribed conditions of test. The test is more conclusive but has the disadvantage of requiring several months and also requiring that coarse aggregate be crushed rather than tested in its normal state. With larger specimens, however, uncrushed aggregate may be tested[52].

The second method of determining the potential reactivity of aggregates is the 'chemical method' as prescribed by IS : 2386 (Part VII)-1963[51]. This method of test determines the reactivity as indicated by the amount of reaction of the aggregate with a 1 N sodium hydroxide solution under controlled test conditions. The method has the advantage that it can be performed in 3 days, but for many aggregates the results are not conclusive[28]. However, the illustration of division between innocuous and deleterious aggregates (based on Mielenz and Witte's work[11]) is reproduced from IS : 2386 (Part VII) - 1963[51] in Fig. 12.

Both the test methods mentioned above do not always detect the alkali-carbonate reactivity but this may be detected by another test, in which concrete prisms made with questionable aggregates and a high alkali cement are exposed to an environment of 23°C and with 100 percent relative humidity and noting the amount of expansion. Expansion of prisms made of questionable aggregate are then compared with those obtained on companion prisms of known sound limestone[53].

The problem of alkali-aggregate reaction can be overcome by use of low-alkali cement (that is, containing less than 0.6 percent alkali calculated as Na_2O) or by addition of suitable finely ground pozzolana to the concrete mix. The pozzolana reacts chemically with the alkalis before they attack the reactive aggregates[53].

Air entrainment is also believed to be useful in counter acting alkali-aggregate reaction. Use of reactive aggregate itself in finely divided form is also known to inhibit destructive effects of alkali-aggregate reaction.

Detailed petrographic examination and X-ray identification are being used to examine suspect aggregates. But the conclusions are unreliable and all available past evidence must be taken into account when evaluating a new aggregate.

Detailed petrographic, mineralogical and chemical data compared with similar data already available for reactive aggregates may provide the most satisfactory means of identifying potentially reactive aggregates[46].

The problem of alkali-aggregate reaction is not generally encountered with natural aggregates used in this country. Limited data are available on alkali reactivity of natural coarse aggregates in India[54,55]. Gogte[55] evaluated some common Indian aggregates with emphasis on their susceptibility to alkali-aggregate reactions from a study of a number of samples of rock aggregates belonging to different Indian geological formations, more or less representing those used for concrete constructions all over the country, the following conclusions were drawn based on their petrographic characters and mortar-bar expansion tests:

a) Indian rocks vary widely in their susceptibility to alkali-aggregate reactions. Even rock aggregates having identical characteristics were found to differ considerably regarding their behaviour with high-alkali cements.

b) The potential alkali reactivity of crystalline rocks, for example granites, granodiorites, gneisses, charnockites, quartzites and schists, is related to the percentage straining effects in quartz. Rock aggregates containing 40 percent or more strongly undulatory, fractured or highly granulated quartz are highly reactive and with 30-35 percent strained quartz are moderately reactive. Rock aggregates belonging to the above groups containing predominantly unstrained or recrystallised quartz show negligible mortar expansions and thus are innocuous.

c) The basaltic rocks with 5 percent or more chalcedony or opal or about 15 percent palagonite show deleterious reactions with high alkali cements. Sedimentary and metasedimentary rocks, for example, sandstones and quartzites containing 5 percent or more of chert also show deleterious reactivity with cement alkalies. These reactions could be attributed to the micro to crypto-crystalline texture and fibrous extinction of these minerals which is also an indication of the presence of dislocated silica zones.

d) The problem of alkali-aggregate reaction needs to be viewed from new angles. The arbitrary classification of silica minerals into alkali reactive and non-reactive, produces many exceptions. Because of dislocated zones of silica in deformed quartz, some seemingly innocuous aggregates, for example, granites, charnockites and quartzites, which do not contain the well-known alkali-reactive constituents, such as opal, chalcedony or volcanic glass show deleterious chemical reactivity with cement alkalies.

2.2.3 LIGHTWEIGHT AGGREGATES — Lightweight aggregates can be either natural like diatomite, pumice, scoria, volcanic cinders, etc, or manufactured like bloated clay, sintered fly ash or foamed blast furnace slag. Lightweight aggregates are used in structural concrete and masonry blocks for reduction of the self weight of the structure. The other usages of lightweight aggregate are for better thermal insulation and improved fire resistance.

Unlike for normal weight aggregates from natural sources, there is no Indian standard specification for lightweight aggregates. However, IS : 9142-1979[56] covers specification for artificial lightweight aggregates for concrete masonry units, while IS : 2686-1977[57] covers cinder aggregate for use in lime concrete for the manufacture of precast blocks. However, IS : 456-1978[1] envisages the use of bloated clay and sintered fly ash aggregate with proven performance in structural concrete. The use of lightweight aggregates in India is not so common; however they have been widely used in USA and other Western countries.

The main requirement of lightweight aggregates is their low density; some specifications limit the bulk density to 1 200 kg/m^3 for fine aggregates and 960 kg/m^3 for coarse aggregates for use in concrete. Both coarse and fine aggregates may be lightweight. Alternatively, lightweight coarse aggregates can be used with natural sands. The characteristics of lightweight aggregates which require consideration for use in structural concrete are as follows:

a) Some lightweight aggregates may contain closed pores or voids in the material, apart from high water absorption of the order of 8 to 12 percent. The closed pore system inside the aggregate mass will not be accessible to mixing water but they will displace an equal amount of mixing water or paste (equal to volume of pore). The water absorption aggregates will absorb part of the mixing water the moment they come in contact with it in a mixer and that part of the water may not be available for reaction with cement. Because of this, the usual concept for designing normal weight concrete mixes may not be applicable to lightweight aggregates and on larger or important works separate relationships between the relative density, water absorption, moisture content of the aggregates on the one hand and workability, density and compressive strength on the other may have to be established.

b) Being artificially produced by sintering or pelletizing, most of the synthetic aggregates may have a smooth surface and rather regular shapes which may reduce the bond characteristics with the mortar and thereby result in lower compressive strength

c) If, during mixing the lightweight aggregates get crushed, the void structure is broken down resulting in a coarse surface texture which may lower the workability.

d) The modulus of elasticity of concretes made with most of the lightweight aggregates is lower than the normal weight concrete, may be ½ to ¾. Creep and shrinkage of concrete are also greater (will vary from equal to about double) (Fig. 13[58]) compared to those of normal weight concrete, having the same compressive strength[59]. These result in higher deflection of the structural members.

Because of high absorption, workable concrete mixes become stiff within a few minutes of mixing. Therefore, it is necessary to wet (but not saturate) the aggregates before mixing in the mixer. In the mixing operation, the required water and aggregate are usually premixed prior to addition of cement. As a rough guide, the extra water needed for lightweight aggregate concrete is about 6 kg/m³ of concrete to obtain a change in the workability of 25 mm slump. Rich mixes containing cement about 350 kg/m³ or more, are usually required to produce satisfactory strength of concrete. The concrete cover to reinforcement using light weight aggregates in concrete should be adequate. Usually it is 25 mm more than for normal concrete. This increased cover is necessary, because of its increased permeability and also because concrete carbonates rapidly by which the protection to the steel by the alkaline lime is lost.

Deflection calculations considering lower tensile strength and modulus of elasticity of lightweight concrete, generally result in higher initial deflection. Tobin[60] observed 10 percent more deflection for the lightweight slab compared to normal weight concrete slab under the same superimposed loading. Lower tensile strength of lightweight concrete must be taken into account in design for the calculation of allowable cracking stress for prestressed members or for deflection calculation based on a cracked section instead of a homogeneous section[59].

The increased creep of lightweight concrete does not add to the deflection of structural members, as the ratio of creep strain to elastic strain is the same for both lightweight and normal weight concrete. The effect of shrinkage on deflection arises from the restraint of shrinkage due to steel reinforcement. Tests have shown that for usual amount of reinforcement, the effect of shrinkage on deflection is quite small regardless of the type of concrete. Thus the difference between the shrinkage deflection of lightweight and normal weight concrete members of comparable design is quite small[59].

Bloated clay aggregates are spherical in shape, hard, light and porous. The size of the particles ranges from 5 to 20 mm. The fine aggregate is produced by crushing the larger particles. The water absorption of bloated clay aggregates is about 8 to 20 percent. The physical properties of such aggregates seem to vary with the bloating characteristics of the raw material and the processing equipment used in manufacturing the aggregate. The use of this aggregate is advocated in places where the cost of crushed stone aggregate is high and suitable clays especially silts from water works are easily available. Concrete produced from this aggregate has bulk density of the order of 1 900 kg/m³.

The sintered fly ash aggregate is rounded aggregate with a bulk density of about 1 000 kg/m³. This type of aggregate is suitable for making masonry units as well as structural concrete. Concrete prepared from this aggregate has unit weight of 1 200 to 1 400 kg/m³.

Foamed blast furnace slag aggregate is produced with a bulk density varying between 300 and 1 100 kg/m³, depending on the details of the cooling process and to a certain degree on the particle size and grading. Concrete made with this aggregate has a density of 950 to 1 750 kg/m³.

Vermiculite is another artificial lightweight aggregate which when heated to a temperature of 650 to 1 000°C expands to as many as 30 times its original volume by exfoliation of its thin plates. Thus the density of exfoliated vermiculite is only 60 to 130 kg/m³ and the concrete made with it is of low strength and exhibits high shrinkage but is used as an excellent heat insulator.

2.3 Water — Compared to other ingredients of concrete, the quality of water usually receives less attention. However, unwanted situations leading to distress of concrete, contributed among others by the mixing and curing water being not of the appropriate quality has focussed the attention on quality of water as well.

Potable water is generally considered satisfactory for mixing concrete. Should the suitability of water be in doubt, particularly in remote areas or where water is derived from sources not normally utilized for domestic purposes, such water should be tested. The permissible limits for solids and impurities for mixing and curing water as specified in various specifications including IS : 456-1978[1] are in excess of the requirements of potable water. Perhaps, the most comprehensive investigations on water for concrete making were carried out by Abrams; these and a few others are summarized in Ref 61 and the recommendations in the various codes of practice including IS : 456-1978[1] are largely based on these tests. Abrams[61, 62] tested 68 different water samples including sea, alkali, mine, mineral and bog waters and highly polluted sewerage and industrial wastes on mortar and concrete specimens. The effects were expressed mainly in terms of differences in the setting times of Portland cement mixes containing impure mixing waters as compared to clean fresh waters and concrete strengths from 3 days to 2 years and 4 months compared with control specimens prepared with distilled water. A difference in 28 days compressive strength by maximum 15 percent of control test was concluded to be the best measure of the quality of mixing water. IS : 456-1978[1] also incorporates a similar provision, except the requirement of compressive strength is to be not less than 90 percent. It was also concluded that the setting time is not a satisfactory test for measuring suitability of a water for mixing concrete and in most of the cases the setting time was found to be the same. IS : 456-1978[1] prescribes a difference in initial setting time of ± 30 minutes with initial setting time not less than 30 minutes.

Based on the minimum strength ratio of 85 percent, the following waters were found to be suitable for concrete-making: bog and marsh waters; waters with a maximum concentration of 1 percent SO_4; sea water, but not for reinforced concrete; alkali water with a maximum of 0.15 percent Na_2SO_4 or Nacl; pumpage water from coal and gypsum mines; and waste water from slaughter houses, breweries, gas plants, paint and soap factories. The waters found unsuitable for the purpose were acid water, lime soak water from tannery waste, carbonated mineral water discharged from galvanizing plants, water containing over 3 percent of sodium chloride or 3.5 percent of sulphates and water containing sugar or similar compounds. The lowest content of total dissolved solids in these unacceptable waters was over 6 000 ppm except for a highly carbonated mineral water that contained 2 140 ppm of total solids.

From the above, the main reason for holding potable water (the exception being water containing sugar) as suitable for making concrete would be obvious, as very few municipal waters would contain more than 2 000 ppm of dissolved solids and specifications of potable water would exclude nearly all of the above mentioned polluted waters. On the basis of these tests and the work done by others, the limits of some impurities in water for mixing and curing concrete that can be held to be tolerable are listed in Table 16[63]. Compared to these, the permissible limits in IS : 456-1978[1] reproduced in Table 17 are on the conservative side. In addition, there are additional tests to determine the alkalinity (as carbonates and bicarbonates) and the acidity of the water sample in IS : 456-1978[1]. The limit of alkalinity is guided by the requirement that to neutralize 200 ml sample of water, using methyl-orange as an indicator, it should not require more than 10 ml of 0.1 normal HCl. This alkalinity is equivalent to 265, 420 and 685 ppm of carbonates (as Na_2CO_3), bicarbonates (as $NaHCO_3$) and the sum of the two, respectively, and should be seen in the background of Abrams' tests which showed that concentrations up to 2 000 ppm were in general tolerable. Regarding acidity, the *p*H value of the water is required to be generally not less than 6. However, the *p*H value may not be a satisfactory general measure of the amount of acid or basic reaction that might occur and the effect of acidity in water is best gauged on the basis of the total acidity 'as determined by titration'[61]. The limit of total acidity is guided by the requirement that to neutralise 200 ml sample of water, using phenolphthalein as indicator, it should not require more than 2 ml of 0.1 normal NaOH. This acidity is equivalent to 49 ppm of H_2SO_4 or 36 ppm of HCl.

2.3.1 MISCELLANEOUS INORGANIC IMPURITIES — Kuhl[61] made a broad survey of the effects of miscellaneous inorganic salts. The salts chosen constituted a cation series and an anion series. For the cation series, chlorides were used except that where solubility was low nitrates were used. The anion series comprised sodium salts. The salts of the cation series that caused a marked reduction in strength of concrete were those of manganese, tin, zinc, copper, and lead (nitrate). The zinc and copper chlorides retarded so much that no strength tests were possible at 2 and 3 days. The action of lead nitrate was completely destructive.

Out of anion series, sodium iodate, sodium phosphate, sodium arsenate and sodium borate reduced the initial strength of concrete to an extraordinary low degree and in certain instances to zero. Another salt that has been found to have detrimental effect on strength development is sodium-sulphide and even a sulphide content of 100 ppm warrants testing (*see* Table 16).

2.3.2 SILT OR SUSPENDED CLAY PARTICLES — Considerable muddiness or turbidity in mix water can be tolerated if it is simply suspended clay or fine rock particles[61]. IS : 456-1978[1] allows 2 000 mg/l of suspended matter (Table 17). Muddy water should remain in settling basins before use.

Algae — Doell[62,33] has shown that the water containing algae has the effect of entraining huge quantities of air in the concrete thus resulting in lower strength. Algae also reduces the bond between aggregates and cement paste. The presence of algae to the extent of 0.1 percent by mass of mixing water entrained 6 to 7 percent of air causing a strength reduction of more than 15 percent.

2.3.3 OIL CONTAMINATION — Generally, oil contamination can be removed by floatation. Mineral oils not mixed with animal or vegetable oils are probably the least objectionable from the standpoint of strength development[61]. Davis[33] carried out tests to investigate the effect of oil contamination. Three different oils were used, namely, a mineral oil (SAE 30), a diesel fuel oil and a vegetable seed oil (sunflower). The results were summarized as follows: 2 percent by mass of cement of mineral oil resulted in significant increases in strength at all ages. Addition of 4 percent and 8 percent of mineral oil also resulted in strength gain but not of the order of 2 percent. In case of diesel oil 8 percent of oil slightly reduced the strength, whereas at lower percentages strength gains were observed. In case of sunflower oil 8 percent of oil had a detrimental effect to the strength of concrete particularly at later ages.

2.3.4 SEA WATER — Sea water generally contains 3.5 percent of dissolved solids, about 78 percent (that is, 27 000 ppm) of which is sodium chloride and 15 percent (that is, 5 300 ppm) of which is chloride and sulphate of magnesium. Opinions differ when it comes to categorically classifying sea water as either allowable or not for use for making concrete. In so far as the requirements of strength are concerned, the early age strength (up to 3 days or so) may indeed be somewhat higher, possibly because of the accelerating effects of the chlorides present; but long term strengths may be somewhat lower. However, the major concern is attributed to the risk of corrosion of reinforcing steel due to the chloride as well as other problems of durability of concrete associated with sulphates. In general, the risk of corrosion of steel is more when the reinforced concrete member is exposed to air than when continuously submerged under water, including sea water[64]. Based on this the consensus is not to permit the use of sea water for making concrete in reinforced concrete constructions[61, 25]. Under unavoidable circumstances, it may be used for plain concrete[61, 65, 66] when it is constantly submerged in water. IS : 456-1978[1] incorporates a similar provision but the use of sea water for prestressed concrete is not permitted in any case.

2.3.5 CURING WATER — IS : 456-1978[1] states that water found satisfactory for mixing concrete can also be used for curing concrete but it should not produce any objectionable stain or unsighty deposit on the surface. Iron and organic matter in the water are chiefly responsible for staining or discolouration and especially when concrete is subjected to prolonged wetting, even a very low concentration of these can cause staining. According to IS : 456-1978[1], the presence of tannic acid or iron compounds in curing water is objectionable.

2.4 Admixtures — Present day concrete often incorporates a fourth ingredient called admixtures, in addition to cement, aggregates and water. Admixtures are added to the concrete mix immediately before or during mixing, to modify one or more of the specific properties of concrete in the fresh or hardened states. IS : 9103-1979[67] lays down the procedures for evaluation of admixtures for concrete and the changes in the properties of concrete that should be expected when the admixtures are used.

The different types of admixtures covered by the Indian Standard (IS : 9103-1979[67]) are as follows:

a) Accelerating admixtures,

b) Retarding admixtures,

c) Water-reducing admixtures, and

d) Air-entraining admixtures.

In addition, IS : 456-1978[1] permits the use of pozzolana like fly ash conforming to IS : 3812 (Part II)-1981[68] or burnt-clay conforming to IS : 1344-1982[69] as admixtures for concrete. They are used mainly to improve the workability of concrete and thereby reduce the water demand for a given workability. However, because of pozzolanic reactions taking place, the role of such pozzolana as admixture cannot be strictly delineated from that of part replacement of cement. Moreover, these finely divided mineral additives are usually added in much larger dosage than chemical admixtures listed above.

Before using an admixture in concrete, the performance of it should be evaluated by comparing the properties of concrete with the admixture and concrete without any admixture. Though the admixtures covered in IS : 9103-1979[67] are intended mainly for modifying a single property in concrete, some of the admixtures available in the market are often capable of modifying more than one property of concrete. For example, water-reducing admixtures can also be set-retarders and air-entraining admixtures increase the workability of the concrete mix, in addition to providing air-entrainment.

In addition, an admixture can improve the desirable properties of concrete in more than one way. For example, water-reducing admixtures can be used: (a) to increase the workability of concrete with the same water and cement contents, (b) to increase the compressive strength of concrete without changing the workability by reduction of the water content in the concrete mix, when the cement content is unaltered, or (c) to effect saving in cement content by reduction in both the cement and water contents in the mix, while maintaining the same workability and compressive strength as in the reference concrete.

Although the standard specifications would define the minimum performance requirements of an admixture, in practice, the resultant improvements in the characteristics of concrete may be required to be more than the minimum specified in these specifications. For example, a viable water-reducing admixture should allow a reduction in the water content between 5 to 12 percent and consequently cause an increase in the 28-day compressive strength of concrete by nearly 10 percent[70]. On the other hand, by modifying the concrete mix design in conjunction with the use of a water-reducing admixture, the economy in the use of cement can be of the order of 10 percent[71].

Concretes made with admixtures when compared with identical concrete made without admixtures should manifest improved physical properties as given in Table 18[67]. In case of air-entraining admixtures, a reference admixture of approved quality should be used in the controlled concrete to entrain identical amount of air.

The performance of an air-entraining admixture can be evaluated in accordance with ASTM C 233-76[72]. As stipulated in IS : 9103-1979[67], the relative durability factor of concrete containing admixture under test should not be less than 80 for specified number of 300 freeze-thaw cycles, the durability factor being related to the relative dynamic modulus of elasticity of standard prism specimens.

Some admixtures are likely to contain water soluble chlorides and sulphates. These, if present in large quantities, may cause damage to the concrete structures during the course of time. The chlorides may cause corrosion to the steel reinforcement whereas the sulphates may cause disintegration of concrete by forming sulphoaluminates. IS : 9103-1979[67] for admixtures for concrete, stipulates that the chloride content of the admixtures should be declared by the manufacturers.

2.4.1 ACCELERATING ADMIXTURES — These are substances which when added to concrete increase the rate of hydration of cement, shorten the setting time and increase the rate of strength development. The chemicals that accelerate the hardening of concrete mixes include soluble chlorides, carbonates, silicates, fluorosilicates and hydroxides and also some organic compounds, for example, triethanolamine.

The most widely known accelerator is calcium chloride. The addition of $CaCl_2$ to the concrete mix increases the rate of development of strength. Accelerating admixtures are used when concrete is to be placed at low temperatures. Indian Standard recommendations IS : 7861 (Part II)-1981[73] for cold weather concreting envisages the use of $CaCl_2$ up to a maximum of 1.5 percent by mass of cement for plain and reinforced concrete works, in cold weather conditions. $CaCl_2$, however, is not permitted to be used in prestressed concrete because of its potential danger in augmenting stress corrosion.

The increase in the rate of development of early strength of concretes containing ordinary and rapid hardening Portland cement is shown in Fig. 14[33]. At normal temperatures, addition of 2 percent $CaCl_2$: (a) accelerates the rate of strength of concrete containing ordinary Portland cement at early ages approximating that of a rapid hardening Portland cement, and (b) increases the strength and abrasion resistance of concrete at all ages up to 1 year[33]. However, addition of calcium chloride admixture may bring about reduction in flexural strength of concrete at ages of 28 days and beyond.

2.4.2 RETARDING ADMIXTURES — A delay in the setting of concrete is achieved by the use of retarding admixtures. Retarding action is exhibited by sugar, carbohydrate derivatives, soluble zinc salts, soluble borates, etc. Retarding admixtures are used in hot weather when normal setting time of cement gets reduced due to high temperature. IS : 7861 (Part I)-1975[74] envisages the use of such admixtures to offset the accelerating effects of high temperature. The retarding action of a set-retarder to the penetration resistance of concrete is shown in Fig. 15[75].

A small quantity of sugar (about 0.05 percent by mass of cement) delays the setting time of concrete by about 4 hours[76]. However, the effect of sugar depends greatly on the chemical composition of cement. The performance of such a retarder should be determined by trial mixes with the actual cement to be used in construction. A large quantity of sugar (say 0.2 to 1 percent by mass of cement) prevents the setting of cement[2].

2.4.3 WATER-REDUCING ADMIXTURES — The water-reducing admixtures are usually based on calcium or sodium salts or derivatives of lignosulphonic acids from the wood pulping industry. Modern admixtures based on lignosulphonic acid derivatives are formulated from the purified products, after removing the sugar and other impurities. Polycarboxylic acids, their salts, modifications and derivatives also find some application as water-reducing admixtures[71].

The increase in the workability of concrete (in terms of compacting factor) using a water-reducing admixture is shown in Fig. 16[77]. The determination of workability is an important factor in testing concrete admixtures. Rapid loss of workability occurs during the first few minutes after mixing of concrete and gradual loss of workability takes place over a period from 15 to 60 minutes after mixing[77]. Thus the relative advantage of water-reducing admixtures decreases with time after mixing. Therefore, IS : 9103-1979[67] stipulates that the workability of fresh concrete containing admixtures should be determined not sooner than 15 minutes nor later than 20 minutes after completion of mixing of concrete.

Water-reducing set-retarders belong to the following two main groups:

a) Lignosulphonic acids and their salts, and

b) Hydroxylated carboxylic acids and their salts.

These admixtures increase the setting time by about 2 to 6 hours during which concrete can be vibrated, revibrated and finished. This is particularly important in hot weather conditions or where the nature of construction demands a time gap between the placing of successive layers of concrete. It is possible to offset the set-retarding property of this kind of admixture, if the situation so demands, by incorporating an accelerator, for example, $CaCl_2$ or triethanolamine[33].

2.4.4 AIR-ENTRAINING ADMIXTURES — These admixtures cause air to be incorporated in the form of minute bubbles in the concrete during mixing, usually to increase workability and resistance to freezing and thawing. They control the amount of air in fresh concrete and disperse properly sized air bubbles throughout the concrete. The origins of air-entraining admixtures are as follows[33]:

a) Natural wood resins;

b) Animal or vegetable fats and oils;

c) Various wetting agents, such as alkali salts of sulphated and sulphonated organic compounds;

d) Water soluble soaps of resin acids and animal or vegetable fatty acids; and

e) Miscellaneous materials, such as sodium salts of petroleum sulphonic acids, hydrogen peroxide, aluminium powder, etc.

The resistance of air-entrained concrete in terms of dynamic modulus of elasticity and change in length, against freezing and thawing is shown in Fig. 17[78].

The entrained air bubbles (approximately 0.05 to 1.25 mm dia) reduce the capillary forces (the force causing absorption of water by concrete) by restricting the effective length of each capillary pore in concrete. The capillaries are interrupted by relatively large air voids in air-entrained concrete. The voids cannot fill with water from the capillaries because of surface tension effects and, therefore, under freezing conditions, they behave as 'expansion chambers' to accommodate the ice formed. When the ice melts, surface tension effects draw the water back into the capillary so that the air bubble acts as a permanent safety valve, giving continuous protection to concrete against frost damage[79]. The resistance of air-entrained concrete is thus attributed to a combination of reduced permeability of cement paste, the breakdown of capillary action and the relief of pressure in the concrete pores under the conditions of freezing and thawing.

Entrainment of small amount of air results in concrete of insufficient durability, whereas with large amount of air-entrainment there is an excessive strength reduction in concrete. Therefore, an optimum percentage of air giving a balance between compressive strength and durability must be used in practice. Table 19[80] gives optimum air contents for concretes of different maximum sizes of aggregate.

2.4.5 INFORMATION ON ADMIXTURES — To facilitate approval of an admixture, the following information is needed:

a) The trade name of the admixture, its source, and the manufacturer's recommended method of use;

b) Typical dosage rates and possible detrimental effects of under and over dosage;

c) Whether compounds likely to cause corrosion of reinforcement or deterioration of concrete (such as those containing chloride in any form as an active ingredient) are present and if so, the chloride ions by mass or expressed as equivalent anhydrous calcium chloride by mass of admixture; and

d) The average expected air content of freshly mixed concrete containing an admixture which causes air to be entrained when used at the manufacturer's recommended rate of dosage.

TABLE 2 COMPOUND COMPOSITION OF ORDINARY PORTLAND CEMENTS

(*Clause* 2.1)

COMPOUND	FORMULA	ABBREVIATION	PERCENTAGE BY MASS IN CEMENT
Tricalcium silicate	$3\ CaO.\ SiO_2$	C_3S	30-50
Dicalcium silicate	$2\ CaO.\ SiO_2$	C_2S	20-45
Tricalcium aluminate	$3\ CaO.\ Al_2O_3$	C_3A	8-12
Tetracalcium aluminoferrite	$4\ CaO.\ Al_2O_3.\ Fe_2O_3$	C_4AF	6-10

TABLE 3 HEAT EVOLUTION OF DIFFERENT COMPOUNDS OF PORTLAND CEMENT (AT 21°C)

(*Clause* 2.1)

Compound	Heat Evolution (cal/g)					
	3 Days	7 Days	28 Days	90 Days	1 Year	6½ Years
C_3S	58 ± 8	53 ± 11	90 ± 7	104 ± 5	117 ± 7	117 ± 7
C_2S	12 ± 5	10 ± 7	25 ± 4	42 ± 3	54 ± 4	53 ± 5
C_3A	212 ± 28	372 ± 39	329 ± 23	311 ± 17	279 ± 23	328 ± 25
C_4AF	69 ± 27	118 ± 37	118 ± 22	98 ± 16	90 ± 22	111 ± 24

NOTE — Table 3 is from 'The Chemistry of Cement and Concrete (Third Edition 1970)' by F. M. Lee and Published by M/s Edward Arnold Ltd, London.

TABLE 4 PHYSICAL AND CHEMICAL REQUIREMENTS OF INDIAN STANDARD SPECIFICATIONS FOR DIFFERENT CEMENTS

(*Clause* 2.1)

Characteristic	Ordinary Portland Cement (IS : 269-1976)	Rapid Hardening Portland Cement (IS : 8041-1976)	Low Heat Portland Cement (IS : 269-1976)	High Strength Portland Cement (IS : 8112-1976)	Portland Pozzolana Cement (IS : 1484-1976)	Portland Slag Cement (IS : 455-1976)
Physical Requirements						
Fineness:						
Specific surface (cm^2/g), *Min*	2 250	3 250	3 200	3 500	3 000	2 250
Setting time, vicat:						
Initial setting time (minutes), *Min*	30	30	60	30	30	30
Final setting time (hours), *Max*	10	10	10	10	10	10
Soundness:						
Le-Chatelier method, expansion (mm), *Max*	10[a], 5[b]	10[a], 5[b]	10[a], 5[b]	10[a], 5[b]	10[a], 5[b]	10[a], 5[b]
Autoclave expansion*, percent, *Max*	0.8	0.8	0.8	0.8	0.8	0.8
Heat of hydration (cal/g), *Max:*						
7 days	—	—	65	—	—	—
28 days	—	—	75	—	—	—
Compressive strength (kgf/cm^2), *Min:*						
1 day	—	160	—	—	—	—
3 days	160	275	100	230	—	160
7 days	220	—	160	330	220	220
28 days	—	—	350	430	310	—
Drying shrinkage (percent), *Max*	—	—	—	—	0.15	—

(Continued)

TABLE 4 PHYSICAL AND CHEMICAL REQUIREMENTS OF INDIAN STANDARD SPECIFICATIONS FOR DIFFERENT CEMENTS — *Contd*

Characteristic	Ordinary Portland Cement (IS : 269-1976)	Rapid Hardening Portland Cement (IS : 8041-1976)	Low Heat Portland Cement (IS : 269-1976)	High Strength Portland Cement (IS : 8112-1976)	Portland Pozzolana Cement (IS : 1484-1976)	Portland Slag Cement (IS : 455-1976)
Chemical Requirements						
Maximum percentage of M (Magnesia)	6.0	6.0	6.0	6.0	6.0	8.0
$\bar{S}$ (Total sulphur content calculated as sulphuric anhydride, SO_3)	2.75, 3.0[c]	2.75, 3.0[c]	2.75, 3.0[c]	2.75, 3.0[c]	2.75, 3.0[c]	3.0
Insoluble residue	2.0	2.0	2.0	2.0	$x^d + \frac{2.0(100-x)}{100}$	2.5
Loss on ignition	5.0	5.0	5.0	4.0	5.0	4.0
Permitted additives (other than gypsum)	1.0[e]	1.0[e]	1.0[e]	1.0[e]	1.0[e]	1.0[e]
Sulphide sulphur	—	—	—	—	—	1.5
Content of slag, percent	—	—	—	—	—	25-65
Content of pozzolana, percent	—	—	—	—	10-25	—
Lime saturation factor[f]	0.66 to 1.02	0.66 to 1.02	—	0.66 to 1.02	—	—
Ratio of percent of alumina to that of iron oxide	≥0.66	≥0.66	≥0.66	≥0.66	—	—

a — Unaerated, b — Aerated (required when sample fails in 'a'), c — When $C_3A > 7$ percent,
d — Where x is the declared percentage of pozzolana,
e — Air-entraining or other agents which have proved not to be harmful,
f — Lime saturation factor $= \frac{CaO - 0.7SO_3}{2.8\ SiO_2 + 1.2\ Al_2O_3 + 0.65\ Fe_2O_3}$, and
x — Declared percentage of pozzolana in the given Portland pozzolana cement.

*The test is to be performed if MgO > 3 percent.

TABLE 5 PROPERTIES OF CONCRETE INFLUENCED BY AGGREGATE PROPERTIES

(*Clause* 2.2.1.2)

CONCRETE PROPERTY	RELEVANT AGGREGATE PROPERTY
Strength and workability	Strength Surface texture Particle shape, flakiness and elongation indices Maximum size, grading, deleterious constituents
Shrinkage and creep	Modulus of elasticity Particle shape Grading Cleanliness Maximum size Presence of clay
Durability	
Resistance to wetting and drying	Pore structure Modulus of elasticity
Resistance to heating and cooling	Coefficient of thermal expansion
Abrasion resistance	Hardness
Alkali-aggregate reaction	Presence of particular silicious constituents
Resistance to freezing and thawing	Soundness Porosity Pore structure Permeability Degree of saturation Tensile strength Texture and structure Presence of clay
Co-efficient of thermal expansion	Co-efficient of thermal expansion Modulus of elasticity
Thermal conductivity	Thermal conductivity
Specific heat	Specific heat
Unit weight	Specific gravity Particle shape Grading Maximum size
Modulus of elasticity	Modulus of elasticity Poisson's ratio
Slipperiness	Tendency to polish
Economy	Particle shape Grading Maximum size Amount of processing required Availability

NOTE — Table 5 is from 'Selection and Use of Aggregates for Concrete' Reported by ACI Committee 621 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

TABLE 6 LIST OF ROCKS PLACED UNDER THE APPROPRIATE GROUPS

(*Clause* 2.2.1.2)

IGNEOUS ROCKS		SEDIMENTARY ROCKS	
Granite Group		*Sandstone Group*	
Granite	Granodiorite	Sandstone	Arkose
Granophyre	Diorite	Quartzitic (silicified)	Graywacke
	Syenite	sandstone	Grit
Gabbro Group		*Limestone Group*	
Gabbro	Peridotite	Limestone	Dolomite
Norite	Pyroxenite		
Anorthosite	Epidiorite	METAMORPHIC ROCKS	
Aplite Group		*Quartzite Group*	
Aplite	Quartz reef	Recrystallized quartzite	
Porphyry		Gondite	
Dolerite Group		*Granulite and Gneiss Groups*	
Dolerite	Lamprophyre	Granite gneiss	Amphibolite
		Composite gneiss	Granulite
Rhyolite Group			
Rhyolite	Felsite	*Schist Group*	
Trachyte	Pumicite	Slate	Phyllite
			Schist
Basalt Group			
Andesite	Basalt	*Marble Group*	
		Marble	Crystalline Limestone

TABLE 7 EXPECTED RELATIONSHIP BETWEEN VARIOUS TESTS FOR DIFFERENT ROCK GROUPS

(*Clause* 2.2.2.1)

Rock Group	Crushing Strength MN/m^2	Aggregate Crushing Value	Abrasion Value	Impact Value	Attrition Value	
					Dry	Wet
Basalt	200	12	17.6	16	3.3	5.5
Flint	205	17	19.2	17	3.1	2.5
Gabbro	195	—	18.7	19	2.5	3.2
Granite	185	20	18.7	13	2.9	3.2
Gritstone	220	12	18.1	15	3.0	5.3
Hornfels	340	11	18.8	17	2.7	3.8
Limestone	165	24	16.5	9	4.3	7.8
Porphyry	230	12	19.0	20	2.6	2.6
Quartzite	330	16	18.9	16	2.5	3.0
Schist	245	—	18.7	13	3.7	4.3

NOTE — Table 7 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by M/s Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

TABLE 8 PARTICLE SHAPE OF AGGREGATES

(*Clause* 2.2.2.2)

CLASSIFICATION	DESCRIPTION	EXAMPLES
Rounded	Fully water worn or completely shaped by attrition	River or seashore gravels; desert, seashore and wind blown sands
Irregular or partly rounded	Naturally irregular, or partly shaped by attrition and having rounded edges	Pit sands and gravels; land or dug flints; cuboid rock
Angular	Possessing well defined edges formed at the intersection of roughly planer faces	Crushed rocks of all types; talus; screes
Flaky	Material, usually angular, of which the thickness is small relative to the width and/or length	Laminated rocks

TABLE 9 SURFACE CHARACTERISTICS OF AGGREGATES

(*Clause* 2.2.2.2)

GROUP	SURFACE TEXTURE	EXAMPLE
1	Glassy	Black flint
2	Smooth	Chert; slate; marble; some rhyolite
3	Granular	Sandstone; oolites
4	Crystalline	*Fine* Basalt; trachyte; keratophyre *Medium* Dolerite; granophyre; granulite; microgranite; some limestones; many dolomites *Coarse* Gabbro; gneiss; granite; granodiorite; syenite
5	Honeycombed and porous	Scoriae; pumice; trass

NOTE — With certain materials, it may be necessary to use a combined description with more than one group number for an adequate description of the surface texture, for example, crushed gravel of Groups 1 and 2, and oolites of Groups 3 and 5.

TABLE 10 POROSITY OF SOME COMMON ROCKS

(*Clause* 2.2.2.3)

ROCK GROUP	POROSITY, PERCENT
Grit stone	0.0-48.0
Quartzite	1.9-15.1
Limestone	0.0-37.6
Granite	0.4- 3.8

NOTE — Table 10 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by M/s Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

TABLE 11 TYPICAL RANGES OF VALUES OF ABSORPTION OF DIFFERENT TYPES OF ROCKS

(*Clause* 2.2.2.3)

ROCK TYPE	ABSORPTION, PERCENT
Basalt	0.1-0.3
Diabase and dolerite	0.1-0.7
Diorite	0.2-0.4
Gneiss	0.1-0.6
Granite	0.2-0.5
Limestone	0.2-0.6
Marble	0.2-0.6
Porphyry	0.2-0.7
Quartzite	0.2-0.5
Sandstone	0.2-9.0
Slate	0.2-0.4
Travertine	2.0-5.0

NOTE — Table 11 is from 'Concrete Technology (Fifth Edition, 1977)' by F. S. Fulton and published by the Portland Cement Institute, Johannesburg.

TABLE 12 LIMITS OF DELETERIOUS MATERIALS IN AGGREGATES (PERCENTAGE)

(*Clause* 2.2.2.4)

Deleterious Substance	Fine Aggregates		Coarse Aggregates	
	Uncrushed	Crushed	Uncrushed	Crushed
Coal and Lignite	1.00	1.00	1.00	1.00
Clay lumps	1.00	1.00	1.00	1.00
Soft fragments	—	—	3.00	—
Material passing 75-micron IS Sieve	3.00	15.00	3.00	3.00
Shale	1.00	—	—	—

TABLE 13 PROPERTIES OF SANDS IN THEIR NATURAL CONDITIONS

(*Clause* 2.2.2.4)

Sl No.	Samples From	Mica Content (Percent)	Fineness Modulus	Types of Mica Particles
1)	*Jamuna River*	11.00		Mainly biotite with a small proportion of muscovite particles
	Sand		1.00	
	Mica		0.99	
2)	*Kosi River*	10.00		Mainly biotite with a small proportion of muscovite particles
	Sand		1.52	
	Mica		1.66	
3)	*Ganga River*	4.60		Mainly muscovite with a small proportion of biotite
	Sand		0.69	
	Mica		0.58	
4)	*Bisnumati River*	10–12		—
	Sand		2.81	
	Mica		—	

TABLE 14 REACTIVE MINERALS

(*Clause* 2.2.2.6)

Reactive Minerals	Chemical Composition	Physical Character
Opal	SiO_2 n H_2O	Amorphous
Chalcedony	SiO_2	Cryptocry stalline fibrous
Tridymite, Phyllite, Zeolite, Heulandite, Ptilotite	SiO_2 (Hydrous silicates)	Crystalline

NOTE — Opal is deliterious in amounts exceeding 0.25 percent by mass of aggregate. Chalcedony is deleterious in amount exceeding 5 percent by mass of aggregate. Chert is composed as a rule of chalcedony, opal and crypto-crystalline quartz.

TABLE 15 REACTIVE ROCKS

(*Clauses* 2.2.2.6 *and* 2.2.3)

Reactive Rocks	Reactive Components
Siliceous Rocks	
Opaline cherts	Opal
Chalcedonic cherts	Chalcedony
Siliceous limestones	Chalcedony and/or opal
Volcanic Rocks	
Rhyolites and rhyolite tuffs Decite and decite tuffs Andesite and andesite tuffs	Volcanic glass, devitrified glass and tridymite (deleterious in excess of 3 percent by mass of aggregate)
Metamorphic Rocks	
Phyllites	Hydromica
Miscellaneous	
Any rocks containing veins, inclusions or grains of the reactive rocks or minerals listed above	—

NOTE — Table 15 is from 'Concrete Technology (Fifth Edition, 1977)', by F. S. Fulton, and published by The Portland Cement Institute, Johannesburg.

TABLE 16 CONCENTRATION OF SOME IMPURITIES IN MIXING WATER WHICH CAN BE CONSIDERED AS TOLERABLE

(*Clause* 2.3)

SL No.	IMPURITY	MAXIMUM TOLERABLE CONCENTRATION
1)	Sodium and potassium carbonates and bicarbonates	1 000 ppm (total). (If this is exceeded, tests for setting time and 28 days strength should be made)
2)	Sodium chloride	20 000 ppm
3)	Sodium sulphate	10 000 ppm
4)	Calcium and magnesium bicarbonates	400 ppm of bicarbonate ion
5)	Calcium chloride	2 percent by weight of cement in plain concrete
6)	Iron salts	40 000 ppm
7)	Sodium iodate, phosphate, arsenate and borate	500 ppm
8)	Sodium sulphide	Even 100 ppm warrants testing
9)	Hydrochloric and sulphuric and other common inorganic acid	10 000 ppm
10)	Sodium hydroxide	0.5 percent by weight of cement if set not affected
11)	Silt and suspended particles	2 000 ppm

TABLE 17 PERMISSIBLE LIMIT FOR SOLIDS

(*Clause* 2.3)

SOLIDS	PERMISSIBLE LIMIT, *Max* mg/l
Organic	200
Inorganic	3 000
Sulphates (as SO_4)	500
Chlorides (as Cl)	2 000 for plain concrete work 1 000 for reinforced concrete work
Suspended matter	2 000

TABLE 18 PHYSICAL REQUIREMENTS FOR CONCRETE ADMIXTURES

(*Clause* 2.4)

REQUIREMENTS	ACCELERATING ADMIXTURE	RETARDING ADMIXTURE	WATER REDUCING ADMIXTURE	AIR-ENTRAINING ADMIXTURE
(1)	(2)	(3)	(4)	(5)
Water content, *Max*, percent of control sample	—	—	95	—
Time of setting, allowable deviation from control sample, hours:				
Initial				
Max	−3	+3	±1	—
Min	−1	+1		
Final				
Max	−2	+3	±1	—
Min	−1			

(*Continued*)

TABLE 18 PHYSICAL REQUIREMENTS FOR CONCRETE ADMIXTURES — *Contd*

Requirements	Accelerating Admixture	Retarding Admixture	Water Reducing Admixture	Air-entraining Admixture
(1)	(2)	(3)	(4)	(5)
Compressive strength, percent of control samples, *Min*				
3 days	125	90	110	90
7 days	100	90	110	90
28 days	100	90	110	90
6 months	90	90	100	90
1 year	90	90	100	90
Flexural strength, percent of control samples, *Min*				
3 days	110	90	100	90
7 days	100	90	100	90
28 days	90	90	100	90
Length change, percent increase over control samples, *Max*				
28 days	0.010	0.010	0.010	0.010
6 months	0.010	0.010	0.010	0.010
1 year	0.010	0.010	0.010	0.010
Bleeding, percent increase over control samples, *Max*	5	5	5	2

TABLE 19 OPTIMUM AIR CONTENTS OF CONCRETES OF DIFFERENT MAXIMUM SIZES OF AGGREGATE

(*Clause* 2.4.4)

Maximum Size of Aggregate (mm)	Optimum Total Air Content (Percent)	Approximate Amount of Air Naturally Entrapped (Percent)
10	8.0	3.0
12.5	7.0	2.5
20	6.0	2.0
25	5.0	1.5
40	4.5	1.0
50	4.0	0.5
70	3.5	0.3
150	3.0	0.2

Note — Table 19 is from 'Recommended Practice for Selecting Proportions for Normal Weight Concrete' Reported by ACI Committee 211 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

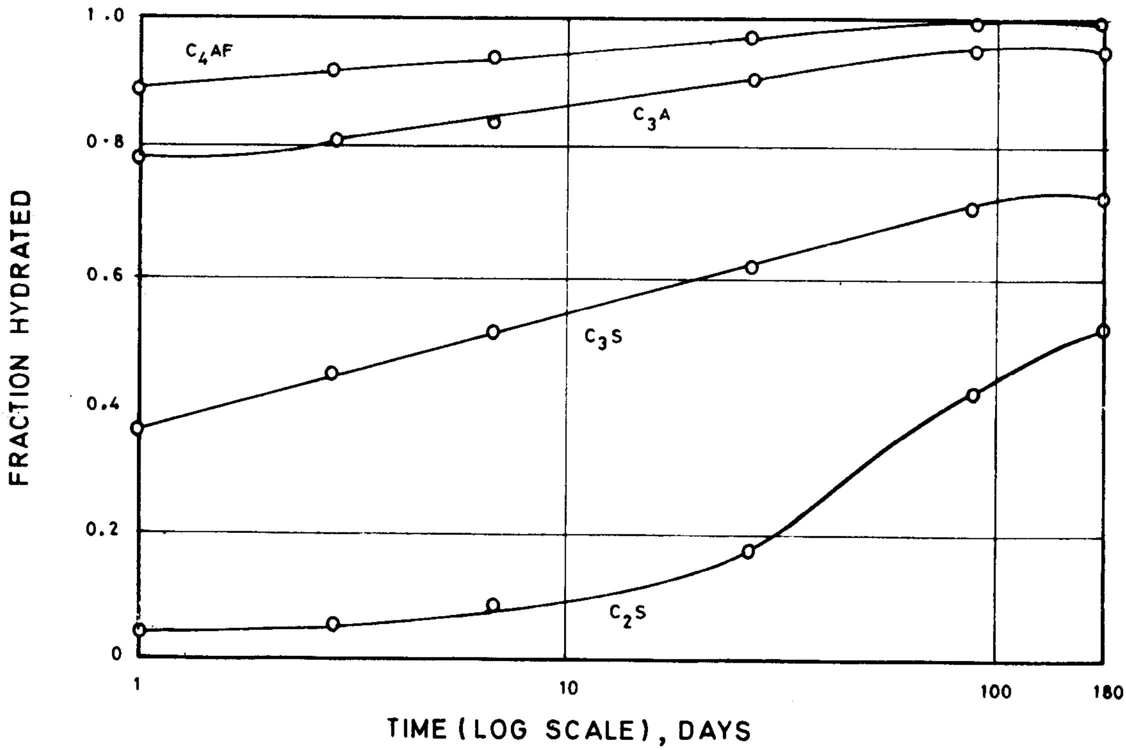


Fig. 1 Rate of Hydration of Pure Compounds

NOTE — Figure 1 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by M/s Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

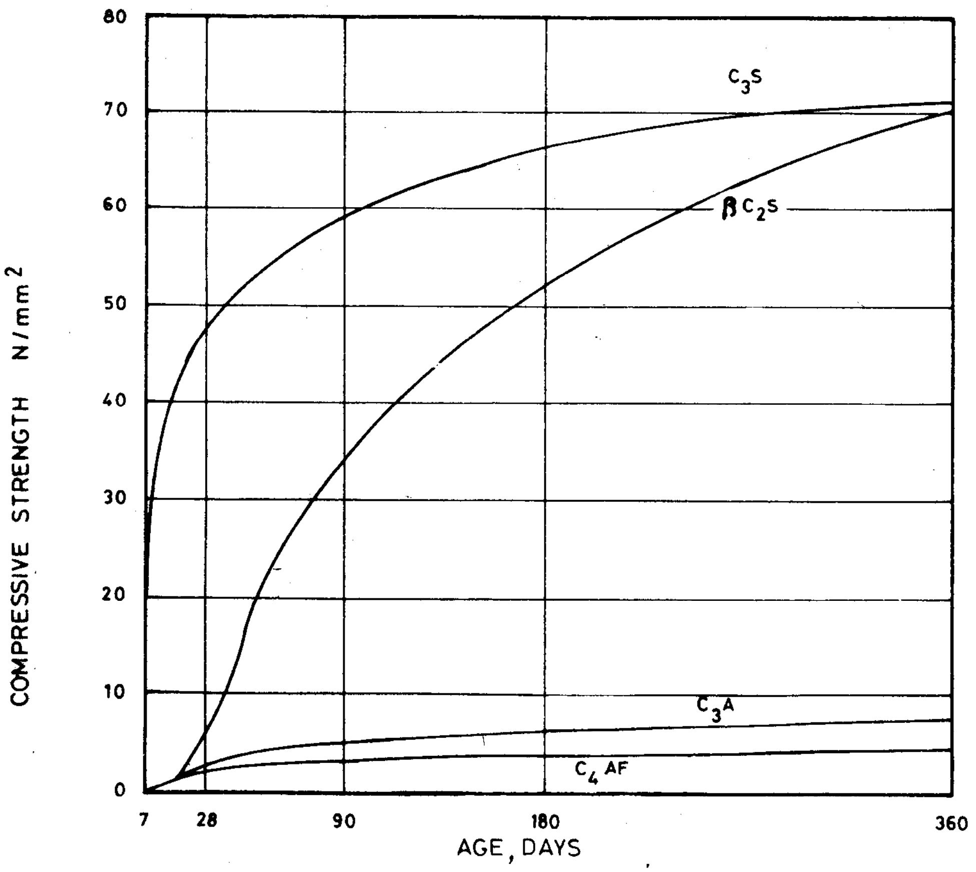


Fig. 2 Development of Strength of Pure Compounds

NOTE — Figure 2 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and the publisher M/s Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

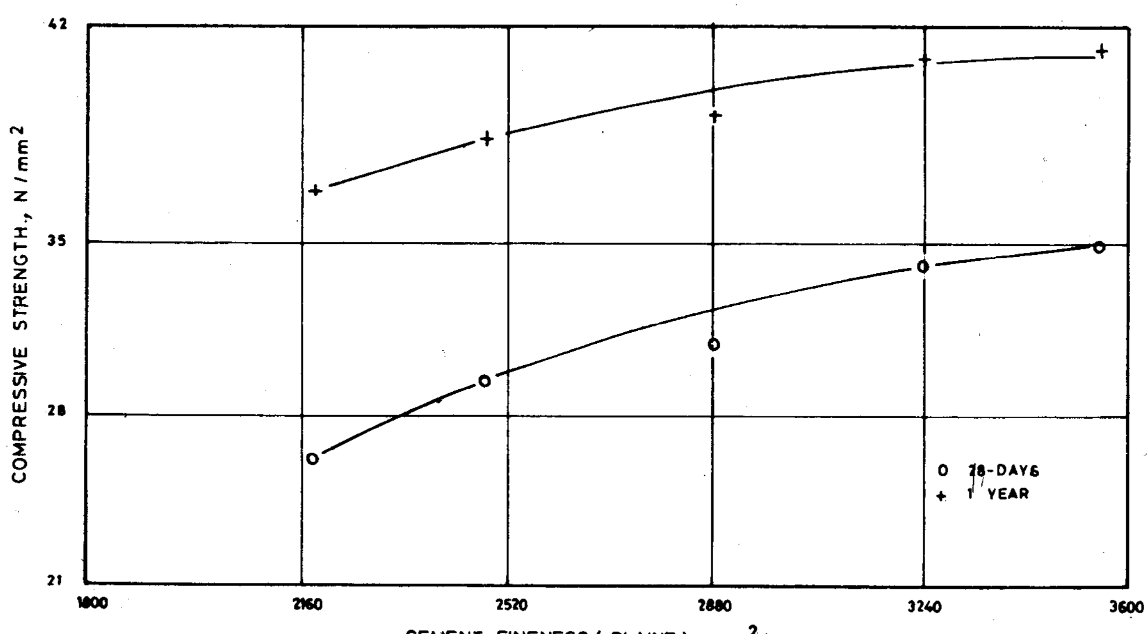


Fig. 3 Relationship Between Strength of Concrete at Different Ages and Fineness of Cement

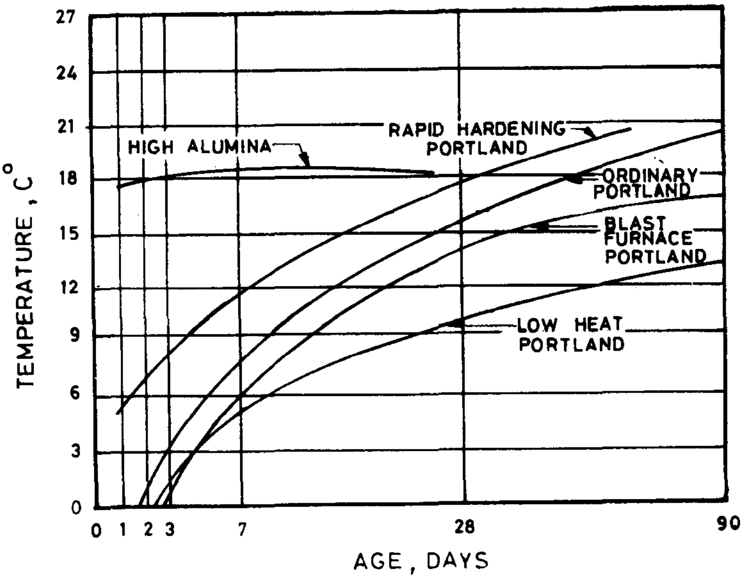


Fig. 4 Calculated Temperature Rise of Different Cements Under Adiabatic Conditions

NOTE — Figure 4 is from 'Concrete Technology (Vol 1, Fourth Edition, 1979)' by D. F. Orchard and published by Applied Science Publishers Ltd, London.

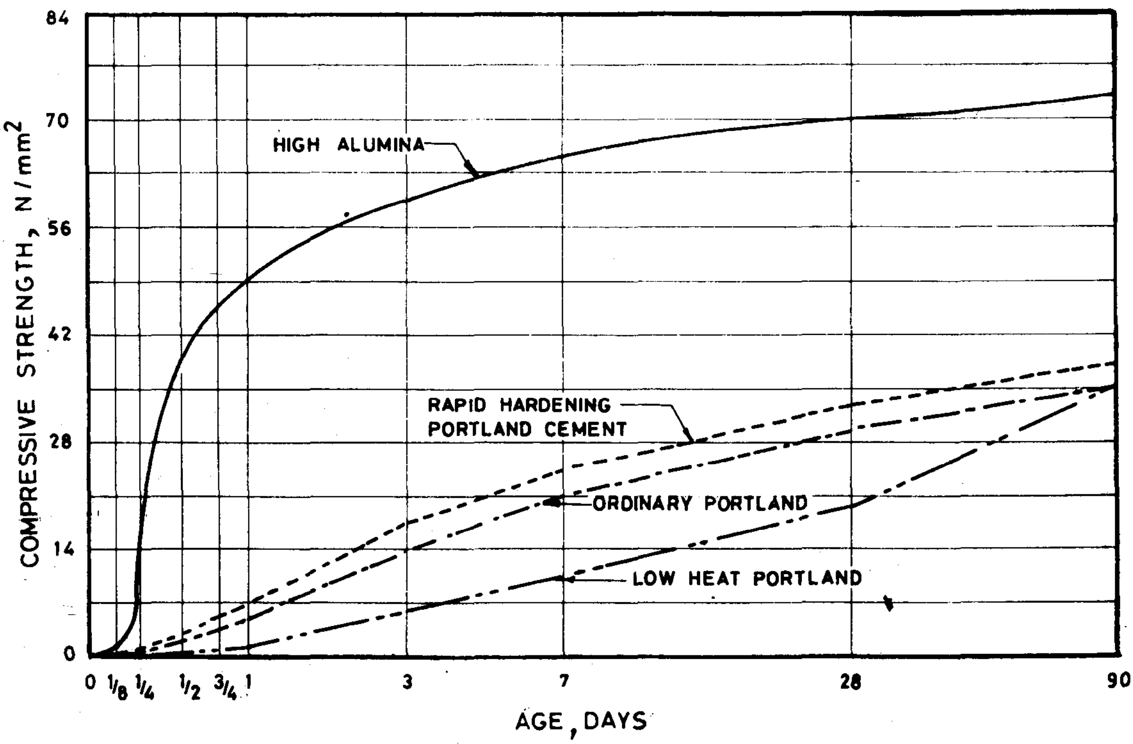


Fig. 5 Strength-Age Relationships for 1:2:4 Concrete by Weight Made with Different Cements

NOTE — Figure 5 is from 'Concrete Technology (Vol 1, Fourth Edition, 1979)' by D. F. Orchard and published by Applied Science Publishers Ltd, London.

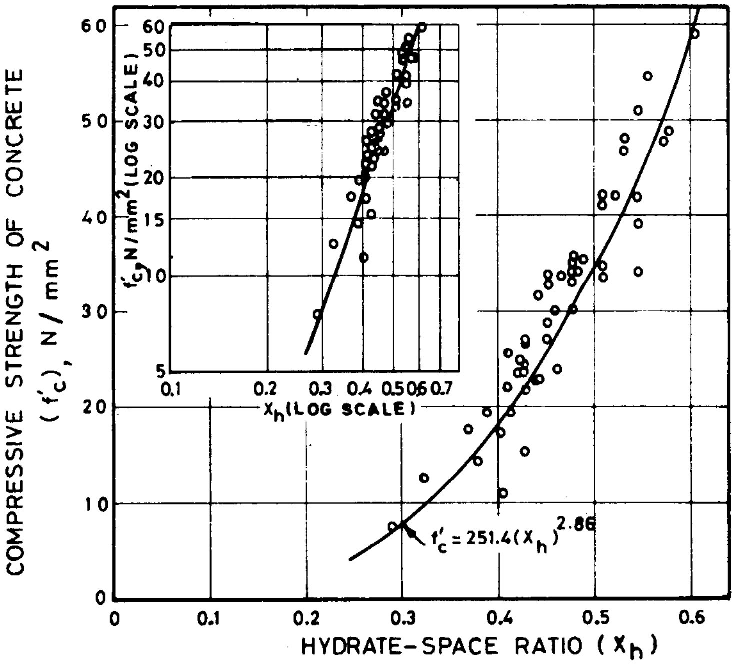


Fig. 6 Relationship Between Compressive Strength and Hydrate-Space Ratio

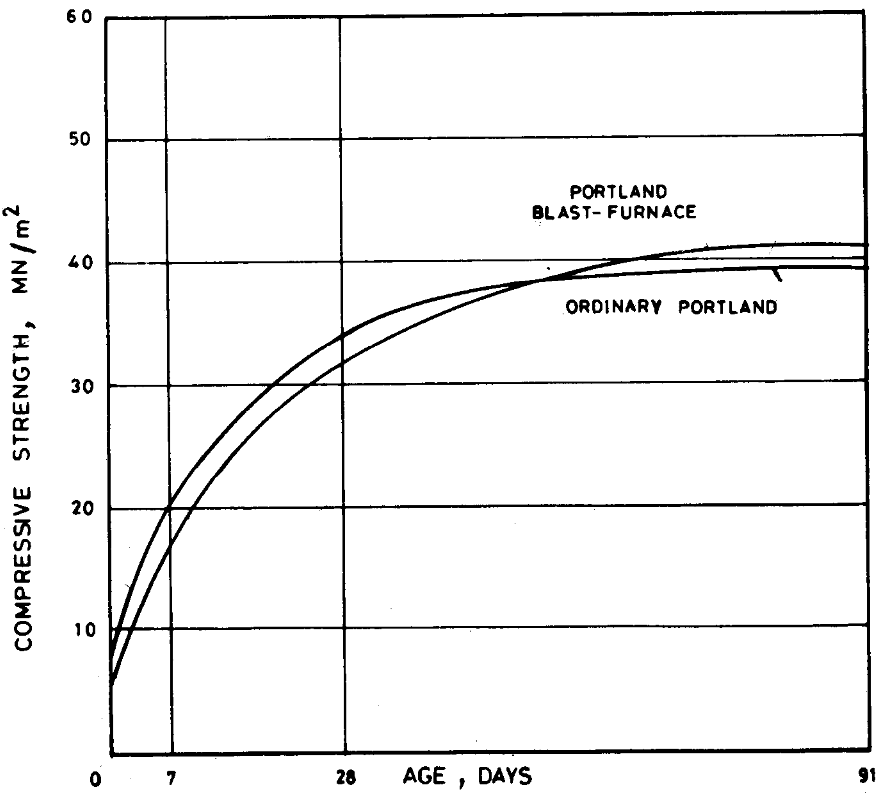


Fig. 7 Strength Development of Concretes Made with Portland Blast-Furnace Cement (Water-Cement Ratio = 0.6)

NOTE — Figure 7 is reproduced from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by M/s Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

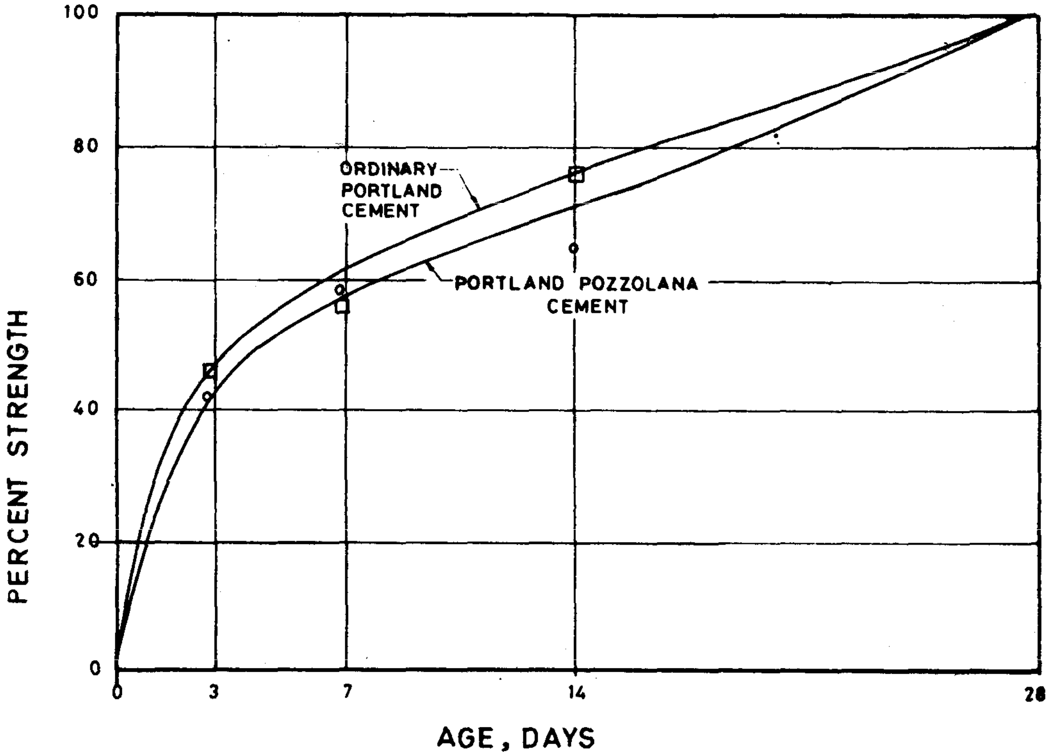


Fig. 8 Strength Development of M 15 Concrete Made with Portland-Pozzolana Cement (Water-Cement Ratio = 0.55)

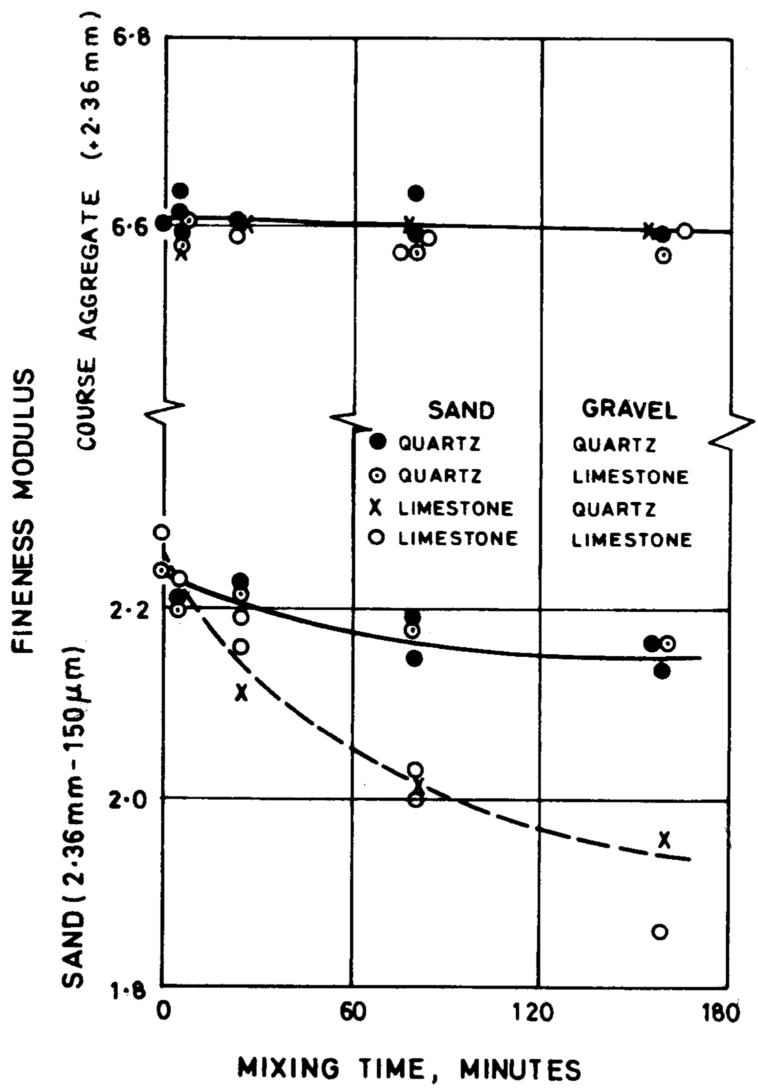


Fig. 9 Effect of Prolonged Mixing on Fineness Modulus of Sand and Gravel Sizes

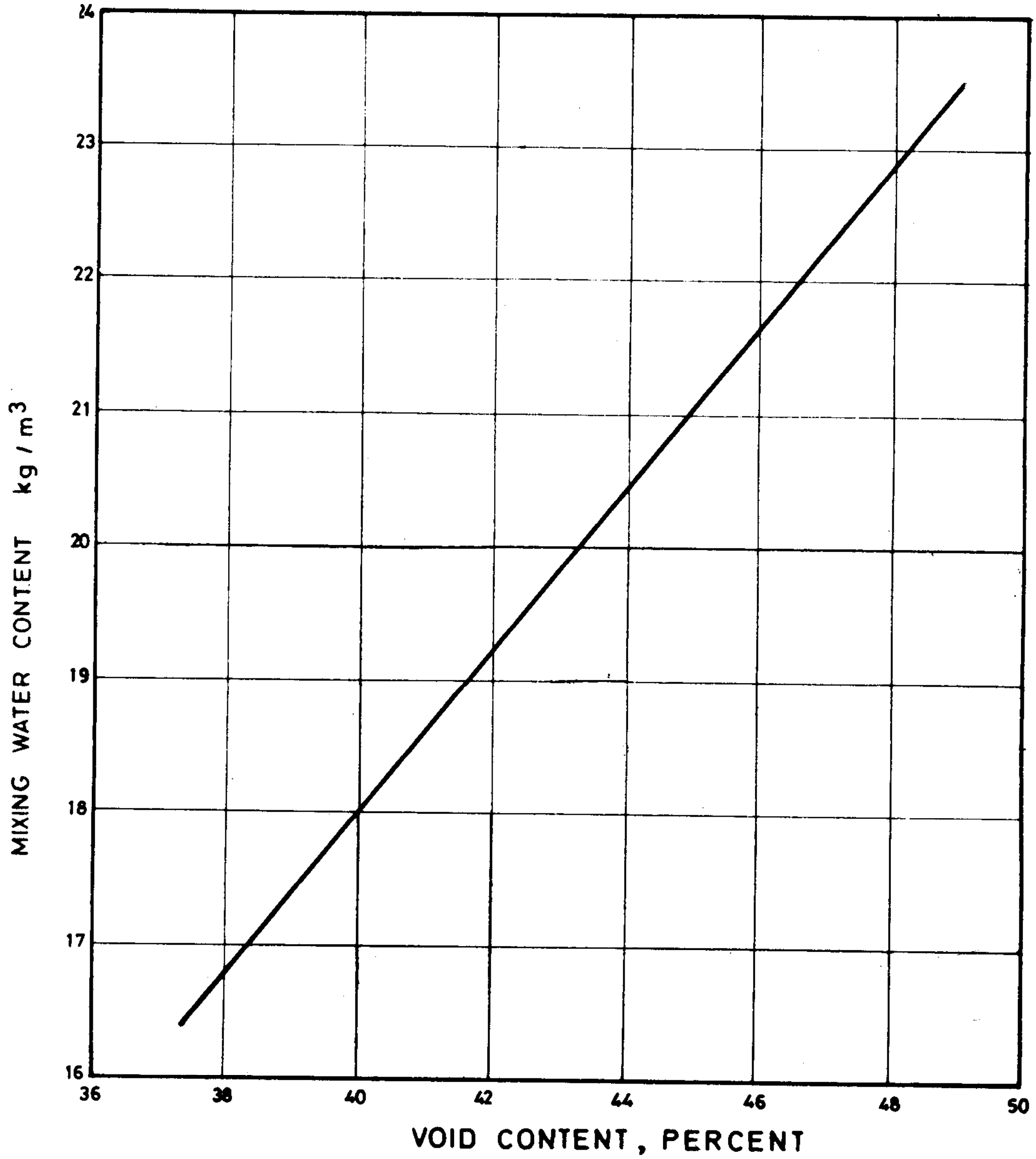


Fig. 10 Influence of Void Content of Sand in a Loose Condition on the Mixing Water Requirement of Concrete

NOTE — Figure 10 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

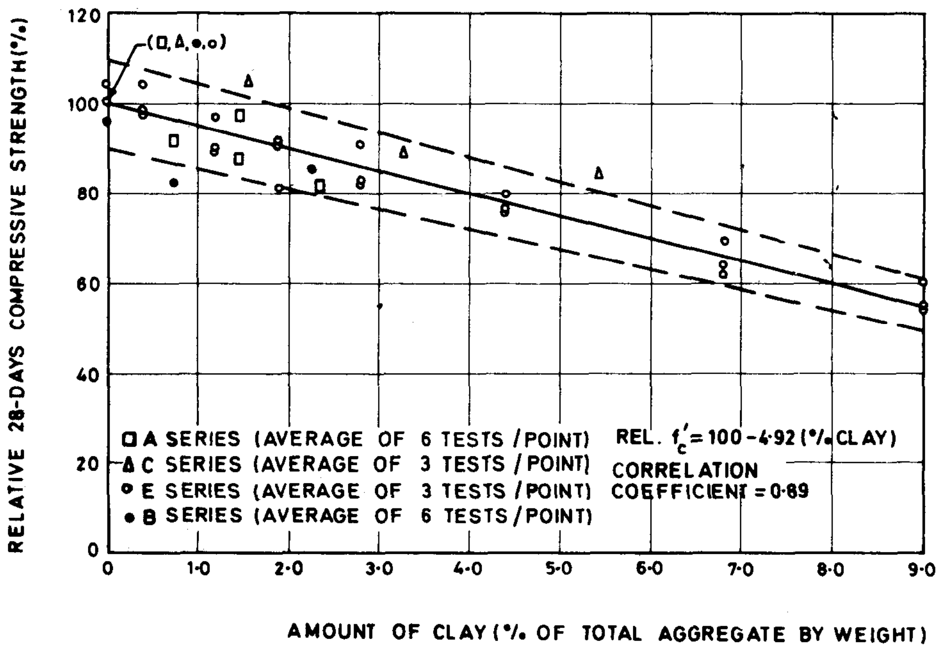
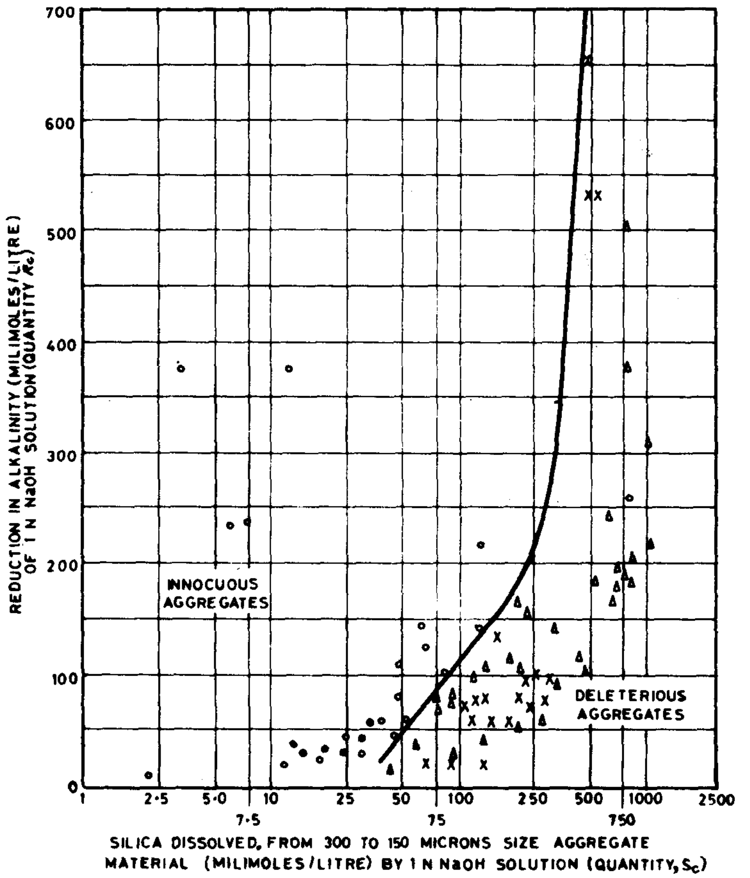


Fig. 11 Influence of Amount of Clay Fraction on 28-Day Compressive Strength



Δ Aggregate causing mortar expansion more than 0.1 percent in a year when used with a cement containing 1.38 percent alkalis

o Aggregate causing mortar expansion less than 0.1 percent in a year under same conditions

× Aggregates for which mortar expansion data are not available but which are indicated to be deleterious by petrographic examination

• Aggregates for which mortar expansion data are not available but which are indicated to be innocuous by petrographic examination

— Boundary line between innocuous and deleterious aggregates

Fig. 12 Illustration of Division Between Innocuous and Deleterious Aggregates on Basis of Reduction in Alkalinity Test

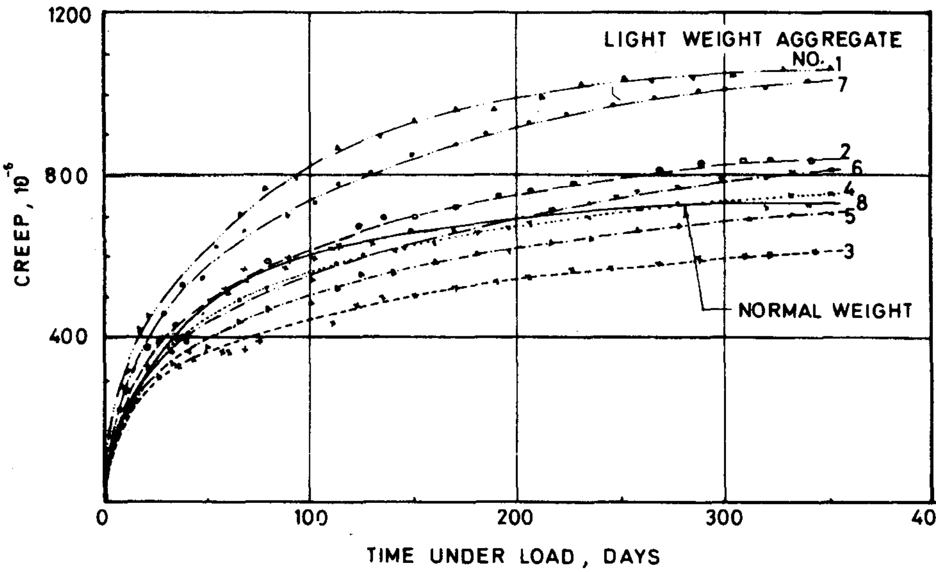


Fig. 13 Creep of Concrete of Nominal 210 kgf/cm² Strength Made with Different Lightweight Aggregates, Loaded at the Age of 7 Days to 42 kgf/cm²

NOTE — Figure 13 is taken from 'Creep of Concrete: Plain, Reinforced and Prestressed' (1970) by A. M. Neville and published by North-Holland Publishing Company, Amsterdam.

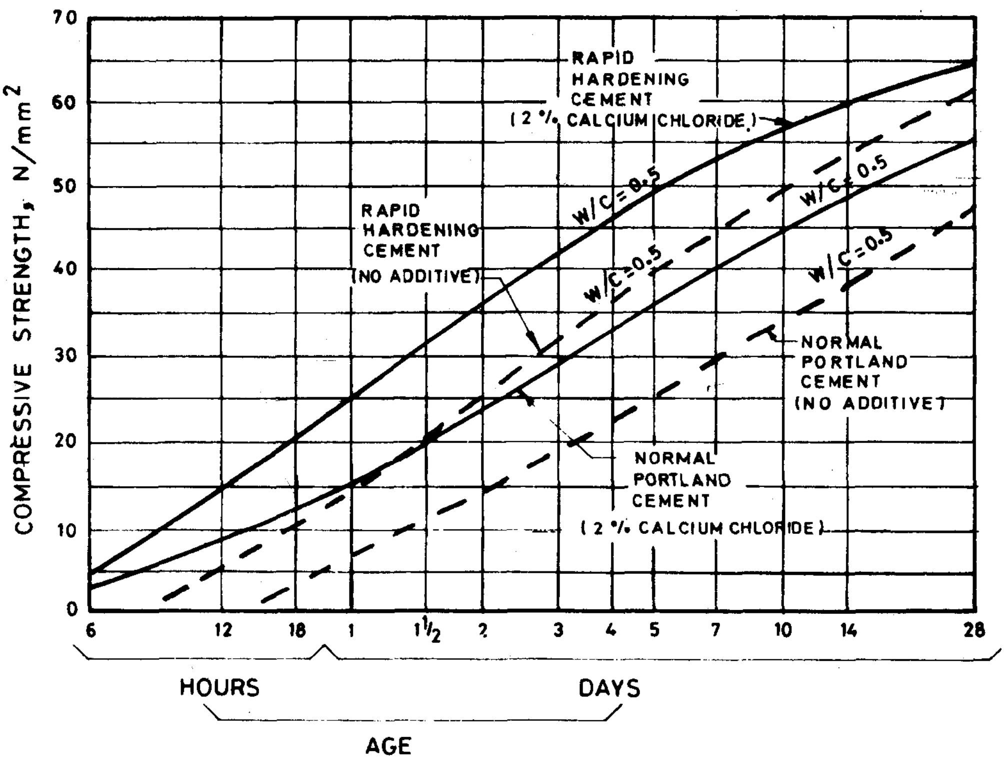


Fig. 14 Compressive Strength of Normal and Rapid Hardening Cement with 2 Percent Calcium Chloride Addition

NOTE — Figure 14 is from 'Concrete Technology (Fifth Edition, 1977)' by F. S. Fulton and published by Portland Cement Institute, Johannesburg.

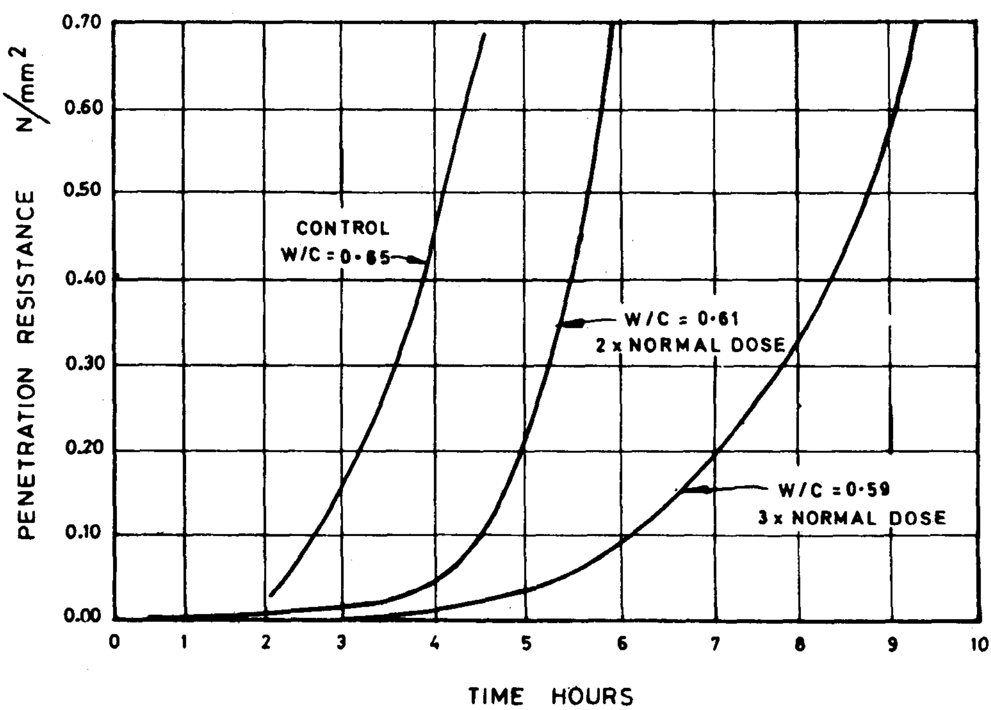


Fig. 15 Effect of Retarding Admixtures on Retention of Workability

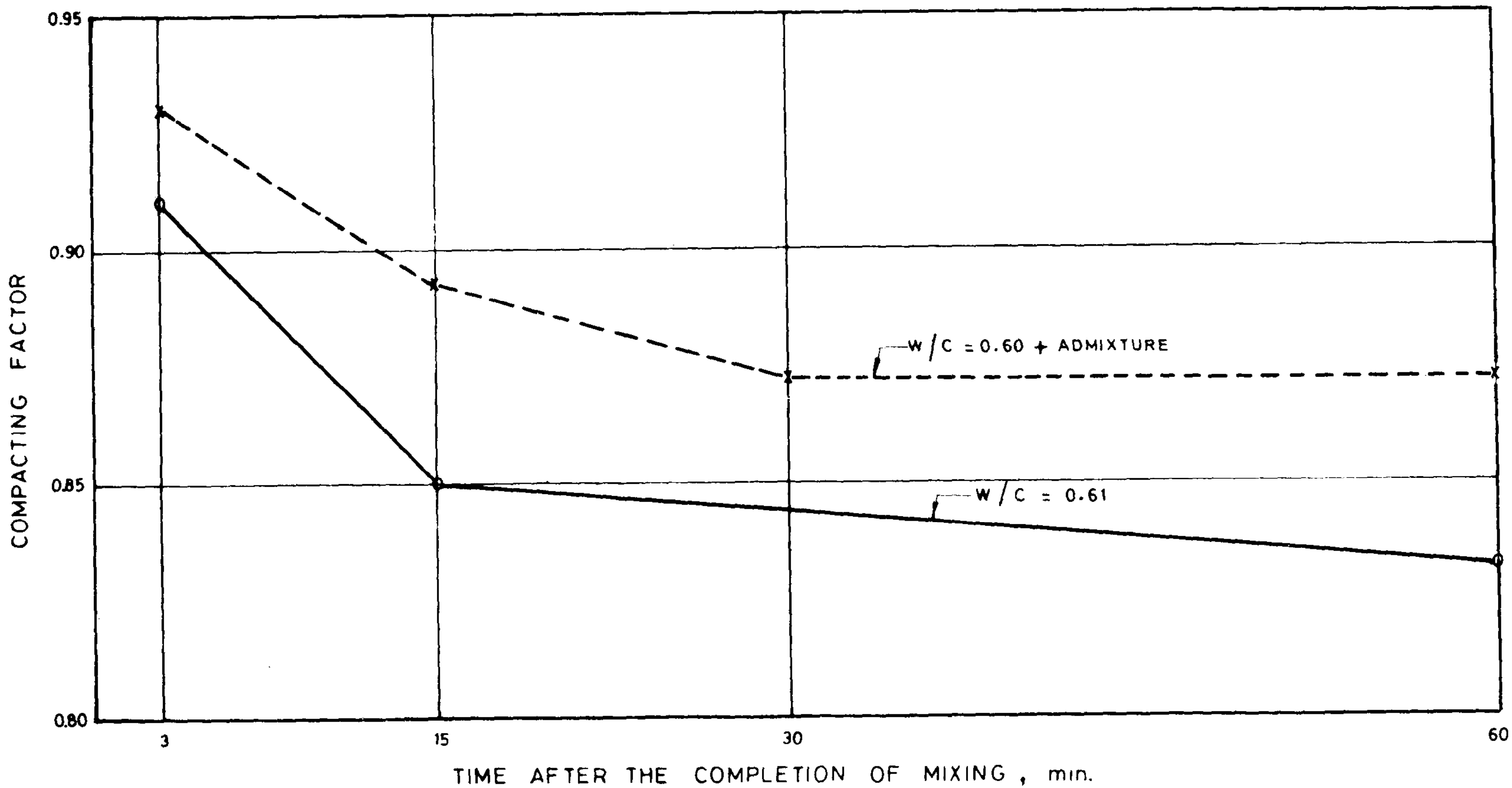


Fig. 16 Effect of Water Reducing Admixture on the Workability of Concrete

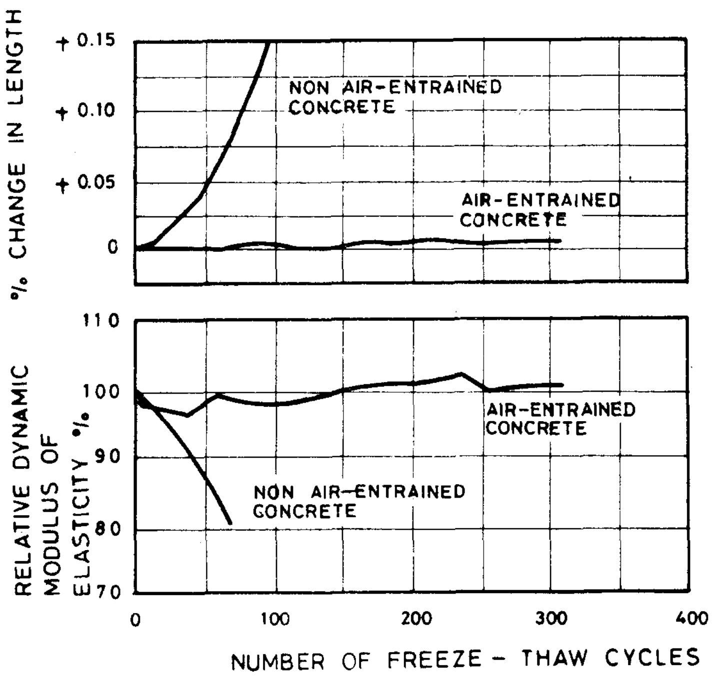


Fig. 17 Effect of Air-Entrainment on Freeze-Thaw Durability of Concrete

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SECTION 3

PROPERTIES OF FRESH AND HARDENED CONCRETE

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SECTION 3 PROPERTIES OF FRESH AND HARDENED CONCRETE

3.0 Introduction — Concrete has to have satisfactory properties both in the fresh and hardened states. While workability as defined below is the cardinal desirable property of fresh concrete, strength and durability are the most important properties of concrete in the hardened state. As was brought out in Section 1, demand of satisfactory properties of concrete in the fresh and hardened states may often bring conflicting requirements in the material and mix proportions; the aim of the rational mix design is also to reconcile these factors.

In this section, these important properties of concrete, namely, workability, compressive and tensile strengths and durability are discussed along with the various factors which influence them.

3.1 Workability — From the stage of mixing till it is transported, placed in the formwork and compacted, fresh concrete should satisfy a number of requirements which may be summarized as follows:

a) The mix should be stable, in that it should not segregate during transportation and placing. The tendency of bleeding should be minimized.

b) The mix should be cohesive and mobile enough to be placed in the form around the reinforcement and should be able to cast into the required shape.

c) The mix should be amenable to proper and thorough compaction as possible in the situation of placing and with the facilities of compaction.

d) It should be possible to obtain a satisfactory surface finish.

The diverse requirements of stability, mobility, compactability, placeability and finishability of fresh concrete mentioned above are collectively referred to as 'workability'. The workability of fresh concrete is thus a composite property. It is difficult to precisely define all the aspects in a single definition. IS : 6461 (Part VII)-1973[1] defines workability as 'that property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, compacted and finished ....'. It is also clear that the optimum workability of concrete varies from situation to situation and concrete which can be termed as workable for pouring into large sections with minimum reinforcement may not be equally workable for pouring in thin sections with heavier concentration of reinforcement. A concrete may not be workable when compacted by hand but may be satisfactory when mechanical vibration is used.

3.1.1 Different Measures of Workability — 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 test methods are being developed frequently, IS : 1199-1959[2] envisages the following three methods:

a) Slump test,

b) Compacting factor test, and

c) Vee-Bee consistency test.

Out of these three, the slump test is perhaps the most widely used, primarily because of the simplicity of the apparatus required and the test procedure. For such concretes where slump test is suitable (*see* below), the concrete after test slumps evenly all round which may be called 'true slump'. When the mixes are harsh or in case of very lean concrete one half of the cone may slide down the other which is called a 'shear slump'; or it may even collapse (Fig. 18[3]). 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 workabilities (that is, slump 25 to 125 mm). For very stiff mixes having zero slump, the slump test does not indicate any difference in concretes of different workabilities. It has been pointed out that different concretes having the same slump may have indeed different workability under the site conditions. However, when the uniformity among different batches of supposedly similar concretes under field conditions is to be measured, slump test has been found to be useful[4].

The compactability, that is, 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, that is, 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' workabilities (that is, compacting factor of 0.9 to 0.8). Its use has been more popular in laboratory conditions. For concrete of very low workabilities (that is, compacting factor of 0.70 and below which cannot be fully compacted for comparison, in the manner described in the test method) this test is not suitable.

Vee-Bee test is preferable for stiff concrete mixes having 'low' or 'very low' workability. Compared to the other two tests, Vee-Bee test has the advantage that the concrete in the test receives a similar treatment as it would be in actual practice. Since the end point of the test (when the glass plate rider completely covers the concrete) is to be ascertained visually, it introduces a source of error which is more pronounced for concrete mixes of high workability and consequently records low Vee-Bee time. The test is therefore, not suitable for concrete of higher workability that is, slump of 75 mm or above.

Experience has shown that mix proportions influence the workability of concrete, and more pertinently, different workability tests to different extent. Therefore, it is futile to expect a rigid correlation between the workabilities of concrete as measured by different methods. Table 20[5] attempts to describe different workabilities of concrete in terms of slump, Vee-Bee time and compacting factor. The table gives the range of expected values by different test methods for comparable concretes may give an indication of the correlation between them. Similarly Fig. 19[6] indicates a general pattern of relations between workability tests for concrete mixes having varying aggregate-cement ratios. In the absence of definite correlations between the different measures of workability under different conditions, it is recommended that, for a given concrete, the appropriate test method be decided beforehand and workability expressed in terms of such test only rather than interpreting from the results of other tests.

3.1.2 FACTORS AFFECTING WORKABILITY — The workability of fresh concrete depends primarily on the materials and mix proportions and also on the environmental conditions. These are discussed in *3.1.2.1* and *3.1.2.2*.

3.1.2.1 INFLUENCE OF MATERIALS AND MIX PROPORTIONS — While a number of relations between the various mix parameters and workability of fresh concrete are available, a rational approach to unify the effects of different mix variables can be sought in the cement-aggregates-water system. Aggregates occupy nearly 70 to 75 percent of the total volume of concrete and economy demands that the volume of aggregates in the concrete should be as large as possible. The particle interference as well as the total specific area of the aggregate are to be minimized to the extent possible by the proper choice of size, shape and proportion of fine and coarse aggregates. Different size fractions are to be so chosen as to minimize the voids content. Such a dry mixture of aggregates having minimum voids content will not be mobile and will need water for lubricating effects. The water-cement ratio in itself determines the intrinsic properties of the cement paste and the requirements of workability are such that there should be enough cement paste to surround the aggregate particles as well as to fill the voids in the aggregates. On an engineering scale, the water content of the mix is the primary factor governing the workability of the fresh concrete. The workability increases with the water content as shown in Fig. 20[7,8] and displays a relationship as follows[9]:

$$y = Cw^n$$

where

y = specified consistency value (for example, slump);

C = term which depends on the composition of the concrete (cement content, air content, aggregate grading, etc) and method of measuring consistency;

w = water content of fresh concrete; and

n = depends on the maximum size of aggregate.

This relationship implies that the amount of change in the measured value of workability (for example, slump or compacting factor) due to relative change of water content in concrete is independent of the composition of concrete within wide limits. As an example, Table 21[10] indicates the changes in the workability in terms of slump that can be expected with relative changes in the water content in the mix, taking the water requirement for a 40 mm slump as unity.

Next in importance are the aggregate properties, the effects of which can be summarized as follows:

a) For the same volume of aggregates in the concrete, use of coarse aggregates of larger size and/or rounded aggregates gives higher workability because of reduction in the total specific surface area and particle interference. Use of flaky/elongated aggregates results in low workability primarily because of increase in particle interference.

b) The use of fine sand with corresponding increase in specific surface area increases the water demand for the same workability or conversely for the same water content, workability decreases. If the sand is very coarse, the net effect on workability is, increase in particle interference and decrease in specific surface area.

c) Because of the greater contribution to the total specific area, the grading of the fine aggregates is more critical than the grading of coarse aggregates. Nevertheless, the proportion of fine to coarse aggregates should be so chosen as neither to increase the total specific surface area (by excess of fine aggregate) nor to increase the particle interference (due to deficiency in fine aggregate).

d) Once the water content in the mix is fixed, there is some relation between the water-cement ratio and the grading of the aggregates. It is seen in practice that there would be one optimum combination of coarse and fine aggregates resulting in the highest workability for a given water-cement ratio[3]. Generally, mixes with higher water-cement ratio would require a somewhat fine grading and for mixes with low water-cement ratio (as in the case of high strength concrete), coarser grading is preferable.

From the consideration of workability, the mix parameters are expressed in terms of water content, water-cement ratio and proportion of fine to coarse aggregates; once this is done, the aggregate-cement ratio is automatically fixed. Influence of the factors like grading and maximum size of aggregates and aggregates-cement ratio on the workability is discussed in more detail in Section 5.

3.1.2.2 EFFECTS OF TIME AND TEMPERATURE ON WORKABILITY — Fresh concrete looses workability with time mainly because of loss of moisture due to evaporation. Part of mixing water is absorbed by aggregates or lost by evaporation in the presence of sun and wind and part of it is consumed in the chemical reaction of hydration of cement. The loss of workability varies with the type of cement used, the concrete mix proportions, the initial workability and the temperature of the concrete. On an average, a 12 cm slump concrete may loose about 5 cm slump in the first one hour. The workability in terms of compacting factor decreases by about 0.10 during a period of one hour from the time of mixing. The workability of concrete mix appears to be relatively stable during the period from 15 to 60 minutes after the completion of mixing (*see* Fig. 21[11]). Although the standard methods of test for workability imply that tests be carried out soon after mixing, Fletcher[11] suggested that for general purposes, workability of fresh concrete should be determined not sooner than 15 minutes nor later than 20 minutes, after the completion of mixing especially where placing of concrete is likely to take so much time or longer. Such decrease in workability with time after mixing may be more pronounced in concretes with admixtures like plasticizers. Evaluation of workability at a certain time after mixing is perhaps more necessary than in normal concretes without admixture.

The workability of a concrete mix is also affected by the temperature of concrete and, therefore, by the ambient temperature. On a hot day, it becomes necessary to increase the water content of the concrete mix in order to maintain the desired workability unless other precautions are taken (*see 7.1.2*). The amount of mixing water required to bring about a certain change in workability also increases with temperature. Fig. 22[12] shows the effect of concrete temperature on the workability of concrete (in terms of slump) and the percentage change in water requirements per 25 mm change in slump, for different concrete temperatures.

3.1.3 Requirement of Workability — In addition to the desired compressive strength, the concrete should have workability such that it can be placed in the formwork and compacted with the minimum effort, without causing segregation or bleeding The choice of workability depends upon the type of compacting equipment available, the size of the section and the concentration of reinforcement. For heavily reinforced sections or when the sections are narrow or contain inaccessible parts or when the spacing of reinforcement makes placing and compaction difficult, concrete should be highly workable for full compaction to be achieved with a reasonable amount of effort. Table 22[13] gives range of workabilities required in terms of slump, compacting factor and V-B time for concretes depending upon the placing conditions at site. It may be noted that the nominal maximum size of aggregates itself makes a difference in the degree of workability that may be suitable under a particular placing condition. The range of values indicated are considered suitable for concretes having aggregates of nominal maximum size 20 mm. Generally the value of workability will increase with the increase in the size of aggregate and will be somewhat lower for aggregates of smaller size than indicated.

Notwithstanding the guidance for workability given in Table 22, the situation at hand should be properly assessed to arrive at the desired workability in each case. The aim should be to have the minimum possible workability consistent with satisfactory placing and compaction of the concrete. It should be remembered that insufficient workability resulting in incomplete compaction may severely affect the strength, durability and surface finish of concrete and may indeed be uneconomical in the long run. The effectiveness of vibration equipment available for compaction should also be taken into consideration.

3.2 Compressive Strength — The compressive strength of hardened concrete is considered to be the most important property. It can be measured easily on standard sized cube or cylindrical specimens and is often taken as an index of the overall 'quality' of concrete. Many other desirable properties of concrete, for example shear and tensile strength, modulus of elasticity, bond, impact, abrasion resistance and durability etc, are also taken to be related to the compressive strength, at least to a general extent.

Among the materials and mix variables, water-cement ratio is the most important parameter governing compressive strength. Besides water-cement ratio, the following factors also effect the compressive strength of concrete:

a) The characteristics of cement,
b) The characteristics and proportions of aggregates,
c) The degree of compaction,
d) The efficiency of curing,
e) The temperature during the curing period,
f) The age at the time of testing, and
g) The conditions of test.

The influence of mix proportions, placing, compaction, curing and age of testing, on the compressive strength of concrete are discussed in *3.2.1* to *3.2.3*.

3.2.1 Influence of Mix Proportions

a) *Water-cement ratio* — The compressive strength of concretes at a given age and under normal temperature, depends primarily on the water-cement ratio; lower the water-cement ratio, greater is the compressive strength and *vice-versa*. This was first enunciated by Abrams as:

$$S = \frac{K_1}{K_2^{\,w/c}}$$

where S is the compressive strength and w/c represents the water-cement ratio of a fully compacted concrete mix, and K_1 and K_2 are empirical constants.

In day to day practice, the constants K_1 and K_2 are not evaluated; instead the relationship between compressive strength and the water-cement ratio are adopted, which are supposed to be valid for a wide range of conditions. These relationships are discussed in Section 6.

b) *Aggregate-cement ratio* — The influence of the physical characteristics of the aggregates on the compressive strength of concrete was discussed earlier in Section 2. As long as the workability of concrete is maintained at a satisfactory level, the compressive strength of concrete had been found to increase, with increase in aggregate-cement ratio, the water-cement ratio being held constant[3]. However, in high-strength concrete mixes of lower workability, or in such situations where due to increase in aggregate-cement ratio the workability is reduced to such an extent that concrete cannot be properly placed and thoroughly compacted, the above is not true. In high-strength mixes of low workability, a decrease in aggregate-cement ratio may result in small increase in compressive strength, provided the water content in the mix is also reduced proportionately (Fig. 23[14]);

c) *Cement content and characteristics* — The water-cement ratio and the aggregate-cement ratio together determine the cement content of the concrete mix. Generally, the cement content itself would not have a direct role on the strength of concrete; if cement content is required to increase the workability of concrete mix for a given water-cement ratio, then the compressive strength may increase with the richness of the mix. However, for a particular water-cement ratio there would always be an optimum cement content resulting in 28-day compressive strength being the highest (*see* Fig. 24[15]). Increasing the cement content above the optimum value may not increase the strength of concrete specially for mixes with low water-cement ratio and larger maximum size aggregate[3].

The influence of strength of cement on strength of concrete is not very precisely known. It has generally been found that greater the 7-day strength of cement (as given by the standard test procedure), the greater is the compressive strength of concrete at 28 day, for identical mix proportions (Fig. 26[16]). However, there may not be a one-to-one correspondence between 7-day strength of cement and 28-day strength of concrete. Because the relative increase in strength of cement at 28-day is smaller, the greater the 7-day strength[17]; perhaps 28-day strength of cement would give better correlation. As a guide, it has been found that for an increase in 7-day strength of cement of 50 kgf/cm², the increase in 28-day strength of concrete is of the order of 40 to 50 kgf/cm².

d) *Effect of age of testing* — Concrete is generally tested for its compressive strength at the age of 28 days. Because of continuing hydration, the later age strength of concrete would generally be higher than at 28 days; however, the exact increase will depend upon the type of cement, mix composition and the extent of curing. The influence of the type of cement on the later strength of concrete has been discussed in Section 2. From Fig. 5 (Section 2) it may be seen that although cements of different types result in different rate of gain of initial strength, the strengths at later ages tend to become similar. Therefore, a cement type which results in comparatively lower strength at 28 days would have proportionately greater increase in strength at later ages and *vice-versa*. The mix proportions themselves influence the rate of gain of strength, in that concrete with lower water-cement ratio tends to attain high initial strength and, therefore, further gain in strength at later ages is proportionately smaller than with higher water-cement ratio. This is so because concrete with lower water-cement ratio would result in a greater gel-space ratio during the initial period but the hydration product may be laid in a more disorderly fashion thereby impairing further hydration to some extent.

Under general conditions, IS : 456-1978[13] allows the strength to be increased by 10 percent at 3 months, 15 percent at 6 months and 20 percent at 12 months over and above the 28-day strength. It is to be noted that IS : 456-1978[13] does not permit any such increase of strength with age where high alumina cement is used; the reason being that high alumina cement concretes tend to reach their potential strength much more quickly than other cements (*see* Fig. 5 of Section 2).

3.2.2 EFFECT OF PLACING, COMPACTION AND CURING — The concrete should be placed in its final position in the formwork as early as possible after the completion of mixing, so that there is no drying out of the mix, and the mix is workable enough to receive compaction. Dropping of concrete from great heights may lead to segregation and entrainment of air bubbles, displacement of the reinforcement and damage to the concrete already placed. If segregation takes place, it will result in concrete of poor quality. If, however, the mix is designed properly and proper precautions are taken to avoid segregation during placing, pouring of concrete from a height of 10 to 15 metres is not uncommon.

The necessity for thorough compaction is basic to successful concrete manufacture, since the concrete mix is designed on the basis that it may be thoroughly compacted with the available compacting equipment. When the fresh concrete is compacted by vibration, the particles are set in motion reducing inter-particle friction so that concrete is easily placed. Vibration eliminates most air pockets on the surface of the concrete.

The increase in compressive strength by lowering the water-cement ratio may be restricted if the compaction is insufficient, as shown schematically in Fig. 26[3]. The presence of even 5 percent voids in the hardened concrete left due to incomplete compaction may result in a decrease in compressive strength by about 35 percent (Fig. 27[19]).

As already pointed out in Section 2, the hydration of cement can take place only when the capillary pores remain saturated. In addition, additional water available from outside is needed to fill the gel-pores, which will otherwise make the capillaries empty. The functions of curing are thus two-fold; to prevent the loss of water in the concrete from evaporation as well as to supplement water consumed in hydration of cement. In concrete mixes with higher water-cement ratio, the hydration can proceed by self-desiccation and prevention of evaporation of water (for example, by covering with wet gunny bags, membranes and curing compounds) may be sufficient.

For high strength concrete with lower water-cement ratio, the mixing water may not be sufficient for hydration to proceed by self-desiccation and mere prevention of evaporation of water will not suffice. In such situations continuous ponding of water will be needed more than in case of mixes with high water-cement ratio (Fig. 28[3]).

Concrete will continue gaining strength with time provided that sufficient moisture is available for the hydration of cement which can be assured only by proper moist curing. (Fig. 29[20]) shows the effect of duration of moist curing on compressive strength of concrete. On an average, the one year strength of continuously moist cured concrete is 50 percent higher than that of 28-day moist cured concrete, while no moist curing can lower the strength by about 30 percent. Moist curing for first 7 to 14 days may result in compressive strength being 85 to 92 percent of that of 28 days moist curing[21]. IS : 456-1978[13] stipulates a minimum of 7 days moist curing, while IS : 7861 (Part I)-1975[12] stipulates a minimum of 10 days under hot weather conditions.

3.2.2.1 STEAM CURING OF CONCRETE — Since the chemical reactions of hydration of cement can be thermally activated, increased rate of strength development of concrete is achieved by resorting to steam curing at atmospheric pressure. The primary object of steam curing is to develop high early strength of concrete, so that concrete products can be removed from the formwork and handled as early as possible, and is mainly adopted in precast concrete works.

A number of considerations govern the choice of steam curing cycle namely the precuring period, the rate of increase and decrease of temperature and the level and time of constant temperature. An early rise in temperature at the time of setting of concrete may be detrimental to concrete because, the green concrete may be too weak to resist the air pressure set up in the pores by the increased temperature. Too high a rate of increase or decrease in temperature introduces thermal shocks and the rates should generally not exceed 10 to 20°C per hour. The higher the water-cement ratio of the concrete, the more adverse is the effect of an early rise in temperature. Therefore, in order to meet the requirement of compressive strength of concrete, the temperature and/or time required for curing can be reduced by having a lower water-cement ratio. While in an identical time cycle, higher the maximum temperature greater is the compressive strength. The advantages of curing above 70°C are negated by dilational tendencies due to expansion of concrete. All the above mentioned factors lead to the conclusion that for concrete of a specified composition and curing period, there is one curing temperature which will be able to produce maximum compressive strength at the end of the curing cycle.

A typical steam curing cycle is given in (Fig. 30[22]). In the normal steam curing procedure, a presteaming period of 1 to 3 hours is usual. The rate of initial temperature rise after the presteaming period is of the order of 10 to 20°C per hour and the maximum curing temperature is limited to 85 to 90°C. Temperature higher than this will not produce any increase in the strength of concrete and in fact, as discussed above, a temperature of 70°C may be sufficient. For a particular product, the maximum desired temperature raised at a moderate rate and then the steam is cut off, and the product is allowed to soak in the residual heat and moisture of the curing chamber. Due to differences in the product and the methods of manufacture, different curing cycles are to be adopted based on local conditions. By adopting proper steam curing cycle, more than 70 percent of the 28-day compressive strength of concrete can be obtained in about 16 to 24 hours[22].

Recent findings[23] suggest that the steam curing of concrete should be followed by water curing for at least 7 days (Fig. 31). In absence of this supplementary wet curing for at least 7 days, the later-age strength of steam-cured concrete may be lower by 20 to 40 percent than that of normally-cured concrete.

3.2.3 Relation with Tensile Strength — The tensile strength (both flexural and split tensile) is closely related to compressive strength of concrete, but there is no direct proportionality between them, the ratio of the two strengths being a function of the level of concrete strength. As compressive strength increases, the tensile strength also increases but at a decreasing rate.

IS : 456-1978[13] gives a formula for flexural strength in terms of the characteristic compressive strength of concrete, as indicated below:

$$f_{cr} = 0.70\sqrt{f_{ck}} \text{ N/mm}^2$$

In order to obtain a quicker idea of the quality of concrete, IS : 456-1978[13] also specifies optional tests (compressive as well as flexural strengths at 7 days) for different grades of concrete. A comparison of the relationships between compressive strengths and flexural strengths, at 7 and 28 days age of concrete is shown in Fig. 32. According to PCA[19], the flexural strength is given by:

$$F_{cr} = K\sqrt{f_c} \text{ N/mm}^2$$

where f_c is the cube strength of concrete, and K has a value of 0.68. In a large number of tests carried out, K varied from 0.73 for crushed quartzite aggregate down to 0.48 for rounded river pebbles.

ACI 318-77[24] gives the relationship as:

$$f_{cr} = 0.63\sqrt{f_c} \text{ N/mm}^2$$

where f_c is the specified compressive strength of concrete.

This relationship is also shown in Fig. 32 for comparison.

Numerous factors influence the above relationship of the strengths. Incomplete compaction has greater effect on compressive strengths than on flexural strength. The tensile strength of concrete is more sensitive to inadequate curing than the compressive strength possibly because of the serious effects of non-uniform shrinkage (of flexure test beams). Thus air-cured concrete has a low tensile to compressive strength value than concrete cured in water and tested wet[19]. The ratio of two strengths is also affected by the grading of the aggregate[3]. This is because of the different magnitude of the wall effect in beams and; the surface/volume ratios being different with the requirement of different quantities of mortar for thorough compaction in compression specimens. The experimental results obtained by various investigators for the relationship between split tensile strength and compressive strength of concrete lie within the zone indicated in Fig. 33[25] while a representative relationship with flexural strength is given in Fig. 34[25]. In absence of detailed test data, it may be held that the intrinsic tensile strength of concrete is of the order of 10 to 15 percent of compressive strength. Flexural strength is on an average 50 percent greater than split tensile strength (Fig. 35[25]).

3.3 Durability of Concrete — Durability of concrete can be defined and interpreted to mean its resistance to deteriorating influences which may through inadvertance or ignorance reside inside the concrete itself, or which are inherent in the environment to which concrete is exposed[26]. Normally every batch of concrete is designed to serve a useful life of many years, and under normal circumstances, concrete is generally durable. Problems arise when concrete contains ingredients which were not known beforehand to be deleterious or when it is exposed to harmful environments not anticipated earlier.

The absence of durability may be either caused by external agencies like weathering, attack by natural or industrial liquids and gases, bacterial growth, etc, or by internal agencies like harmful alkali-aggregate reactions, volume changes due to non-compatibility of thermal and mechanical properties of aggregate and cement paste, presence of sulphates and chlorides from ingredients of concrete, etc. In the case of reinforced concrete, the ingress of moisture or air will facilitate the corrosion of steel, leading to an increase in the volume of the steel and cracking and spalling of concrete cover.

Recommendations for making durable concrete in various codes of practices envisage limits for maximum water-cement ratio, minimum cement content, cover thickness, type of cement and amount of chlorides and sulphates in concrete, etc. All these recommendations taken together tend to result in concrete being dense, workable, placeable and having as low a permeability as possible under the given situation. Therefore, adherence to one limit without considering others, or uniform application of these recommendations with no regard to the situation of placing, for example, congestion of reinforcement, cover thickness, workability of concrete or the characteristics of the aggregates, may not ensure the fulfilment of the objectives. These can be explained by means of Fig. 36[28] and 37[28]. Starting with the water-cement ratio, it is known that the permeability of cement paste increases exponentially with increase in water-cement ratios above 0.45 or so; from the considerations of permeability, the water-cement ratio is thus usually restricted to 0.45 to 0.55, except in mild environment. For a given water-cement ratio, a given cement content in the concrete mix will correspond to a given workability, namely high, medium or low (Fig. 36) and an appropriate value has to be chosen keeping in view the placing condition, cover thickness and the concentration of reinforcement. In addition, the cement content is chosen by two other considerations. Firstly, it should ensure sufficient alkalinity (*p*H value of concrete) to provide a passive environment against corrosion of steel, for example, in concrete in marine environment or in sea water, a minimum cement content of 350 kg/m^3 or more is required for this consideration[13, 28]. Secondly, the cement content and water-cement ratio is so chosen as to result in sufficient volume of cement paste to overfill the voids in the compacted aggregates. Clearly, this will depend upon the type and nominal maximum size of aggregate employed. For example, crushed rock or rounded river gravels of 20 mm maximum size of aggregate will, in general, have respectively 27 and 22 percent of aggregate voids. A cement content of 400 kg/m^3 and water-cement ratio of 0.45 will result in paste volume being 30 percent which may be suitable for the former (that is crushed rock of 20 mm maximum size aggregate), whereas cement content of 300 kg/m^3 and water-cement ratio of 0.50 will result in 25 percent paste volume (Fig. 37) being sufficient to overfill the voids in 20 mm rounded gravel aggregates. Increasing cement content will result in higher workabilities.

IS : 456-1978[13] lists the requirements for durable concretes in terms of minimum cement content, type of cement and maximum water-cement ratio required for reinforced concrete structures to ensure durability against: (a) specified conditions of exposure, and (b) different concentration of sulphates present in soil and ground water. These are reproduced in Tables 23 and 24 respectively. Similar requirements for prestressed concrete structures as per IS : 1343-1980[27] are reproduced in Tables 25 and 26. The purpose of specifying a minimum cement content is to ensure reasonable durability as discussed above. The values specified in Tables 23 and 24 are in general for 20 mm nominal maximum size of aggregate. The cement content has to be reduced or increased as the nominal maximum size of aggregate increases or decreases, respectively.

Concrete in sea-water or exposed directly along the sea-coast should be at least M 15 grade in the case of plain concrete and M 20 in the case of reinforced concrete. The use of Portland slag or Portland pozzolana cement is advantageous under such conditions[13]. In addition, IS : 456-1978[13] stipulates that the cover to concrete be modified as follows:

Increased cover thickness may be provided when surfaces of concrete members are exposed to the action of harmful chemicals (as in the case of concrete in contact with earth contaminated with such chemicals), acid, vapour, saline atmosphere, sulphurous smoke (as in the case of steam-operated railways), etc, and such increase of cover may be between 15 mm and 50 mm beyond the values for normal conditions; however, the actual cover should not exceed 75 mm.

For reinforced concrete members totally immersed in sea-water, the cover should be 40 mm more than that specified for normal conditions. For reinforced concrete members, periodically immersed in sea-water or subject to sea spray, the cover of concrete should be 50 mm more than that specified for normal conditions. For concrete of grade M 25 and above, the additional thickness of cover specified above may be reduced to half.

The type of cement is also important in order to resist the sulphate solutions in soil and ground water. The ordinary Portland cement having C_3A content less than 5 percent has got the maximum resistance against sulphatic environment. Super sulphated cement is supposed to provide an acceptable life for concrete against acidic environment, when the concrete is dense and made with a water-cement ratio of 0.40 or less. The Portland slag cement and the Portland pozzolana cement are preferable in marine or sulphuric conditions[26,28] although IS : 456-1978[13] has treated it at par with ordinary Portland cement.

The chances of deterioration of concrete from harmful chemical salts (chlorides and sulphates) should be minimized. The levels of such harmful salts in concrete coming from the concrete making materials that is, cement, aggregates, water and admixture as well as by diffusion from environment should be limited.

For the corrosion of embedded steel to begin, there is a threshold value for the chloride content in concrete dependent upon the alkalinity present (*p*H of concrete). Chloride in concrete may be present in the water (in soluble form) or chemically combined with other ingredients. Soluble chlorides induce corrosion but chemically combined chloride is believed to have little effect[29]. Tests on soluble chloride are time-consuming and difficult. It is easier to measure the total (soluble plus combined) chloride and to test for soluble chloride, only when follow up studies are needed[29]. Oxygen and moisture are necessary for electro-chemical corrosion. Table 27 indicates the recommendations of ACI Committee[26] on limits for chloride ion in concrete in different conditions of exposure. IS : 456-1978[13] limits the total amount of chloride in concrete to 0.15 percent by mass of cement, on the basis of recommendations contained in Reference 29. For prestressed concrete, IS : 1343-1980[27] limits the total amount of chloride ions in concrete to 0.06 percent by mass of cement.

The sulphates in concrete may be from different ingredients of concrete, that is cement, coarse and fine aggregates, water and admixture. IS : 456-1978[13] limits the total sulphate content of the concrete mix from all such sources to 4 percent by mass of cement.

Resistance to alternate freezing and thawing is not so important for the conditions prevailing in the country. But wherever situations demand, air-entrained concrete could be employed using an air-entraining admixture. Air-entrainment lowers the compressive strength but increase workability of concrete, which may permit certain reduction in the water content of the concrete mix so that the loss in the compressive strength of concrete can be compensated by lowering the water-cement ratio.

TABLE 20 COMPARISON OF CONSISTENCY MEASUREMENTS BY VARIOUS METHODS

(*Clause* 3.1.1)

Workability Description	Slump mm	Vee-Bee Time s	Compacting Factor
Extremely dry	—	32-18	—
Very stiff	—	18-10	0.70
Stiff	0-25	10-5	0.75
Stiff plastic	25-50	5-3	0.85
Plastic	75-100	3-0	0.90
Flowing	150-175	—	0.95

NOTE — Table 20 is from 'Recommended Practice for Selecting Proportions for Normal Weight Concrete' Reported by ACI Committee 211 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

TABLE 21 RELATION BETWEEN SLUMP AND RELATIVE WATER CONTENT

(*Clause* 3.1.2.1)

Sl No.	Slump mm	Relative Volume of Water in the Mix
1)	25	0.962
2)	40	1.000
3)	50	1.012
4)	75	1.045
5)	100	1.069
6)	125	1.088
7)	150	1.105
8)	175	1.119

NOTE — Table 21 is from 'The Properties of Fresh Concrete' (1968) by T.C. Powers and published by John Wiley and Sons, Inc., New York.

TABLE 22 SUGGESTED RANGES OF VALUES OF WORKABILITY OF CONCRETE FOR DIFFERENT PLACING CONDITIONS

(*Clause* 3.1.2.2)

Placing Conditions	Degree of Workability	Values of Workability
(1)	(2)	(3)
Concreting of shallow sections with vibration	Very low	20-10 seconds, Vee-Bee time or 0.75-0.80, compacting factor
Concreting of lightly reinforced sections with vibration	Low	10-5 seconds, Vee-Bee time or 0.80-0.85, compacting factor
Concreting of lightly reinforced sections without vibration, or heavily reinforced sections with vibration	Medium	5-2 seconds, Vee-Bee time or 0.85-0.92, compacting factor or 25-75 mm, slump for 20 mm* aggregate
Concreting of heavily reinforced sections without vibration	High	Above 0.92, compacting factor or 75-125 mm, slump for 20 mm* aggregate

*For smaller aggregate the values will be lower.

TABLE 23 MINIMUM CEMENT CONTENT REQUIRED IN CEMENT CONCRETE TO ENSURE DURABILITY UNDER SPECIFIED CONDITIONS OF EXPOSURE

(*Clause* 3.3)

EXPOSURE	PLAIN CONCRETE		REINFORCED CONCRETE	
	Minimum Cement Content	Maximum Water Cement Ratio	Minimum Cement Content	Maximum Water Cement Ratio
(1)	(2)	(3)	(4)	(5)
	kg/m³		kg/m³	
Mild — For example, completely protected against weather, or aggressive conditions, except for a brief period of exposure to normal weather conditions during construction	220	0.7	250	0.65
Moderate — For example, sheltered from heavy and wind driven rain and against freezing, whilst saturated with water; buried concrete in soil and concrete continuously under water	250	0.6	290	0.55
Severe — For example, exposed to sea water, alternate wetting and drying and to freezing whilst wet, subject to heavy condensation of water or corrosive fumes	310	0.5	360	0.45

NOTE 1 — When the maximum water-cement ratio can be strictly controlled, the cement content may be reduced by 10 percent.

NOTE 2 — The minimum cement content is based on 20 mm aggregate. For 40 mm aggregate, it should be reduced by about 10 percent; for 12.5 mm aggregate, it should be increased by about 10 percent.

TABLE 24 REQUIREMENTS FOR PLAIN AND REINFORCED CONCRETE EXPOSED TO SULPHATE ATTACK

(*Clause* 3.3)

CLASS	CONCENTRATION OF SULPHATES, EXPRESSED AS SO_3			TYPE OF CEMENT	REQUIREMENTS FOR DENSE, FULLY COMPACTED CONCRETE MADE WITH AGGREGATES COMPLYING WITH IS : 383-1970*	
	In Soil		In Ground Water (Parts per 100 000)			
	Total SO_3 (percent)	SO_3 in 2:1 water extract g/l			Minimum Cement Content	Maximum Free Water/ Cement Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)
					kg/m³	
1)	Less than 0.2	—	Less than 30	Ordinary Portland cement or Portland slag cement or Portland pozzolana cement	280	0.55
2)	0.2 to 0.5	—	30 to 120	Ordinary Portland cement or Portland slag cement or Portland pozzolana cement	330	0.50
				Supersulphated cement	310	0.50
3)	0.5 to 1.0	1.9 to 3.1	120 to 250	Supersulphated cement	330	0.50

NOTE 1 — This table applies only to concrete made with 20 mm aggregates complying with the requirements of IS : 383-1970* placed in near-neutral ground waters of *p*H 6 to 9, containing naturally occuring sulphates but not contaminants, such as ammonium salts. For 40 mm aggregate the value may be reduced by about 15 percent and for 12.5 mm aggregate, the value may be increased by about 15 percent. Concrete prepared from ordinary Portland cement would not be recommended in acidic conditions (*p*H 6 or less). Supersulphated cement gives an acceptable life in mineral acids, down to *p*H 3.5 provided that the concrete is dense and prepared with a water/cement ratio of 0.4 or less.

NOTE 2 — The cement contents given in Class 2 are the minimum recommended. For SO_3 contents near the upper limits of Class 2, cement contents above these minimum are advised.

NOTE 3 — Where the total SO_3 in col 2 exceeds 0.5 percent, a 2:1 water extract may result in a lower site classification if much of the sulphate is present as low solubility calcium sulphate.

NOTE 4 — For severe conditions such as thin sections under hydrostatic pressure on one side only and sections partly immersed, considerations should be given to a further reduction of water-cement ratio, and if necessary an increase in the cement content to ensure the degree of workability needed for full compaction and thus minimum permeability.

NOTE 5 — Portland slag cement conforming to IS : 455-1976† with slag content more than 50 percent exhibits better sulphate resisting properties.

NOTE 6 — Ordinary Portland cement with the additional requirement that C_3A content be not more than 5 percent and $2\ C_3\ A + C_4AF$ (or its solid solution $4CaO,\ Al_2\ O_3,\ Fe_2O_3 + 2CaO,\ Fe_2O_3$) be not more than 20 percent may be used in place of supersulphated cement.

*Specification for coarse and fine aggregates from natural sources for concrete (*second revision*).
†Specification for Portland slag cement (*third revision*).

TABLE 25 MINIMUM CEMENT CONTENT REQUIRED IN CEMENT CONCRETE TO ENSURE DURABILITY UNDER SPECIFIED CONDITIONS OF EXPOSURE FOR PRESTRESSED CONCRETE

(*Clause* 3.3)

Exposure	Minimum Cement Content kg/m^3	Maximum Water Cement Ratio
(1)	(2)	(3)
Mild — For example, completely protected against weather or aggressive conditions, except for a brief period of exposure to normal weather conditions during construction	300	0.65
Moderate — For example, sheltered from heavy and wind driven rain and against freezing, whilst saturated with water, buried concrete in soil and concrete continuously under water	300	0.55
Severe — For example, exposed to sea water, alternate wetting and drying and to freezing whilst wet subject to heavy condensation or corrosion fumes	360	0.45

NOTE — The minimum cement content is based on 20 mm nominal maximum size. For 40 mm aggregate, minimum cement content should be reduced by about 10 percent under severe exposure condition only, for 12.5 mm aggregate the minimum cement content should be increased by about 10 percent under moderate and severe exposure conditions only.

TABLE 26 REQUIREMENTS FOR PRESTRESSED CONCRETE EXPOSED TO SULPHATE ATTACK

(*Clause* 3.3)

CLASS	CONCENTRATION OF SULPHATES, EXPRESSED AS SO_3			TYPE OF CEMENT	REQUIREMENTS FOR DENSE, FULLY COMPACTED CONCRETE MADE WITH AGGREGATES COMPLYING WITH IS : 383-1970*	
	In Soil		In Ground Water (Parts per 100 000)			
	Total SO_3 (percent)	SO_3 in 2:1 water extract g/l			Minimum Cement Content	Maximum Free Water/ Cement Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)
					kg/m^3	
1)	Less than 0.2	—	Less than 30	Ordinary Portland cement or Portland slag cement	280	0.55
2)	0.2 to 0.5	—	30 to 120	Ordinary Portland cement (*see* Note 5) or Portland slag cement	330	0.50
3)	0.5 to 1.0	1.9 to 3.1	120 to 250	Ordinary Portland cement (*see* Note 5)	330	0.50

NOTE 1 — This Table applies only to concrete made with 20 mm aggregates complying with the requirements of IS : 383-1970* placed in near-neutral ground waters of *p*H 6 to 9, containing naturally occuring sulphates but not contaminants, such as ammonium salts. For 40 mm aggregate the value may be reduced by about 15 percent and for 12.5 mm aggregate the value may be increased by about 15 percent. Concrete prepared from ordinary Portland cement would not be recommended in acidic conditions (*p*H 6 or less).

NOTE 2 — The cement contents given in Class 2 are the minimum recommended. For SO_3 contents near the upper limit of Class 2, cement contents above these minimum are advised.

NOTE 3 — Where the total SO_3 in col 2 exceeds 0.5 percent, then a 2:1 water extract may result in a lower site classification if much of the sulphate is present as low solubility calcium sulphate.

NOTE 4 — For severe conditions such as thin sections under hydro-static pressure on one side only and sections partly immersed, considerations should be given to a further reduction of water-cement ratio, and if necessary an increase in the cement to ensure the degree of workability needed for full compaction and thus minimum permeability.

NOTE 5 — For Class 3 concrete, ordinary Portland cement with the additional requirement that C_3A content be not more than 5 percent and $2C_3A + C_4AF$ (or its solid solution $4CaO, Al_2O_3, Fe_2O_3 + 2CaO, Fe_2O_3$) not more than 20 percent is recommended. If this cement is used for Class 2 concrete, minimum cement content may be reduced to 310 kg/m^3.

*Specification for coarse and fine aggregates from natural sources of concrete (*second revision*).

TABLE 27 LIMIT FOR CHLORIDE ION IN CONCRETE PRIOR TO EXPOSURE IN SERVICE

(*Clause* 3.3)

SL No.	TYPE OF CONCRETE OR SERVICE	CHLORIDE ION, PERCENT BY MASS OF CEMENT
(1)	(2)	(3)
1)	Prestressed concrete	0.06
2)	Conventionally reinforced concrete located in a moist environment and exposed to chloride	0.10
3)	Conventionally reinforced concrete located in a moist environment but not exposed to chloride	0.15
4)	Above ground building construction where the concrete will stay dry (does not include location where the concrete will be occasionally wetted, such as kitchens, parking garages and water front structures)	No limit for protection against corrosion*

*If calcium chloride is used as an admixure, it is generally recommended that the limit be set at 2 percent by mass of cement for reason other than corrosion.

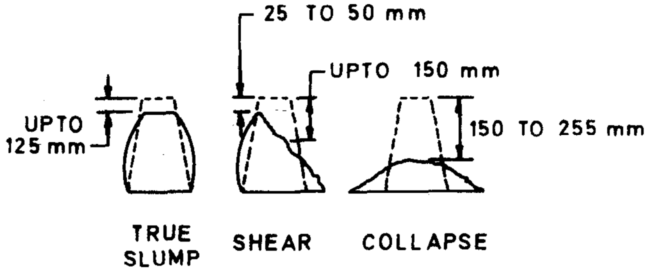


Fig. 18 Slump: True, Shear and Collapse

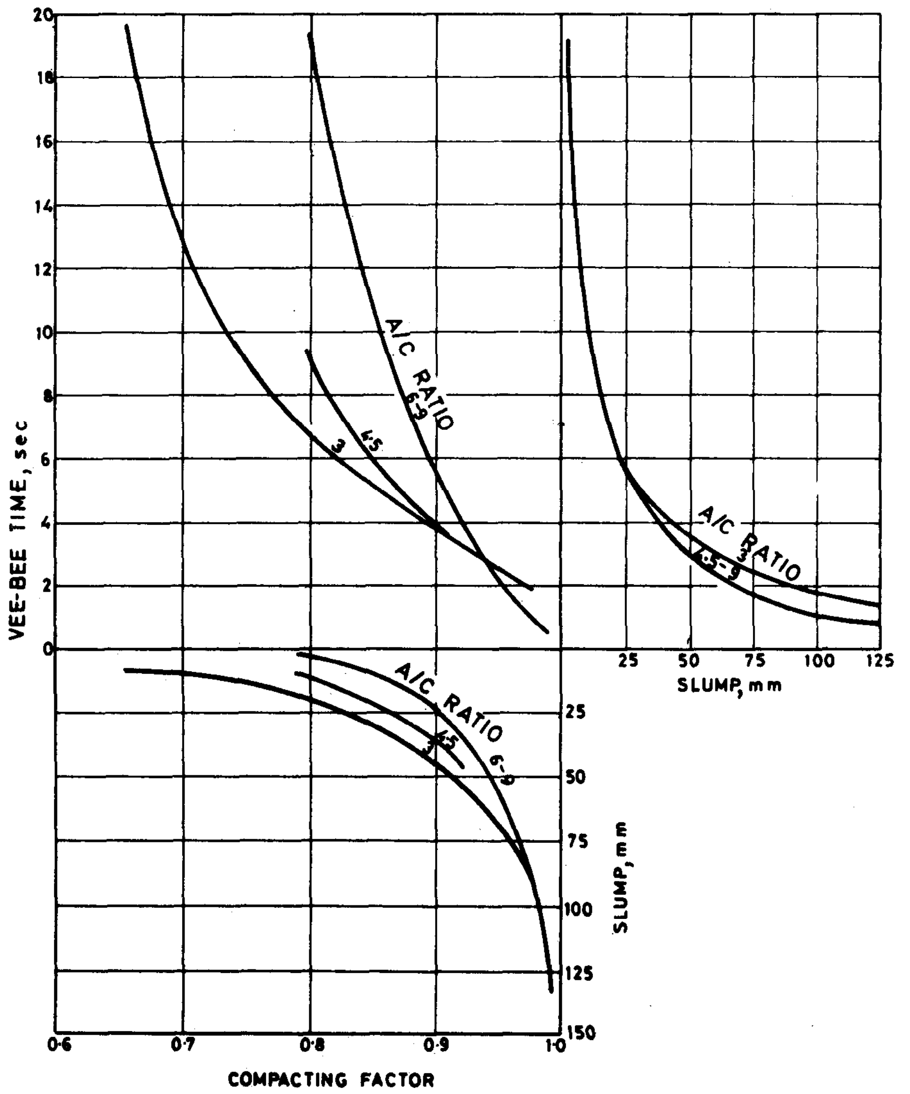


Fig. 19 Relationship Between Slump, Compacting Factor and Vee-Bee Time for Concrete of Different Aggregate-Cement Ratios

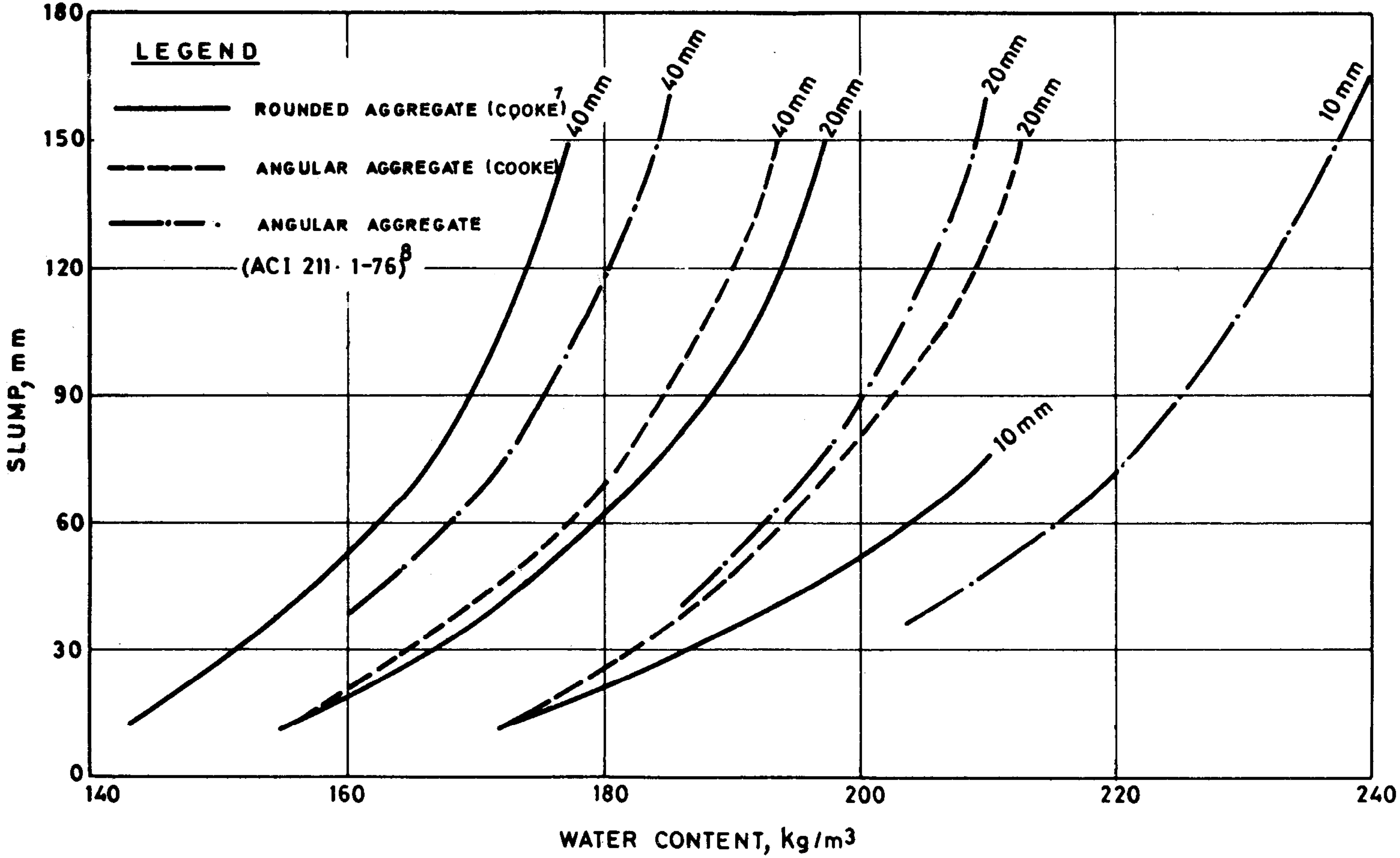


Fig. 20 Relationship Between Workability and Water Content of Concrete for Different Maximum Sizes of Aggregate

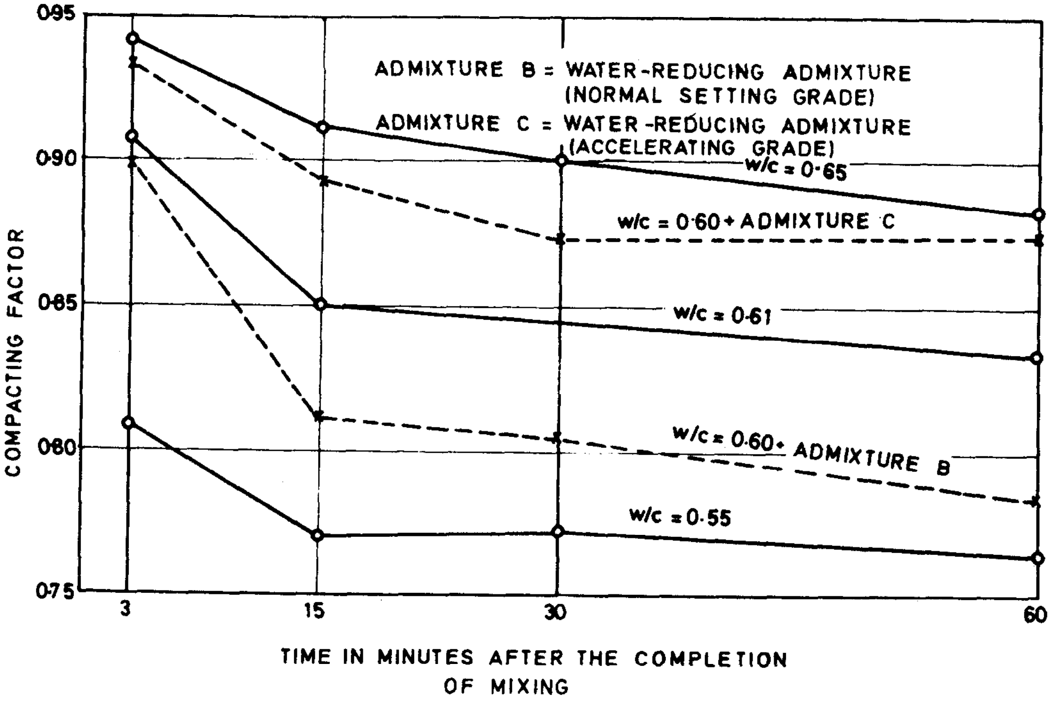


Fig. 21 Change in Compacting Factor with Time

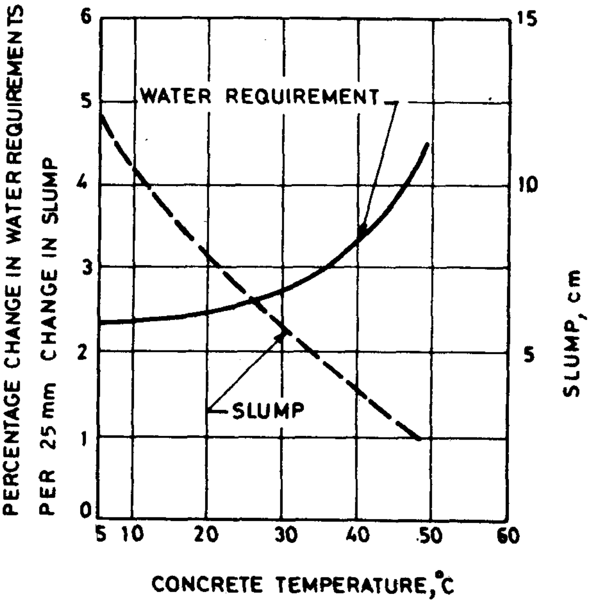


Fig. 22 Effect of Concrete Temperature on Slump and on Water Required to Change Slump

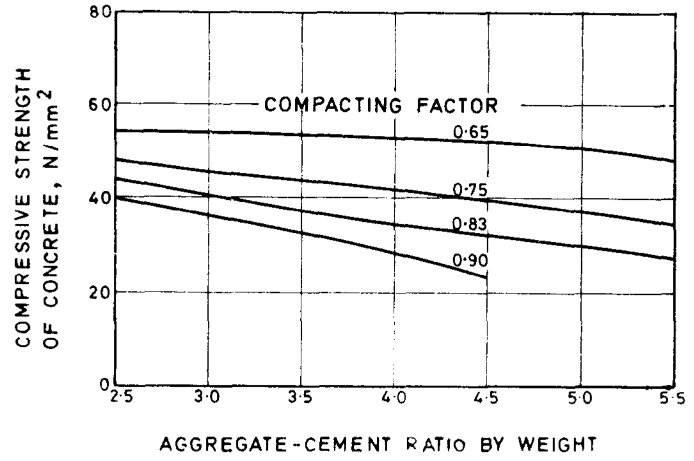


Fig. 23 Typical Relation Between Compressive Strength of Concrete and Aggregate-Cement Ratio for Various Compacting Factors

NOTE — Figure 23 is from 'Concrete Mix Design (Second Edition, 1966)' by J. D. Mcintosh and published by Cement and Concrete Association, London.

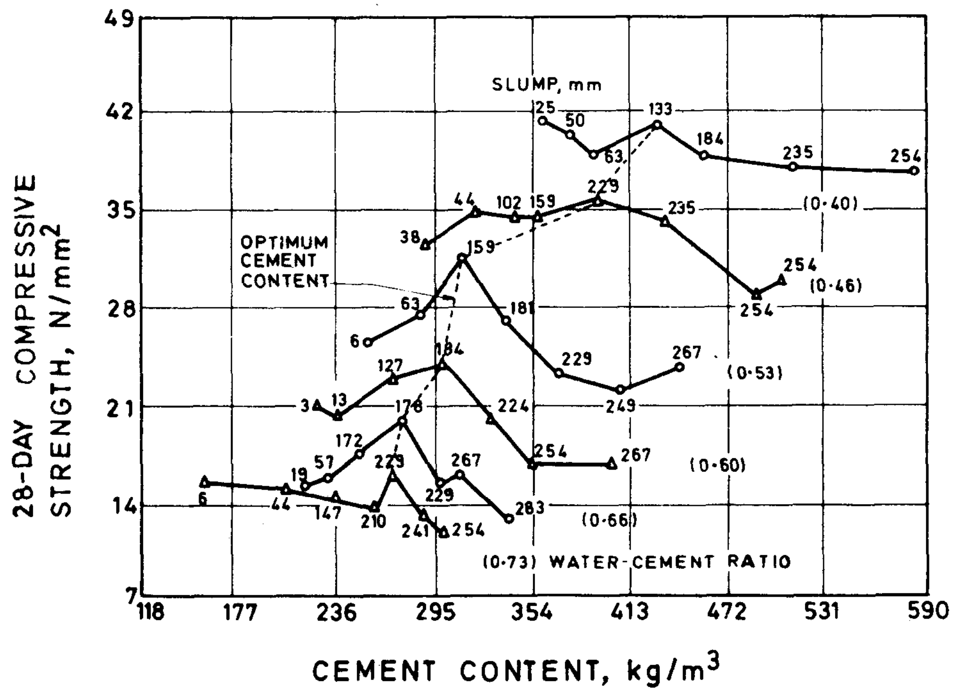


Fig. 24 Effect of Cement Content on the Compressive Strength of Concrete

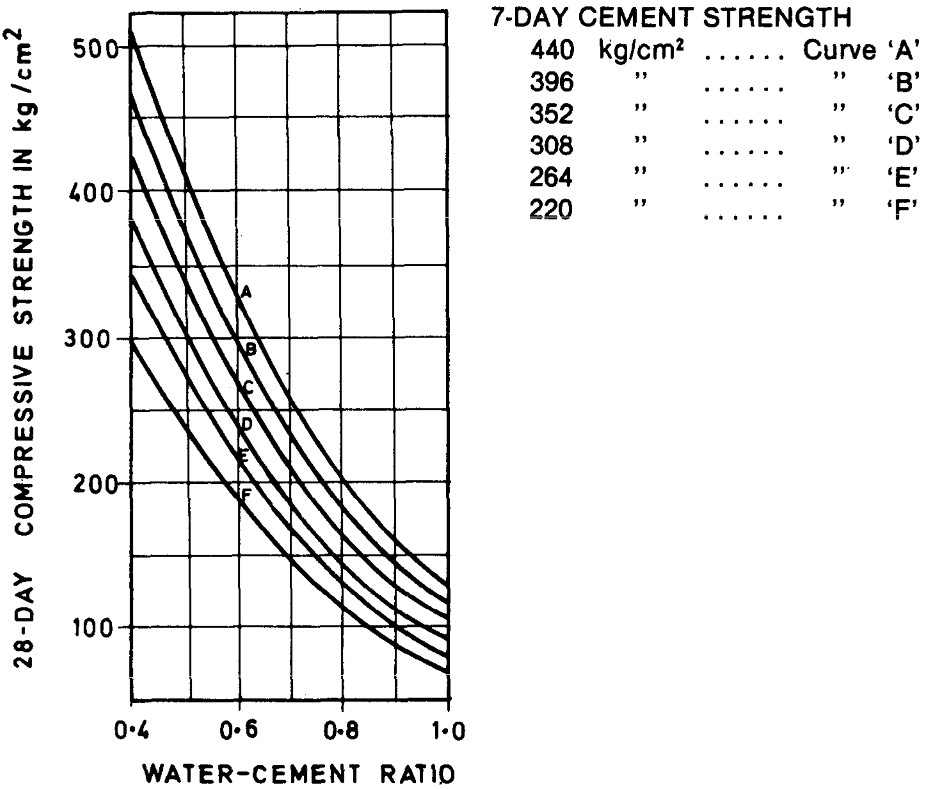


Fig. 25 Design Curve for Cement Concrete Mixes in Relation to 7-Day Compressive Strength of Cement

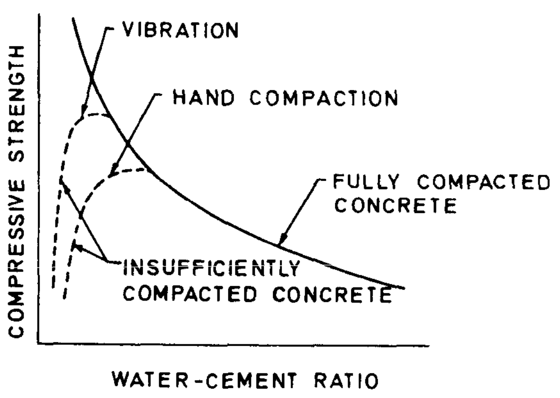


Fig. 26 Effect of Compaction on Compressive Strength of Concrete

NOTE — Figure 26 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

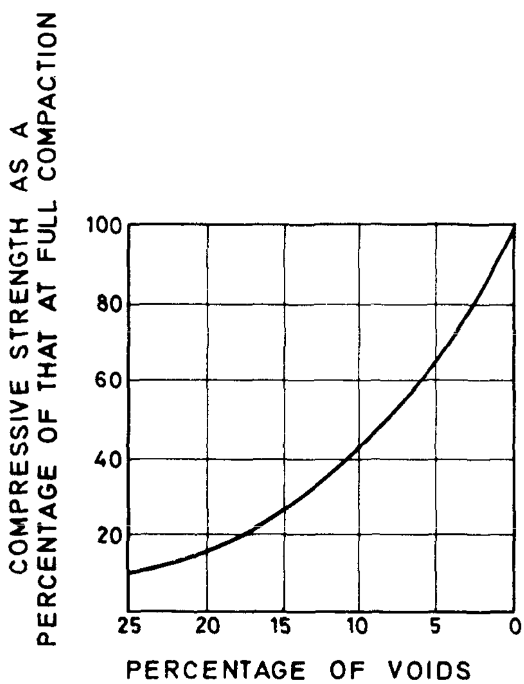


Fig. 27 Relationship Between Percentage of Voids and Compressive Strength of Concrete

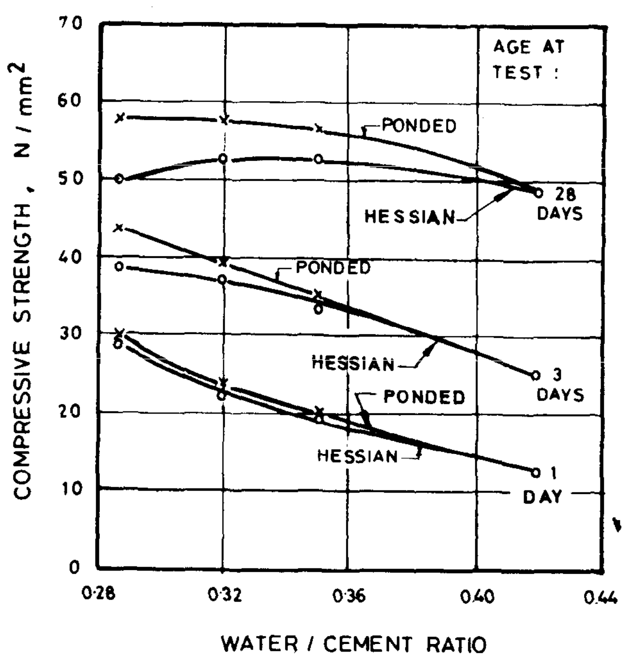


Fig. 28 Influence of Curing Conditions on Strength of Test Cylinders

NOTE — Figure 28 is from 'Properties of Concrete (Second Edition, 1973)' by A. M. Neville and published by Pitman Publishing Corporation, Sir Isaac Pitman and Sons Ltd, London.

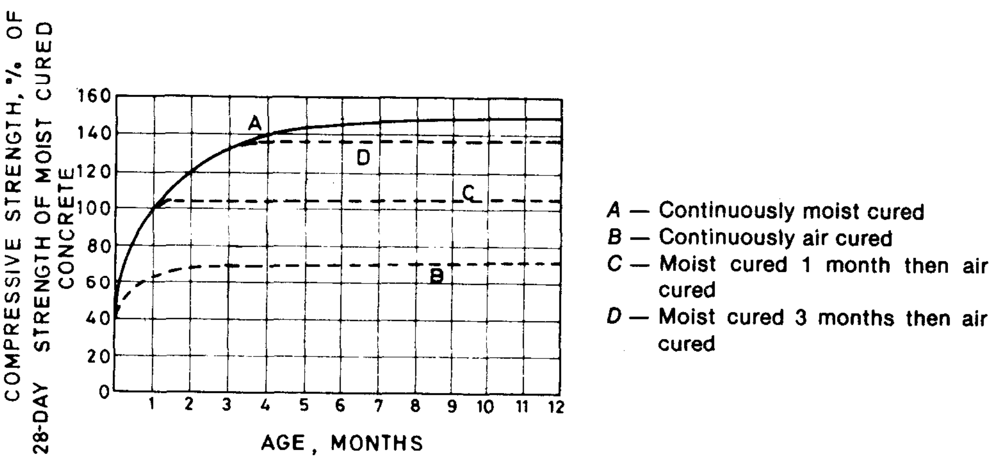
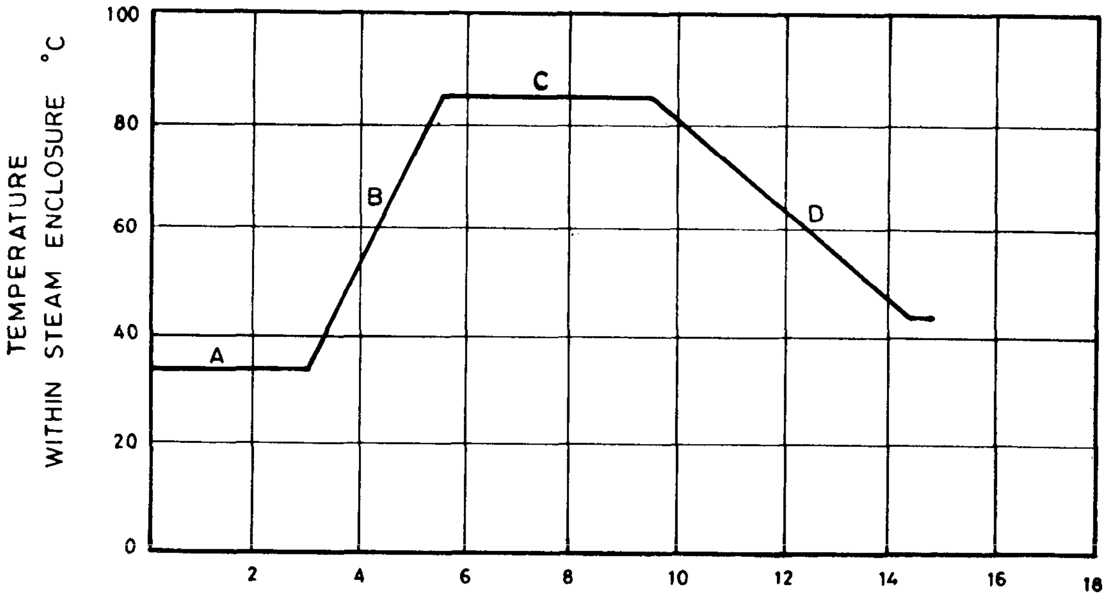


Fig. 29 Curing and Strength Relationship for Portland Cement Concrete

NOTE — Figure 29 is from 'Concrete Technology and Practice (Fourth Edition, 1977)' by W. H. Taylor and published by Mc Graw-Hill Book Company Australia Pty Limited, Sydney.



A — Presteaming period, 3 hours
B — Temperature-rise period, 2½ hours
C — Period at maximum temperature, 4 hours
D — Cooling period, 5 hours

Fig. 30 Typical Steam Curing Cycle

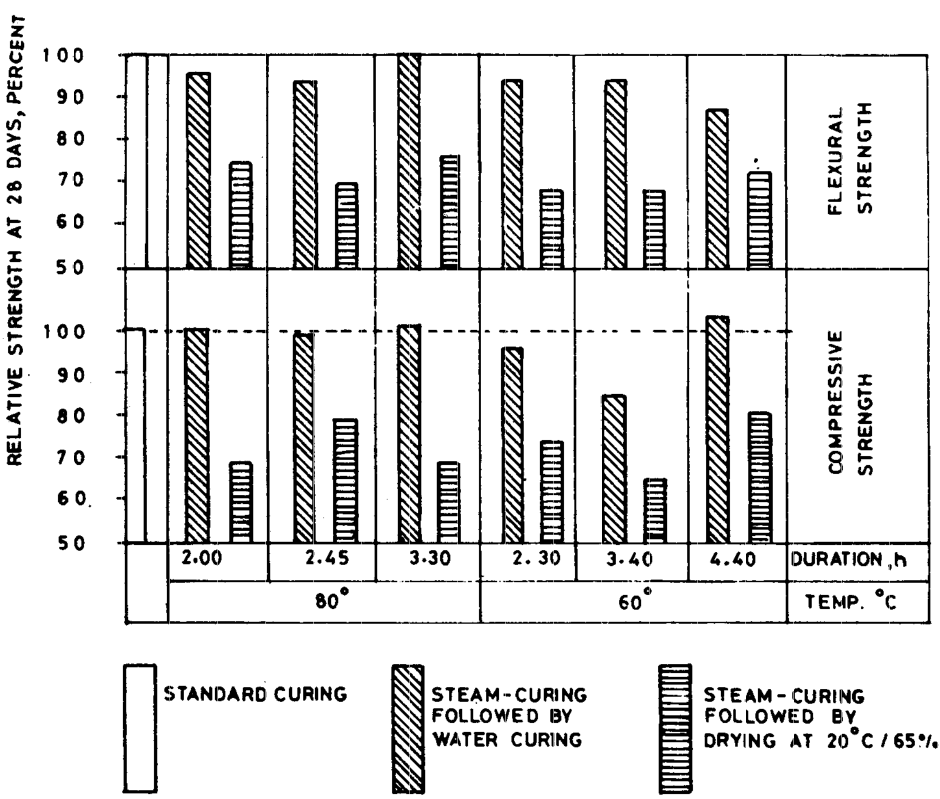


Fig. 31 Effect of Curing Regime on 28-Day Strength of Concrete

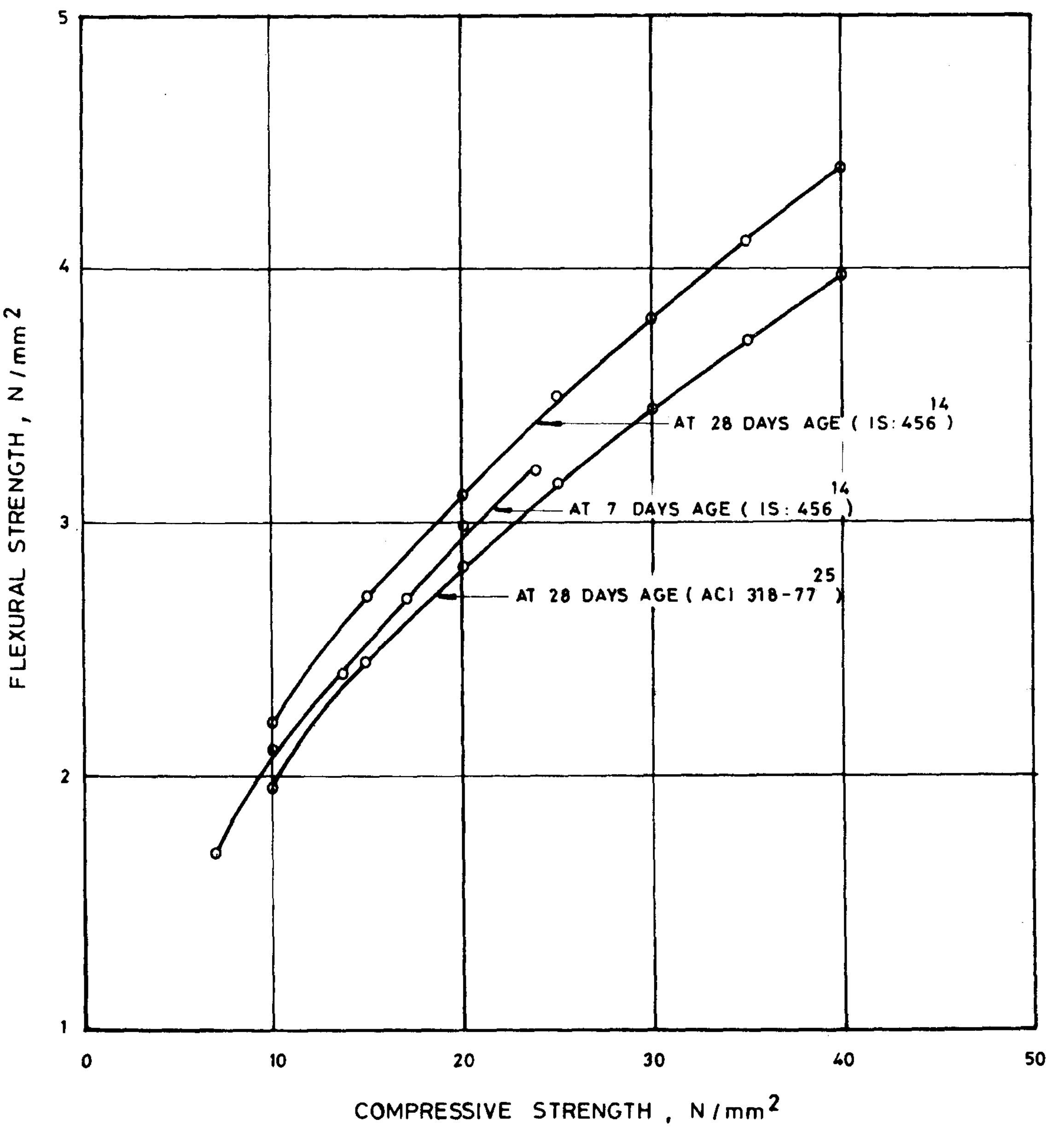


Fig. 32 Comparison of Relationships Between Compressive and Flexural Strengths of Concrete

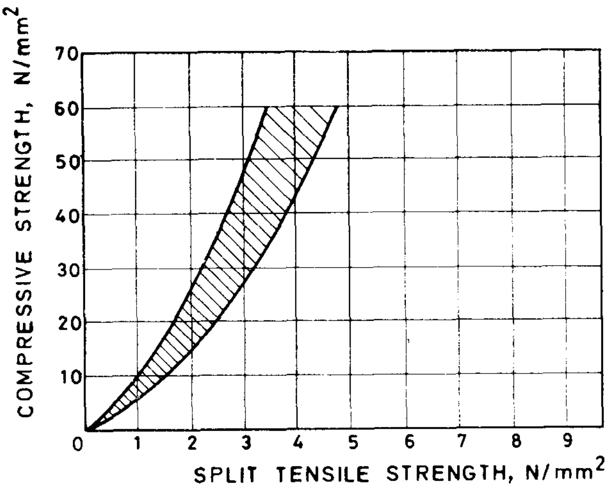


Fig. 33 Relationship Between Split Tensile Strength and Compressive Strength of Concrete

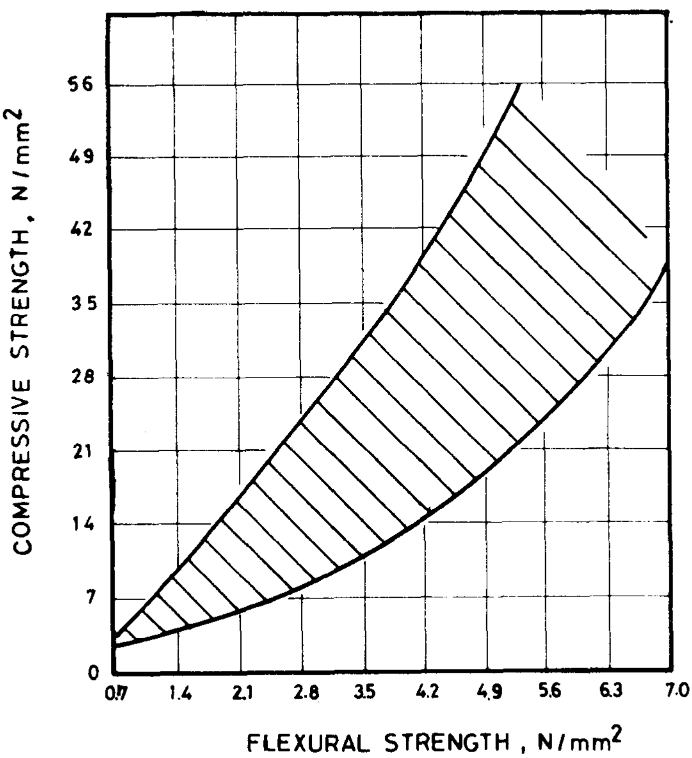


Fig. 34 Relationship Between Compressive and Flexural Strengths of Concrete

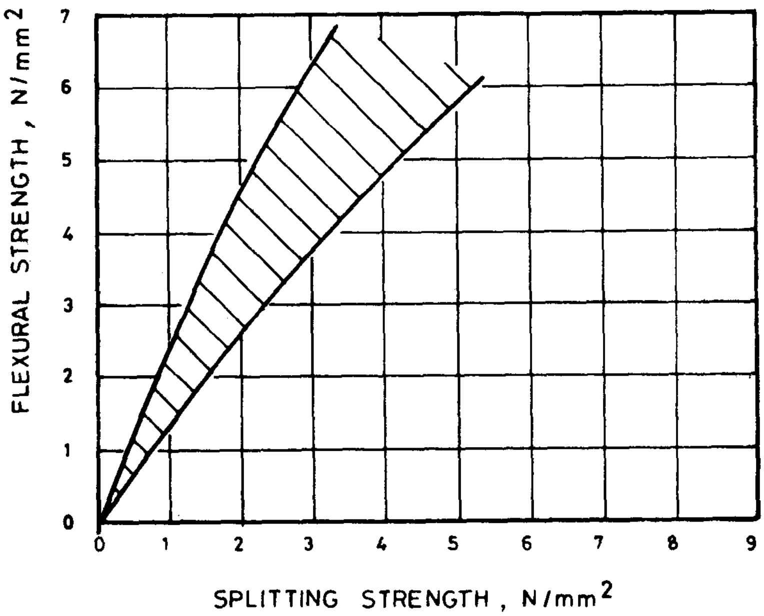


Fig. 35 Relationship Between Split Tensile Strength and Flexural Strength of Concrete

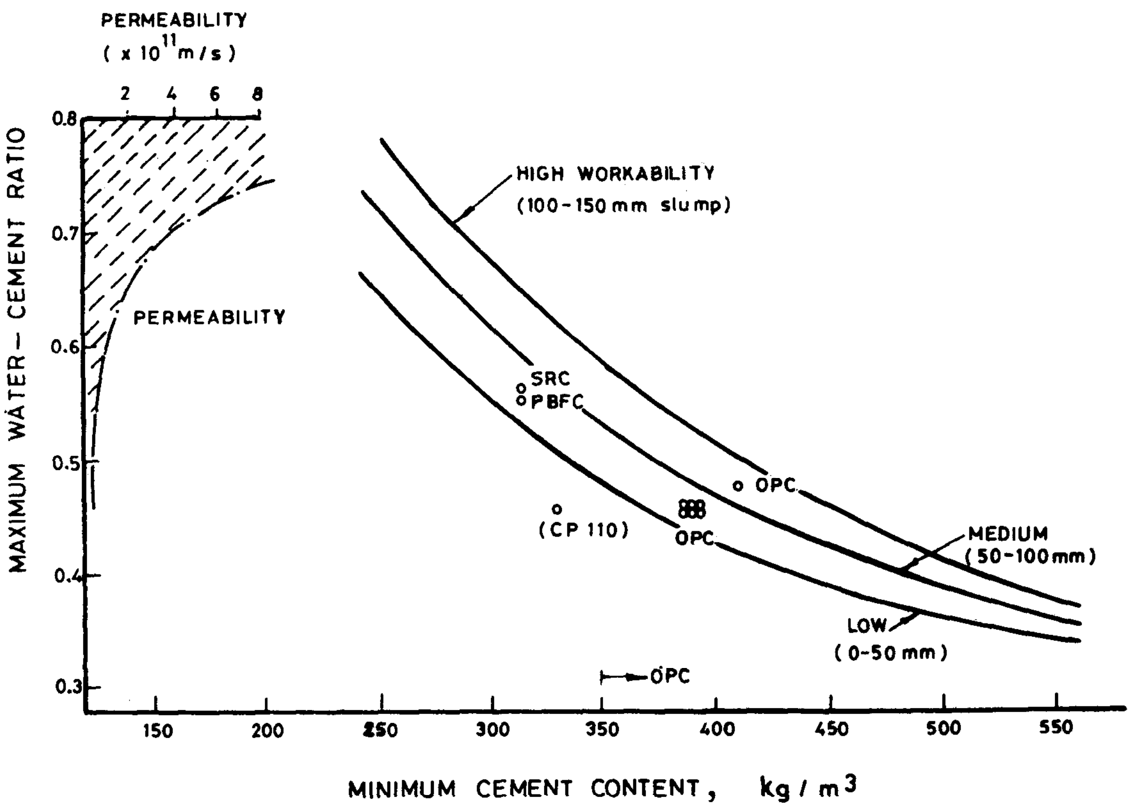


Fig. 36 Interrelationships of Water-Cement Ratio and Cement Content on Workability and Permeability of Fresh Concrete

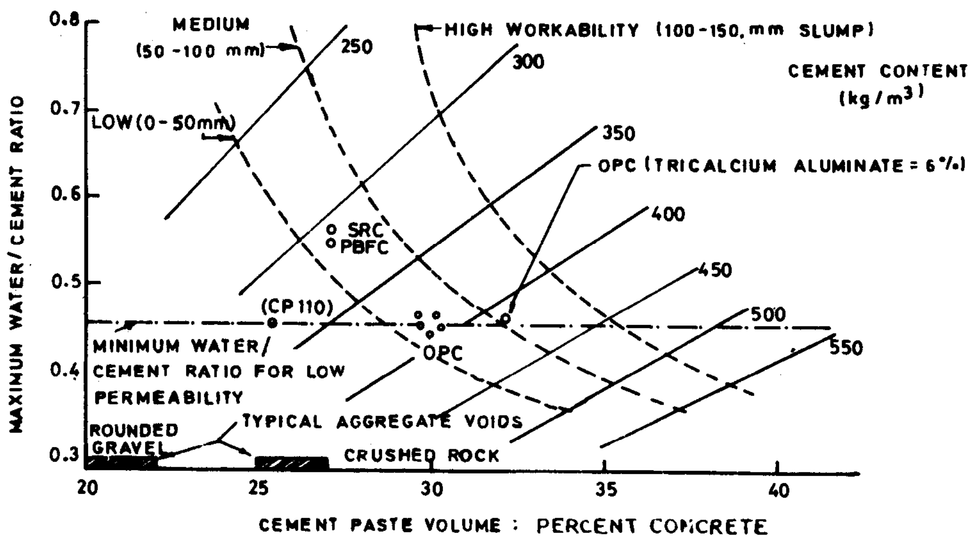


Fig. 37 Cement Paste Volume Required in Concrete of Various Water-Cement Ratios for Marine Durability

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SECTION 4

VARIABILITY OF CONCRETE STRENGTH

STATISTICAL ASPECTS

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SECTION 4 VARIABILITY OF CONCRETE STRENGTH — STATISTICAL ASPECTS

4.0 In this Section, information is presented on different methods of measuring the variability of concrete strength and the application of principles of statistics to concrete mix design. In addition, the acceptance criteria for concrete as specified in IS : 456-1978[1] have also been discussed.

4.1 Measures of Variabilities of Concrete Strength

4.1.1 FACTORS CONTRIBUTING TO VARIABILITY — It is found that the strength of concrete varies from batch to batch over a period of time. The sources of variability in the strength of concrete may be considered to be due to the following factors:

a) Variation in the quality of constituent materials used,
b) Variation in the mix proportions due to batching process,
c) Variations in the quality of batching and mixing equipment available,
d) The quality of supervision and workmanship, and
e) Variation due to sampling and testing of concrete specimens.

The above variations are inevitable during production to varying degrees. For example, cement from different batches may exhibit different strengths and the variability is more when cement from different sources is involved. The grading and shape of aggregates even from the same source varies widely and it is not economically feasible to eliminate such variations particularly when the aggregates are not factory made. Considerable variations occur in the mix proportions from batch to batch irrespective of whether the batching is by weight or volume. These can be attributed partly to the quality of plant available and partly due to the efficiency of operation.

Some of the variations in the test results are due to variations in the sampling, making, curing and testing the specimen even when carried out in terms of the relevant Indian Standard specifications.

The relative influence of some of the factors affecting the variability of concrete strength in terms of standard deviation is given in Table 28[2]. These values are based on conditions in UK and may not be directly applicable for conditions in India. For instance, the variability contributed by cement from one works or from different works in India may be higher than those given in Table 28. Similarly, batching of cement in bulk, servo-operated weighing systems is not common in India. Hence the values quoted should be taken as illustrative only. Even though the overall variability arising out of the above five factors may be reduced, such a reduction is not humanly/economically feasible beyond certain limits.

4.1.2 THE DISTRIBUTION OF RESULTS — Having accepted the inevitability of variations, it is necessary to understand the statistical concepts underlying such variations and provide for suitable safeguards in the mix design by way of margin.

It is now generally recognized that the variations in concrete strength follow the Normal or Gaussion distribution. When a large number of test results are plotted in the form of a histogram, the resultant curve approaches that of a Normal distribution curve (*see* Fig. 38). The area beneath the curve represents the total number of test results. The proportion of the results less than the specified value is represented by the area beneath the curve to the left hand side of a vertical line drawn through the specified value.

The normal distribution curve is symmetrical about its mean, has a precise mathematical equation and is completely specified by two parameters, namely, mean strength and standard deviation.

The mean strength is defined as the arithmetic mean of the set of actual test results.

The standard deviation (S) is a measure of the spread of the results and the formula for working out the standard deviation is given in IS : 456-1978[1].

Appendix A gives a sample calculation of standard deviation (S) for samples belonging to different groups and updating the S values. Alternatively, the values of S may be quickly arrived at by using a scientific calculator having standard deviation programme.

Figure 39 indicates the shape of the distribution curves for different degrees of control for a typical grade of concrete[3]. It may be appreciated that the value of S is minimum for very good control and progressively increases as the level of control decreases.

The correlation between the characteristic strength and standard deviation indicates that for a given degree of control, the standard deviation increases as the specified characteristic strength up to about 20 N/mm^2 and remains constant beyond this characteristic strength value. However, Table 29 (*see* IS : 456-1978[1]) gives specific values of assumed standard deviation for various grades of concrete.

Coefficient of variation is a non-dimensional parameter and is equal to the standard deviation (S) divided by the mean strength. However, its use is not envisaged in IS : 456-1978[1].

4.1.3 Characteristic Strength — As the cube test results follow the Normal distribution, there is always the probability that some results may fall below the specified strength. Recognising this factor, IS : 456-1978[1] has brought in the concept of characteristic strength. The term 'characteristic strength' means that value of the strength of the material below which not more than 5 percent of the test results are expected to fall.

4.1.4 Target Mean Strength — Considering the inherent variability of concrete strength during production, it is necessary to design the mix to have a target mean strength which is greater than the characteristic strength by a suitable margin.

$$f_t = f_{ck} + K.S$$

where

f_t = target mean strength,

f_{ck} = characteristic strength,

K = a constant, depending on the definition of characteristic strength and is derived from the mathematics of Normal distribution (*see* Table 30), and

S = standard deviation.

The value of K is equal to 1.65 (*see* IS : 456-1978[1]) where not more than 5 percent of the test results are expected to fall below the characteristic strength.

$$f_t = f_{ck} + 1.65\ S$$

4.2 Statistical Concepts in Concrete Mix Design — The design of the concrete mix shall be so done as to ensure that the target mean strength is realized during preliminary trial mix testing stage. The only parameter to be defined in arriving at the value of f_t is the value of standard deviation.

The value of standard deviation for such use shall be either an assumed value depending on the degree of control available or a value based on past data using the same plant, materials and standard of supervision. The later value, if available, is to be preferred.

Where the past data is not available during initial stages of preliminary trials, the values given in Table 29[1] may be used. As soon as adequate number of test results (*Min* 30) are available, the actual calculated value of standard deviation shall be used for revising the mix design.

4.3 Acceptance Criteria — IS : 456-1978[1] stipulates that random samples from fresh concrete shall be taken as specified in IS : 1199-1959[4] and cubes shall be made, cured and tested as described in IS : 516-1959[5]. If required for some other purposes, for example, to estimate the time when the formwork can be stripped, tests may be conducted at early ages also but the acceptance or otherwise is always on the basis of 28 days strength. The average of the strength of three specimens is the test strength of any sample. The acceptance criteria as given in IS : 456-1978[1] is reproduced below:

'*15.* ACCEPTANCE CRITERIA

15.1 The concrete shall be deemed to comply with the strength requirements if:

a) every sample has a test strength not less than the characteristic value; or
b) the strength of one or more samples though less than the characteristic value, is in each case not less than the greater of:
   1) the characteristic strength minus 1.35 times the standard deviation; and

2) 0.80 times the characteristic strength; and the average strength of all the samples is not less than the characteristic strength plus

$$\left[1.65 - \frac{1.65}{\sqrt{\text{number of samples}}}\right] \text{times the standard deviation}$$

15.2 The concrete shall be deemed not to comply with the strength requirements if:

a) the strength of any sample is less than the greater of:
   1) the characteristic strength minus 1.35 times the standard deviation; and
   2) 0.80 times the characteristic strength; or
b) the average strength of all the samples is less than the characteristic strength plus

$$\left[1.65 - \frac{3}{\sqrt{\text{number of samples}}}\right] \text{times the standard deviation}$$

15.3 Concrete which does not meet the strength requirements as specified in *15.1* but has a strength greater than that required by *15.2* may, at the discretion of the designer, be accepted as being structurally adequate without further testing.

15.4 If the concrete is deemed not to comply persuant to *15.2*, the structural adequacy of the parts affected shall be investigated and any consequential action as needed shall be taken.

15.5 Concrete of each grade shall be assessed separately.

15.6 Concrete shall be assessed daily for compliance.

15.7 Concrete is liable to be rejected if it is porous or honey-combed; its placing has been interrupted without providing a proper construction joint; the reinforcement has been displaced beyond the tolerances specified; or construction tolerances have not been met. However, the hardened concrete may be accepted after carrying out suitable remedial measures to the satisfaction of the engineer-in-charge.'

The main statistical features of the Acceptance Criteria are as under:

a) The 'minimum' strength is expressed by 'characteristic strength' below which certain test results are allowed to fall. Such 'low' results, which are inevitable to occur, are not regarded as 'failures' and the concrete in the structure that they represent should not be considered automatically for rejection.
b) Notwithstanding such statistical concept for minimum strength, the absolute value is also specified which is taken as 0.8 times the characteristic strength.
c) For the purposes of acceptance each sample is expected to be equal to or exceed the characteristic strength required.
d) Because of the random nature of strength, a sample may have strength lower than the characteristic strength although expectedly not lower than 3 times standard deviation below the mean, that is, 1.35 times the standard deviation below the characteristic strength but by the same logic, there should be other samples whose strength should have exceeded the characteristic strength. The average of all such samples should then be not less than the characteristic strength plus

   $$\left[1.65 - \frac{1.65}{\sqrt{n}}\right] \text{times the standard deviation}$$

   Note that the standard deviation of average of n samples is $\frac{1}{\sqrt{n}}$ times that of individual samples. When n is sufficiently large, this approaches the 'mean' or 'target' strength, as it should be.

   The acceptance is thus on the basis of the average of all samples tested till that time rather than on individual low results. Thus a larger set of test data is utilized in decision making.
e) However, when the average of n such samples is less than the characteristic strength plus

   $$\left[1.65 - \frac{3}{\sqrt{n}}\right] \text{times the standard deviation}$$

the concrete is deemed not to have complied with the acceptance criteria and the structural adequacy is to be investigated.

Illustrative Example:

In a construction work, concrete of grade M 20 (20 N/mm²) is to be used. The standard deviation for this grade of concrete has been established to be 4 N/mm². In the course of testing concrete cubes, the following results are obtained from a week's production (average strength of 3 specimens tested at 28 days in each case, expressed in N/mm²):

24.8, 27.0, 28.5, 23.6, 18.0, 21.6, 15.0 . . . . N/mm². Applying the criteria of IS : 456-1978, it may be noted as follows:

a) The first four results are straightway accepted, the sample strength being greater than the characteristic strength (20 N/mm²) in each case.

b) The 5th result of 18 N/mm² is less than the characteristic strength (20 N/mm²) and is compared with:

i) 0.8 × characteristic strength that is, 16 N/mm².

ii) $(20.0 - 1.35 \times 4.0)$ that is 14.6 N/mm².

Since 18 N/mm² is greater than 16 N/mm², we check the average strength of the samples, which is

$(24.8 + 27.0 + 28.5 + 23.6 + 18.0) \div 5 = 24.4$ N/mm², and

$$20.0 + \left[1.65 - \frac{1.65}{\sqrt{5}}\right] \times 4.0$$

$= 23.6$ N/mm².

Since the average of 5 samples is greater than 23.6 N/mm² the 5th sample is acceptable.

iii) The 6th result is also acceptable. The 7th one (15 N/mm²) is lower than 16.0 N/mm².

The average strength of all the seven samples is,

$(24.8 + 27.0 + 28.5 + 23.6 + 18.0 + 21.6 + 15.0) \div 7 = 22.6$ N/mm²

which is greater than

$$20.0 + \left[1.65 - \frac{3}{\sqrt{7}}\right] \times 4.0$$

$= 22.1$ N/mm².

The 7th sample thus does not comply with the requirement (but cannot be deemed not to have complied with the requirement) the acceptance will depend upon the discretion of the designer.

TABLE 28 BREAKDOWN OF STANDARD DEVIATION FOR COMPRESSIVE STRENGTH FOR DIFFERENT STANDARDS OF CONTROL

(*Clause* 4.1.1)

Item of Control	Standard Deviation for Mean Strengths of 35 N/mm² or More in N/mm²			
	Item Alone	Item Plus Testing	All Items	
			Cement (one works)	Cement (many works)
Cement from one works	3.05	3.65	—	—
Cement from many works	3.60	4.15	—	—
Batching:				
i) Cement in bulk, Servo-operated weighing	2.55	3.30	4.50	4.85
ii) Cement and aggregates weighed	3.80	4.30	5.25	5.60
iii) Cement weighed, aggregates by volume	4.55	4.95	5.80	6.15
iv) Cement and aggregates by volume	5.20	5.50	6.35	6.65
v) Cement and aggregates by volume in a continuous mixer	3.05	3.65	4.75	5.10
Testing	2.05	—	—	—

NOTE — Table 28 is from 'Concrete Constituents and Mix Proportions' (1974) by B. W. Shacklock and published by Eyre an Spottiswoode Publications Ltd., London.

TABLE 29 ASSUMED STANDARD DEVIATION

(*Clauses* 4.1.2 *and* 4.2)

Grade of Concrete	Assumed Standard Deviation N/mm^2
M 10	2.3
M 15	3.5
M 20	4.6
M 25	5.3
M 30	6.0
M 35	6.3
M 40	6.6

TABLE 30 VALUES OF *K*

(*Clause* 4.1.4)

Percentage of Results Below the Characteristic Strength	K
50	0
16	1.00
10	1.28
5	1.65
2.5	1.96
1.0	2.33
0.5	2.58
0.0	Infinity

APPENDIX A

(*Clause* 4.1.2)

SAMPLE CALCULATION OF STANDARD DEVIATION

The results below represent a series of test results for a given grade of concrete.

Sample Number	Concrete Strength (kgf/cm^2)	Sample Number	Concrete Strength (kgf/cm^2)
1	290	30	264
2	299	31	301
3	339	32	329
4	353	33	301
5	302	34	350
6	323	35	302
7	274	36	304
8	292	37	299
9	288	38	297
10	329	39	285
11	316	40	287
12	297	41	297
13	255	42	271
14	301	43	280
15	316	44	281
16	252	45	283
17	308	46	344
18	255	47	313
19	302	48	316
20	274	49	271
21	294	50	343
22	308	51	294
23	313	52	273
24	283	53	266
25	304	54	260
26	260	55	266
27	285	56	253
28	278	57	350
29	311	58	339

(Continued)

APPENDIX A — *Contd*

Sample Number	Concrete Strength (kgf/cm²)	Sample Number	Concrete Strength (kgf/cm²)
59	330	73	290
60	327	74	313
61	308	75	288
62	339	76	273
63	281	77	322
64	301	78	304
65	276	79	313
66	290	80	346
67	288	81	357
68	367	82	313
69	297	83	332
70	276	84	339
71	334	85	351
72	292	86	353

Group 1 (Samples 1 to 30)

$$n = 30$$
$$\Sigma x = 8\ 865$$
$$\Sigma x^2 = 2\ 637\ 609$$
$$\bar{x} = 295$$
$$\Sigma(x-\bar{x})^2 = 17\ 994.73$$
$$S = \sqrt{\frac{17\ 994.73}{30-1}} = 24.91 \text{ kgf/cm}^2$$

This is the standard deviation of the concrete produced to date.

Group 2 (Samples 31 to 56)

$$n = 26$$
$$\Sigma x = 7\ 666$$
$$\Sigma x^2 = 2\ 277\ 204$$
$$\bar{x} = 294.85$$
$$\Sigma(x-\bar{x})^2 = 16\ 913.38$$
$$S = \sqrt{\frac{16\ 913.38}{26-1}} = 26.01 \text{ kgf/cm}^2$$

Standard Deviation of Concrete Produced Up to End of Group 2

$$n = 30+26 = 56$$
$$\Sigma x = 8\ 865+7\ 666 = 16\ 531$$
$$\Sigma x^2 = 2\ 637\ 609 + 2\ 277\ 204 = 4\ 914\ 813$$
$$(n-1) = 56-1 = 55$$
$$\bar{x} = \frac{\Sigma x}{n} = 295.20$$
$$\Sigma(x-\bar{x})^2 = 34\ 920.84$$
$$S = \sqrt{\frac{34\ 920.84}{55}} = 25.20 \text{ kgf/cm}^2$$

Group 3 (Samples 57 to 86)

$$n = 30$$
$$\Sigma x = 9\ 389$$
$$\Sigma x^2 = 2\ 960\ 011$$
$$\bar{x} = 312.97$$
$$\Sigma(x-\bar{x})^2 = 21\ 566.97$$
$$S = \sqrt{\frac{21\ 566.97}{30-1}} = 27.27 \text{ kgf/cm}^2$$

Pooling of Standard Deviations

Now to obtain the standard deviation of the concrete class to date, it is necessary to pool the standard deviations from different groups.

Total sum of squares, that is,

$$\Sigma(x-\bar{x})^2 = 34\ 920.84+21\ 566.97 = 56\ 487.81$$
$$\Sigma(n-1) = 55+29 = 84$$
$$S^2 = \text{Total sum of squares/total } (n-1)$$
$$S = 25.93 \text{ kgf/cm}^2$$

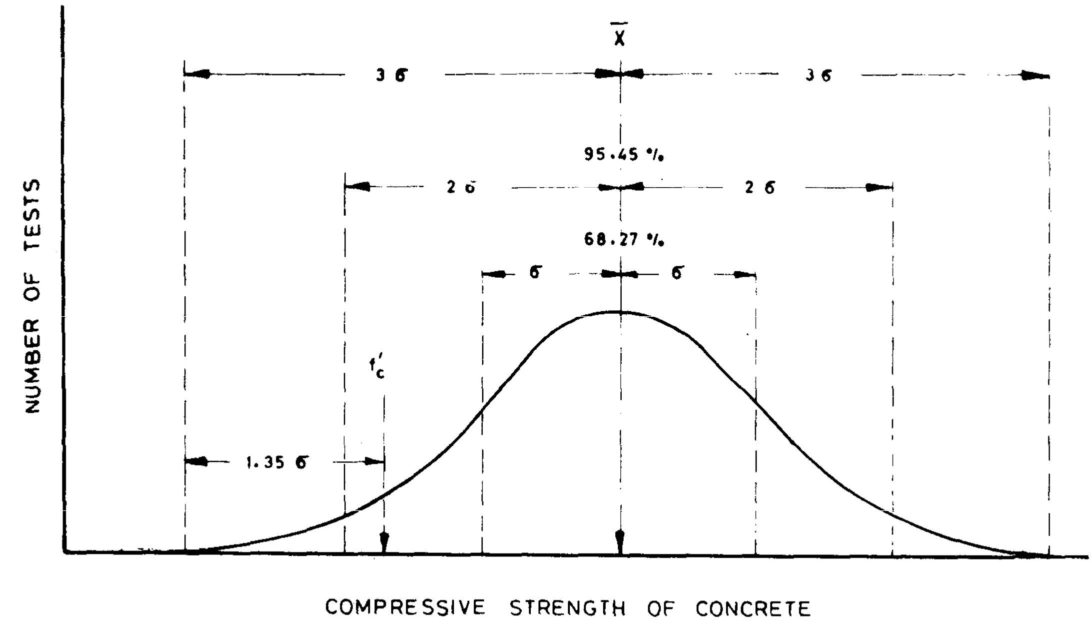

Fig. 38 Normal Distribution of Concrete Strengths

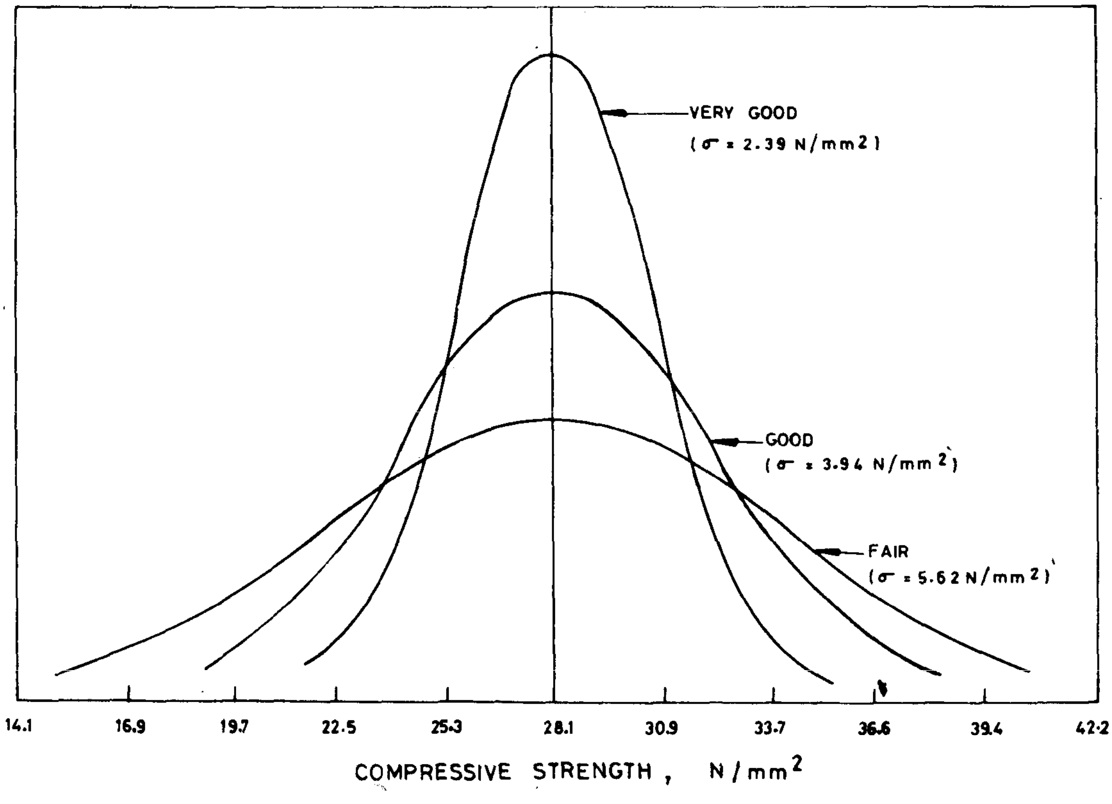

Fig. 39 Typical Normal Frequency Curves for Different Control Ratings

NOTE — Figure 39 is from 'Recommended Practice for Evaluation of Compression Test Results of Field Concrete' reported by ACI Committee 214 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

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SECTION 5

PRINCIPLES OF CONCRETE MIX DESIGN

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SECTION 5 PRINCIPLES OF CONCRETE MIX DESIGN

5.1 Basic Considerations — Design of concrete mixes involves determination of the proportions of the given constituents, namely, cement, water, coarse and fine aggregates and admixtures, if any, which would produce concrete possessing specified properties both in the fresh and hardened states with the maximum overall economy. Workability is specified as the important property of concrete in the fresh state; for hardened state compressive strength and durability are important. The mix design is, therefore, generally carried out for a particular compressive strength of concrete with adequate workability so that fresh concrete can be properly placed and compacted, and to achieve the required durability. In special situations, concrete can be designed for flexural strength or, for that matter, for any other specific property of concrete[1,2].

The proportioning of concrete mixes is accomplished by the use of certain relationships established from experimental data, which afford reasonably accurate guide to select the best combination of ingredients so as to achieve the desirable properties. The following basic assumptions are made in design of plastic concrete mixes of medium strength:

a) The compressive strength of concrete is governed by its water-cement ratio, and

b) For a given aggregate characteristics, the workability of concrete is governed by its water content.

For high strength concrete mixes of low workability, considerable interaction occurs between these two criteria and validity of such assumptions may become limited. Moreover, there are various other factors which affect the properties of concrete, for example, the quality and quantity of cement, water and aggregates; procedures of batching, mixing, placing, compaction and curing, etc. Therefore, the specific relationships that are used in proportioning concrete mixes should be considered only as a basis for trial mixes. Further modifications are necessary at the site based on the situation as well as specific materials available.

It is noted that a design in the strict sense of the word is not possible in relation to concrete mix proportioning. Concrete making materials are essentially variable and the two basic assumptions enumerated above may not be held to be quantitatively exact under all situations. If more accurate relationships between the proportions of materials and the properties of concrete (for example, relationship between compressive strength and water-cement ratio or water content and workability) are available, they should be used. Mix design on the basis of recommended guidelines is really a process of making an initial guess at the optimum combination of ingredients and final mix proportions is obtained only on the basis of further trial mixes.

5.2 Factors in the Choice of Mix Design — Both IS : 456-1978[3] as well as IS : 1343-1980[4] envisage that design of concrete mix be based on the following factors:

a) Grade designation,

b) Type of cement,

c) Maximum nominal size of aggregates,

d) Minimum water-cement ratio,

e) Workability, and

f) Minimum cement content.

Out of these, the grade designation gives the characteristic strength requirement of concrete. Depending upon the level of quality control available at the site, the concrete mix has to be designed for a target mean strength (*see 4.1.4*) somewhat higher than the characteristic strength.

The workability of concrete for satisfactory placing and compaction is related to the size and shape of the section to be concreted, the quantity and spacing of reinforcement, and the methods to be employed for transportation, placing and compaction of concrete. A guide to workability requirements for different conditions of placing is described in Section 3 (*see 3.1.3*).

The type of cement is important mainly through its influence on the rate of development of compressive strength of concrete as well as durability under aggressive environments. The different types of cements that can be used with the approval of the Engineer-in-Charge are discussed in *2.1*. From among the different types of cements available, the Engineer-in-Charge is required to make his choice depending upon the requirements of performance at hand. Where very high compressive strength is required, for example, in prestressed concrete railway sleepers, high strength ordinary Portland cement conforming to IS : 8112-1976[5] will be found suitable. Where an early strength development is required, rapid hardening Portland cement conforming to IS : 8041-1978[6] is preferable. On the other hand in situations where heat of hydration has to be limited, for example, in mass concrete constructions, low heat Portland cement conforming to IS : 269-1976[7] is preferable. Portland pozzolana cement and Portland slag cement are permitted for use in reinforced concrete constructions; while Portland slag cement is also permitted for prestressed concrete constructions. With such blended cements, the rate of development of early strength may be somewhat slower. On the other hand, these blended cements render greater durability to the concrete in sulphatic environment and sea water. The requirements of durability are achieved by limitations in terms of minimum cement content, the type of cement and the maximum water-cement ratio, as discussed in detail in Section 3.

The maximum nominal size of aggregates to be used in concrete is governed by the size of the section and spacing of the reinforcement. Both IS : 456-1978[3] and IS : 1343-1980[4], specify that the nominal maximum size of coarse aggregate should not be greater than one-fourth of the minimum thickness of the member, and it should be restricted to 5 mm less than the minimum clear distance between the main bars or 5 mm less than the minimum cover to the reinforcement and 5 mm less than the spacing between the cables, strands or sheathing in case of prestressed concrete. Within these limits, the nominal maximum size of coarse aggregates may be as large as possible. In general, it is found that larger the maximum size of aggregate, smaller is the cement requirement for a particular water-cement ratio (*see* Fig. 40[8]). This arises mainly from the fact that workability of concrete increases with increase in maximum size of aggregate. However, the maximum size of aggregates also influences the compressive strength of concrete in that, for a particular volume of aggregate, the compressive strength tends to increase with decrease in the size of coarse aggregate. This is due to the fact that smaller size aggregates present a larger surface area for bonding with the mortar matrix; it also results from the fact that the stress concentration in the mortar-aggregate interfaces increase with increase in the maximum size of aggregate[8]. There is thus an interaction of the maximum size of aggregate as well as the grade of concrete which determine the 'strength efficiency' of the cement and, therefore, the requirement of cement for a particular compressive strength is to be specified (Fig. 41[9]). From Fig. 41[9] it is seen that for concrete with higher water-cement ratio, larger maximum size of aggregates may be beneficial whereas for high strength concretes 10 or 20 mm size of aggregates is preferable. It is because of such reasons that IS : 456-1978[3] and IS : 1343-1980[4], while recommending that nominal size of coarse aggregates be as large as possible, also suggest that for reinforced and prestressed concrete works, aggregates having a maximum nominal size of 20 mm or smaller are generally considered satisfactory.

In appropriate circumstances, the maximum limit of cement content in the concrete may also have to be specified. This is because concrete mixes having high cement content may give rise to shrinkage, cracking and creep of concrete also increases with the cement paste content. In thick concrete sections restrained against movements, high cement content may give rise to excessive cracking caused by differential thermal stresses due to hydration of cement in young concretes. For high strength concretes, increasing cement content beyond a certain value, of the order of 550 kg/m^3 or so, may not increase the compressive strength. From these considerations as well as those of overall economy, the maximum cement content in the concrete mixes was limited to 530 kg/m^3 for prestressed concrete structures (*see* IS : 1343-1980[4]) and for reinforced concrete liquid retaining structures [*see* IS : 3370 (Part I)-1965[10]].

5.3 Outline of Mix Design Procedure — The various factors for determining the concrete mix proportions and the step by step procedure for concrete mix design can be schematically represented as in Fig. 42. The basic steps involved can be summarised as follows:

a) Arrive at the mean target strength from the characteristic strength specified and the level of quality control,
b) Choose the water-cement ratio for mean target strength and check for requirements of durability,
c) Arrive at the water content for the workability required,
d) Calculate cement content and check for requirements of durability,
e) Choose the relative proportion of the fine and coarse aggregates from the characteristics of coarse and fine aggregates,
f) Arrive at the concrete mix proportions for the first trial mix, and
g) Conduct trial mixes with suitable adjustments till the final mix composition is arrived at.

Most of the available mix design methods are essentially based on the above procedure, the details of the same are discussed in Section 6.

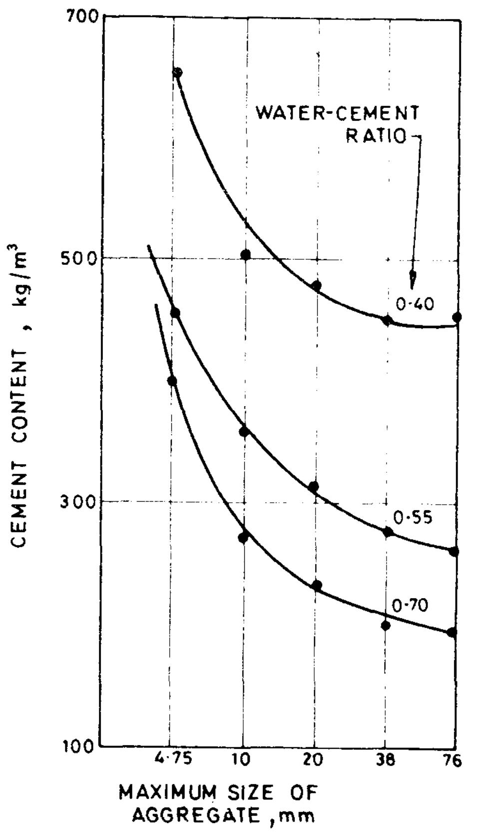

Fig. 40 Influence of Maximum Size of Aggregate on Cement Requirement of Concrete Mix

NOTE — Figure 40 is from 'Hardened Concrete: Physical and Mechanical Aspects: ACI Monograph No. 6' by A. M. Neville and published by American Concrete Institute.

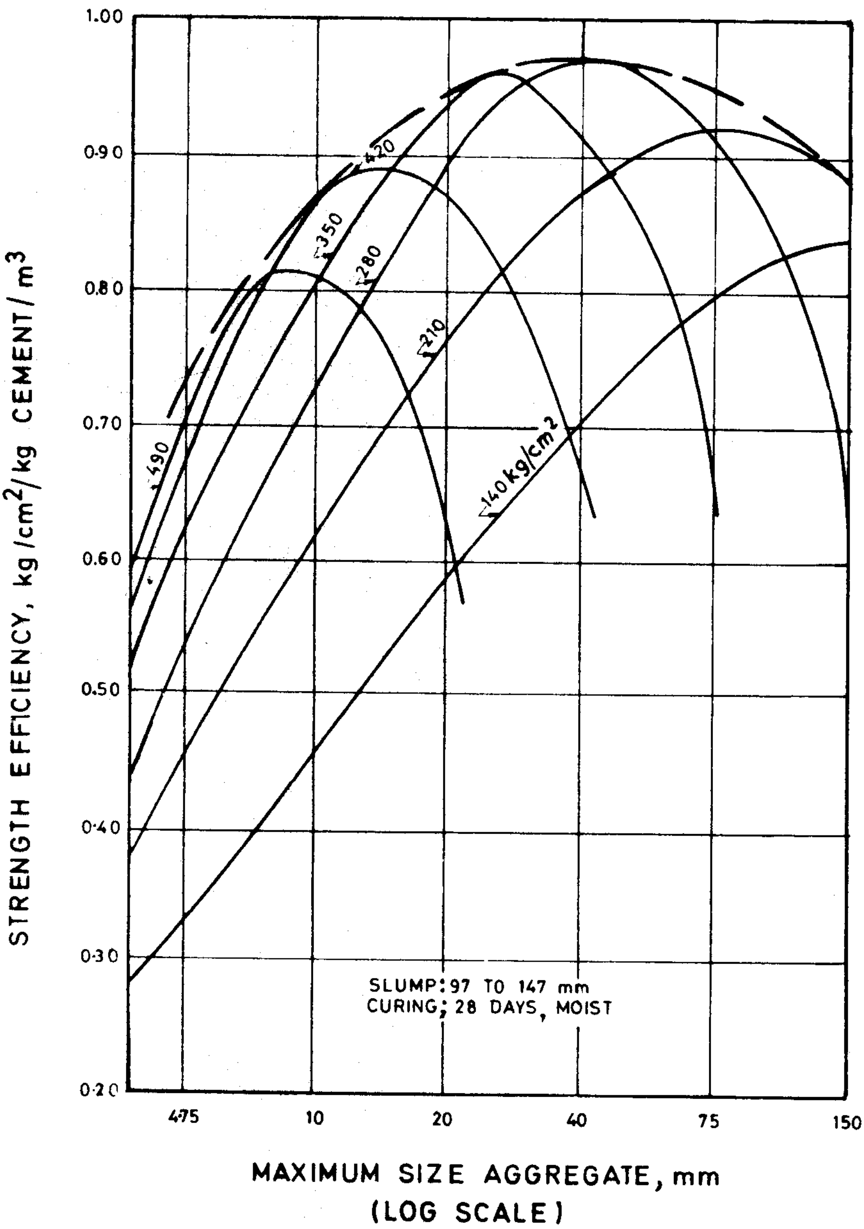

Fig. 41 Maximum Size Aggregate for Strength Efficiency Envelope

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SECTION 6

METHODS OF CONCRETE MIX DESIGN

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SECTION 6 METHODS OF CONCRETE MIX DESIGN

6.0 Introduction — The mix design methods being used in different countries are mostly based on empirical relationships, charts and graphs developed from extensive experimental investigations. Most of them follow the same basic principles enunciated in Section 5 and only minor variations exist in different mix design methods in the process of selecting the mix proportions. Some of the common mix design methods for medium and high strength concretes have been discussed in this section. They are: (a) The ACI Mix Design Method, (b) The USBR Mix Design Practice, (c) The British Mix Design Method and (d) The Mix Design Method according to Indian Standard Recommended Guidelines for Concrete Mix Design.

The ACI method[1] gives mix design for normal and heavy weight concrete (air-entrained and non-air-entrained) in the workability range of 25 to 100 mm slump, the maximum 28-day cylinder compressive strength being 450 kgf/cm². There is a separate method[2] for mix design of 'no slump' (slump being zero to 25 mm) concrete (air-entrained and non-air-entrained) having maximum 28-day cylinder compressive strength of 475 kgf/cm².

The British method[3] outlines a procedure for design of normal concrete mixes (superseding Road Note No. 4 method[4]) having 28-day cube compressive strength as high as 750 kgf/cm² for non-air-entrained concrete. The workability of concrete is given in terms of slump and Vee-Bee time.

In the USBR method[5], mix proportioning is done only for air-entrained concrete, the maximum 28-day cylinder compressive strength being 455 kgf/cm², when water reducing and set controlling admixtures are used.

In all the four methods, the water-cement ratio is chosen for the target mean strength from empirical strength — w/c ratio relationships and water content is chosen for the required workability for aggregates in saturated surface dry condition. In so far as the aggregate volume is concerned, the methods differ to some extent. In the ACI method[1], the volume of coarse aggregate in the concrete mix is first determined depending on the maximum size of aggregate and the grading of fine aggregate, whereas in the British method[3], the proportion of fine aggregate is determined first depending on the maximum size of aggregate, the degree of workability, the grading of fine aggregate and the water-cement ratio of the concrete mix.

In USBR method, the proportion of dry-rodded coarse aggregate is determined corresponding to the maximum size of aggregate, a fixed fineness modulus of sand and a fixed workability in terms of slump. The ACI method also determines the proportions of dry-rodded coarse aggregate in the concrete mix, the rodding being done according to ASTM C 29[6] for unit weight of aggregate. It is based on the concept that in dry-rodded void content, the differences in the amount of mortar required for workability with different aggregates due to differences in particle shape and grading are automatically compensated for.

The latest British mix design method[3] does not consider the combined aggregate grading curves like those used in Reference 4 (for maximum sizes of aggregate of 40 mm and 20 mm) and those developed for 10 mm maximum size of aggregate by Mcintosh and Erntroy[7]. This implies admission to the use of aggregates of any grading as long as they are within the grading limits specified by the appropriate Codes/Specifications.

In the ACI and USBR methods, the air content of concrete is considered to arrive at the absolute volume of the mix ingredients. The batch weight of the materials per unit volume of concrete is calculated from the absolute volumes. In the British method, the quantities of the ingredients are calculated directly from the wet density of concrete which is dependent on specific gravity of the combined aggregates (on saturated surface-dry condition).

The 'Indian Standard recommended guidelines for mix design[8]' includes design of normal concrete mixes (non-air-entrained), both for medium and high strength concrete. In this method of mix design, the water content and proportion of fine aggregate corresponding to a maximum size of aggregate are first determined for reference values of workability, water-cement ratio and grading of fine aggregate. The water content and the proportion of fine aggregate are then adjusted for any difference in workability, water-cement ratio and grading of fine aggregate in any particular case from the reference values. The batch weight of materials per unit volume of concrete is finally calculated by the absolute volume method. The specific relationships (Figures and Tables) that are given in this method of mix design, have been arrived at by exhaustive tests at the Cement Research Institute of India[9,10] as well as on the basis of data on concrete being designed and produced in the country[11].

These guidelines, although based on data on concrete, majority of which were made with OPC, are also almost equally applicable to concretes made with PPC. The final mix proportions, selected after trial mixes, may entail some minor changes in each case; such variations may also be necessary in case of cements of one type (either OPC or PPC) but from different sources, or aggregates varying in quality. In so far as selection of water-cement ratio for the target compressive strength at 28-day in concerned, Fig. 46 is applicable for both ordinary portland and portland pozzolana cements with comparable validity. However, if a more precise estimate is made with the help of Fig. 47 where cements are classified on the basis of their 28-day strengths, then use of OPC or PPC is not expected to make much difference.

Experiences with fly-ash-cement concretes indicate that in such cases, for comparable workabilities, the water content can be reduced by about 3 to 5 percent and proportion of fine aggregates reduced by 2 to 4 percentage points. It is doubtful whether such generalization can be straightway extended in case of concretes made with PPC also, but any difference that would be necessary can be easily established by trials with the materials at hand.

On the other hand, the British and American methods mentioned earlier, sound as they are in principle, are based on the experience, materials and construction techniques prevalent in those countries and the Tables and Charts may need some modifications before they become applicable to Indian conditions. In its approach, the IS method is similar to USBR method as well as the method specified in IRC : 44-1976[12] mix design for concrete pavements. In the IRC method, 7 days compressive strength of cement has been considered as an additional parameter influencing the relationship between water-cement ratio and the 28 days compressive strength of concrete. The necessary curves are shown in Fig. 25 of Section 3.

6.1 The ACI Mix Design Practice — The ACI 211.1-77[1] recommends a method of mix design in which the water content determines the workability of the concrete mix for different maximum size of aggregate. The bulk volume of coarse aggregate per unit volume of concrete is determined for different maximum sizes of aggregate and for different fineness moduli of sand. The water-cement ratio is determined in the usual procedure to satisfy both strength and durability requirements. The volume of fine aggregate is determined for unit volume of concrete, from the difference in volume between the concrete and other ingredients. Allowance for air content in concrete is made prior to calculating the volume of fine aggregate. The procedure adopted for the selection of mix proportions is as follows:

a) The water-cement ratio is selected from Table 31 for the target mean 28-day compressive strength.
b) The water content is selected from Table 32 for the desired workability and maximum size of aggregate.
c) The cement content is calculated from the water content and the water-cement ratio required for durability or strength.
d) The coarse aggregate content is estimated from Table 33 for the maximum size of aggregate and the fineness modulus of sand.
e) The fine aggregate content is determined by subtracting the sum of the volumes of coarse aggregate, cement, water and air content from the unit volume of concrete.

For stiffer concrete mixes 'ACI 211-65[2] Recommended practice for selecting proportions for no-slump concrete' is to be followed. This is an extension of the ACI standard

211.1-1977[1] with two differences: (a) the measurement of workability is done by compacting factor, Vee-Bee consistency or drop table test, instead of slump test, and (b) the coarse aggregate content is higher for more workable mixes. Thus the tables for water requirement for different degrees of workability and coarse aggregate volume per unit volume of concrete are changed. The rest of the mix design procedure is unaltered.

6.2 The USBR Mix Design Practice — In this method of mix design, the water content of air-entrained concrete and the proportions of fine and coarse aggregates are determined for a fixed workability and grading of fine aggregate. The water content and percentages of sand or coarse aggregate are adjusted for changes in the materials and mix proportions. The water-cement ratio for compressive strength is determined in the usual procedure. The step-by-step procedure of mix proportioning is as follows:

a) The water-cement ratio for the target mean 28 day compressive strength of concrete is determined from Table 34. This is for either air-entrained concrete or for air-entrained concrete with water-reducing, set-controlling admixtures.

b) Approximate air and water contents and the percentages of sand and coarse aggregate per cubic metre of concrete are determined from Table 35, for concrete containing natural sand with a fineness modulus of 2.75 and having workability of 75 to 100 mm slump.

c) Adjustment of values in water content and percentages of sand or coarse aggregate are made as provided in Table 36 for changes in the fineness modulus of sand, slump of concrete, air content, water-cement ratio and sand content other than the reference values in Table 35.

d) The cement content is calculated using the selected water-cement ratio and the final water content of the mix is arrived after adjustment.

e) Proportions of aggregates are determined by estimating the quantity of coarse aggregate from Table 35 (dry-rodded unit weight coarse aggregate method) or by computing the total solid volume of sand and coarse aggregate in the concrete mix and multiplying the final percentage of sand after adjustment. Either method is satisfactory and will produce approximately the same proportions under average conditions.

6.3 The British Mix Design Method (DOE Method) — The latest method[3] replaces the traditional British mix design method[4] of Road Note No. 4. It discards the use of specific grading curves of the combined aggregates, uses the relationship between water-cement ratio and compressive strength of concrete depending on the type of cement and type of aggregates used. It replaces the mix design Tables correlating water-cement ratio, aggregate-cement ratio, maximum size of aggregate, type of aggregate differing in shapes (rounded and irregular), degree of workability and overall grading curves of the combined aggregates in earlier Road Note No. 4. Instead, water content required to give various levels of workability is determined for two types of aggregates, namely, crushed and uncrushed.

The degree of workability 'very low', 'low', 'medium' and 'high' have now been referred in terms of specific values of slump and Vee-Bee time. The method of mix design results in expressing the mix proportions in terms of quantities of materials per unit volume of concrete in line with European and American practice. The procedure of mix proportioning is as follows:

a) The water-cement ratio for target mean compressive strength is determined using Table 37 and Figure 43 and compared with the maximum water-cement ratio specified for durability and the lower of these two values used.

b) The water content depending upon the type and maximum size of aggregate to give a concrete of the specified slump or Vee-Bee time is selected from Table 38.

c) The cement content is calculated from the water-cement ratio and water content of the mix.

d) The total aggregate content (saturated and surface-dry) is determined by subtracting the cement and water content from the wet density of concrete, the wet density being obtained from Fig. 44 depending upon the water content and the relative density of the combined aggregate.

e) Finally, the proportions of fine and coarse aggregates are determined from Fig. 45 depending on the water-cement ratio, the maximum size of aggregate, the workability level and the grading zone of the fine aggregate.

6.4 Mix Design in Accordance with Indian Standard Recommended Guidelines for Concrete Mix Design[8] — The following basic data are required to be specified for design of a concrete mix:

a) Characteristic compressive strength (that is, below which only a specified proportion of test results are allowed to fall) at 28 days (f_{ck});
b) Degree of workability desired (for guidance *see* Table 22);
c) Limitations on the water-cement ratio and the minimum cement content to ensure adequate durability for the type of exposure (*see* Tables 23 to 26);
d) Type and maximum size of aggregate to be used;
e) Standard deviation (S) for compressive strength of concrete: The standard deviation has to be calculated from the results of tests as described in *4.1.2*. When the results of sufficient number of tests under site conditions and for the grade of concrete are not available, the values of standard deviation for different degree of control as given in Table 39 may be adopted. The degree of quality control expected depends upon a number of parameters and it is necessary that appropriate values from Table 39 are chosen. Table 40 provides guidance regarding the degree of quality control to be expected, depending upon the infrastructure and practices adopted at the construction site. This can be used to characterise the level of quality control in the particular situation for using Table 39.

The step-by-step procedure of mix proportioning is as follows:

a) The target mean strength is first determined as follows:

$$f_t = f_{ck} + K.S. \qquad (1)$$

where f_t = target mean compressive strength at 28 days,

f_{ck} = characteristic compressive strength at 28 days,

S = standard deviation,

K = a statistical value depending upon the accepted proportion of low results and the number of tests (*see* Table 30).

NOTE — As per IS : 456-1978, the characteristic strength is defined as that value below which not more than 5 percent of the test results are expected to fall. In such case, $K = 1.65$ in equation (1).

b) The water-cement ratio for the target mean strength is chosen from Fig. 46. The water-cement ratio so chosen is checked against the limiting water-cement ratio for the requirements of durability (Tables 23 to 26) and the lower of the two values adopted. Fig. 46 is based on a large number of results under Indian conditions, but on a given situation, may need slight modifications depending upon the characteristics of cement available[14]. As such, it is used more as a guide and actual water-cement ratio is determined by means of trial mixes as described in Ref 8. A more precise estimate of the preliminary water-cement ratio corresponding to the target average strength may be made from the relationships shown in Fig. 47, using the curves corresponding to the 28-day compressive strength of cement. It is to be noted that cements have been characterised by its 28-day strength in Fig. 47 rather than upon its 7-day strength (*see* Fig. 25) because 28-day strength of concrete is found to be better related to the 28-day strength of cement rather than at earlier ages, more so for blended cements. The relationship in Fig. 46 is really a mean curve through Fig. 47.

However, such trials will need 28 days for determining the strength characteristics of cement and atleast another 28 days for the trial mixes. In order to cut down the time required for trials, an alternative method has been suggested in Ref 8.

In this method, the accelerated strength (boiling water method in accordance with IS : 9013-1978[15]) of a 'reference' concrete mix having water-cement ratio 0.35 and workability of 0.80 compacting factor with the cement proposed to be used is determined on 15 cm cube specimens. The nominal maximum size of aggregate of the 'reference' concrete should be 10 mm and fine aggregate should conform to Zone II of Table 4 of IS : 383-1970[13]. Corresponding to this accelerated strength, the water-cement ratio is determined for the target mean strength, from Fig. 48. These curves are based on the relation between 28-day compressive strength of concrete having water-cement ratio of 0.35, which is found to be, on an average, 0.934 times that of 28-day strength of cement tested as per IS : 4031-1968[16], and correlation of accelerated and normal 28-day strength of concrete (*see* Section 8[14]).

c) The air content (amount of entrapped air) is estimated from Table 41 for the maximum size of aggregate used.

d) The water content and percentage of sand in total aggregate by absolute volume are next selected from Tables 42 and 43 for medium and high strength concretes, respectively, for the following standard reference conditions:

   i) Crushed (angular) coarse aggregate,

   ii) Fine aggregate consisting of natural sand conforming to grading zone II of Table 4, IS : 383-1970[13], in saturated surface dry condition,

   iii) Water-cement ratio of 0.60 and 0.35 for medium and high strength concretes respectively, and

   iv) Workability corresponding to compacting factor of 0.80.

e) For other conditions of workability, water-cement ratio, grading of fine aggregate and for rounded aggregates, adjustments in water content and percentage of sand in total aggregate are made as per Table 44.

f) The cement content is calculated from the water-cement ratio and the final water content arrived after adjustment. The cement content so calculated is checked against the minimum cement content from the requirements of durability (Tables 23 to 26) and the greater of the two values adopted.

g) With the quantities of water and cement per unit volume of concrete and the percentage of sand in the total aggregate already determined, the coarse and fine aggregates content per unit volume of concrete are calculated from the following equations:

$$V = \left[W + \frac{C}{S_c} + \frac{1}{p} \cdot \frac{f_a}{S_{fa}}\right] \times \frac{1}{1000} \qquad \text{(2) and}$$

$$V = \left[W + \frac{C}{S_c} + \frac{1}{1-p} \cdot \frac{C_a}{S_{ca}}\right] \times \frac{1}{1000} \qquad (3)$$

where,

V = absolute volume of fresh concrete
= gross volume (1 m^3) minus the volume of entrapped air,

S_c = specific gravity of cement,

W = mass of water (kg) per m^3 of concrete,

C = mass of cement (kg) per m^3 of concrete,

p = ratio of fine aggregate to total aggregate by absolute volume,

f_a, C_a = total masses of fine aggregate and coarse aggregate, (kg) per m^3 of concrete respectively, and

S_{fa}, S_{ca} = specific gravities of saturated surface dry fine aggregate and coarse aggregate respectively.

An illustrative example of mix design is reproduced from Reference 8. The actual mix proportions are arrived at by means of a number of trial mixes. In order to account for the variability in results of laboratory trials, it is advisable to carry out a number of tests with the final mix proportions arrived at after such trial mixes. In addition, necessary adjustments in mix proportions should also be carried out depending upon the results obtained under actual constructions.

ILLUSTRATIVE EXAMPLE ON CONCRETE MIX DESIGN (Grade M 20)

a) *Design Stipulations*

1)	Characteristic compressive strength required in the field at 28-days	20 N/mm²
2)	Maximum size of aggregate	20mm (angular)
3)	Degree of workability	0.90 compacting factor
4)	Degree of quality control, and	Good
5)	Type of exposure	Mild

b) *Test Data for Materials*

1)	Cement used-ordinary Portland cement satisfying the requirements of IS : 269-1976[17]	
2)	Specific gravity of cement	3.15
3)	i) Specific gravity of coarse aggregates	2.60
	ii) Specific gravity of fine aggregate	2.60
4)	Water absorption	
	i) Coarse aggregate	0.5 percent
	ii) Fine aggregate	1.0 percent
5)	Free (surface) moisture	
	i) Coarse aggregate	Nil (absorbed moisture also nil)
	ii) Fine aggregate	2.0 percent

6) Sieve analysis
 i) Coarse aggregate
 ii) Fine Aggregate

IS Sieve Size	*Fine Aggregate (Percent Passing)*	*Remarks*
4.75 mm	100	Conforming to grading
2.36 mm	100	Zone III of Table 4 of
1.18 mm	93	IS:383-1970[13]
600 micron	60	
300 micron	12	
150 micron	2	

c) *Target Mean Strength of Concrete* — For a tolerance factor of 1.65 and using Table 39, the target mean strength for the specified characteristic cube strength is $20+4.6\times1.65=27.6$ N/mm².

d) *Selection of Water-Cement Ratio* — From Fig. 46, the water-cement ratio required for the target mean strength of 27.6 N/mm² is 0.50. This is lower than the maximum value of 0.65 prescribed for 'Mild' exposure (*see* Table 23).

e) *Selection of Water and Sand Content* — From Table 42, for 20 mm nominal maximum size aggregate and sand conforming to grading Zone II, water content per cubic meter of concrete is equal to 186 kg and sand content as percentage of total aggregate by absolute volume is equal to 35 percent.

For change in values in water-cement ratio, compacting factor and sand

IS Sieve Size (mm)	*Analysis of Coarse Aggregate Fractions (Percent Passing)* I	II	*Percentage of Different Fractions* I 60 Percent	II 40 Percent	Combined 100 Percent	*Remarks*
20	100	100	60	40	100	Conforming to
10	0	71.20	0	28.5	28.5	Table 2 of
4.75	—	9.40	—	3.7	3.7	IS : 383-1970[13]
2.36	—	0	—	—	—	

belonging to Zone III, the following adjustment is required:

Change in Condition (*see* Table 44)	*Adjustment Required in* Water content percent	Percentage sand in total aggregate
For decrease in water-cement ratio by (0.60 – 0.50) that is, 0.1	0	– 2.0
For increase in compacting factor (0.9 – 0.8) that is, 0.10	+ 3	0
For sand conforming to Zone III of Table 4 of IS:383-1970[13]	0	– 1.5
Total	+ 3	– 3.5

Therefore required sand content as percentage of total aggregate by absolute volume = 35 – 3.5 = 31.5 percent.

Required water content = 186 + 5.58 = 191.6 litres/m³.

f) *Determination of Cement Content*

water-cement ratio = 0.50

water = 191.6 litres

$$\text{cement} = \frac{191.6}{0.50} = 383 \text{ kg/m}^3$$

This cement content is adequate for 'mild' exposure condition (*see* Table 23).

g) ***Determination of Coarse and Fine Aggregate Content*** — From Table 41, for the specified maximum size of aggregate of 20 mm, the amount of entrapped air in the wet concrete is 2 percent. Taking this into account and applying equations 2 & 3,

$$0.98 \text{ m}^3 = \left[191.6 + \frac{383}{3.15} + \frac{1}{0.315} \cdot \frac{f_a}{2.60}\right] \times \frac{1}{1\,000}$$

and

$$0.98 \text{ m}^3 = \left[191.6 + \frac{383}{3.15} + \frac{1}{0.685} \cdot \frac{C_a}{2.60}\right] \times \frac{1}{1\,000}$$

$f_a = 546$ kg/m³, and
$C_a = 1\,187$ kg/m³

The mix proportion then becomes:

Water	*Cement*	*Fine Aggregate*	*Coarse Aggregate*
191.6 litres	383 kg	546 kg	1 187 kg
0.50 :	1 :	1.42 :	3.09

h) *Actual Quantities Required for the Mix per Bag of Cement* — The mix is, 0.50:1:1.42:3.09 (by mass). For 50 kg of cement, the quantity of materials are worked out as below:

1) Cement = 50 kg
2) Sand = 71.0 kg
3) Coarse Aggregate = 154.5 kg
   (Fraction I = 92.7 kg,
   Fraction II = 61.8 kg).

4) Water:
   i) For water-cement ratio of 0.50, water = 25.0 litres
   ii) Extra water to be added for absorption in case of coarse aggregate, at 0.5 percent by mass = (+) 0.77 litres
   iii) Water to be deducted for free moisture present in sand, at 2 percent by mass = (–) 1.42 litres
   iv) Actual quantity of water to be added = 25.0 + 0.77 – 1.42 = 24.35 litres

5) Actual quantity of sand required after allowing for mass of free moisture = 71.0 + 1.42 = 72.42 kg

6) Actual quantity of coarse aggregate required:
   i) Fraction I = 92.7 – 0.46
      = 92.24 kg
   ii) Fraction II = 61.8 – 0.31
      = 61.49 kg

Therefore, the actual quantities of different constituents required for the mix are:

water = 24.35 kg
cement = 50.00 kg
sand = 72.42 kg

Coarse aggregate:
Fraction I = 92.24 kg
Fraction II = 61.49 kg

TABLE 31 RELATIONSHIP BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH OF CONCRETE

(*Clause* 6.1)

COMPRESSIVE STRENGTH AT 28 DAYS, kgf/cm²	WATER-CEMENT RATIO, BY WEIGHT	
	Non-Air-Entrained Concrete	Air-Entrained Concrete
(1)	(2)	(3)
450	0.38	—
400	0.43	—
350	0.48	0.40
300	0.55	0.46
250	0.62	0.53
200	0.70	0.61
150	0.80	0.71

NOTE — Table 31 is from 'Recommended Practice for Selecting Proportions for Norm⁻¹ Weight Concrete' Reported by ACI Committee 211 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

TABLE 32 APPROXIMATE MIXING WATER (kg/m³ OF CONCRETE) REQUIREMENTS FOR DIFFERENT SLUMPS AND MAXIMUM SIZES OF AGGREGATES

(*Clause* 6.1)

SLUMP, cm	MAXIMUM SIZES OF AGGREGATES IN mm							
	10	12.5	20	25	40	50	70	150
	Non-Air-Entrained Concrete							
3 to 5	205	200	185	180	160	155	145	125
8 to 10	225	215	200	195	175	170	160	140
15 to 18	240	230	210	205	185	180	170	—
Approximate amount of entrained air in non-air-entrained concrete, percent	3.0	2.5	2.0	1.5	1.0	0.5	0.3	0.2
	Air-Entrained Concrete							
3 to 5	180	175	165	160	145	140	135	120
8 to 10	200	190	180	175	160	155	150	135
15 to 18	215	205	190	185	170	165	160	—
Recommended average total air content, percent	8.0	7.0	6.0	5.0	4.5	4.0	3.5	3.0

NOTE — Table 32 is from 'Recommended Practice for Selecting Proportions for Normal Weight Concrete' Reported by ACI Committee 211 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

TABLE 33 VOLUME OF DRY-RODDED COARSE AGGREGATE PER UNIT VOLUME OF CONCRETE

(*Clause* 6.1)

Maximum Size of Aggregate mm	Fineness Module of Sand			
	2.40	2.60	2.80	3.00
(1)	(2)	(3)	(4)	(5)
10	0.50	0.48	0.46	0.44
12.5	0.59	0.57	0.55	0.53
20	0.66	0.64	0.62	0.60
25	0.71	0.69	0.67	0.65
40	0.76	0.74	0.72	0.70
50	0.78	0.76	0.74	0.72
70	0.81	0.79	0.77	0.75
150	0.87	0.85	0.83	0.81

NOTE — Table 33 is from 'Recommended Practice for Selecting Proportions for Normal Weight Concrete' Reported by ACI Committee 211 (ACI Manual of Concrete Practice, Part I, 1979). American Concrete Institute, USA.

TABLE 34 PROBABLE MINIMUM AVERAGE COMPRESSIVE STRENGTH OF CONCRETE FOR VARIOUS WATER-CEMENT RATIOS

(*Clause* 6.2)

Water-Cement Ratio by Weight	Compressive Strength at 28 Days (kgf/cm²)	
	Air-Entrained Concrete	Air-Entrained Concrete With Water-Reducing, Set-Controlling Admixtures
(1)	(2)	(3)
0.40	399	455
0.45	343	392
0.50	294	336
0.55	252	294
0.60	217	252
0.65	182	217
0.70	154	189

NOTE — Table 34 is from 'Concrete Manual' (Eighth Edition Revised Reprint, 1981). Bureau of Reclamation, United States Department of the Interior, USA.

TABLE 35 APPROXIMATE AIR AND WATER CONTENTS PER CUBIC METER OF CONCRETE AND THE PROPORTIONS OF FINE AND COARSE AGGREGATE

(*Clause* 6.2)

Maximum Size of Coarse Aggregate mm	Recommended Air-content, Percent	Sand, Percent of Total Aggregate by Solid Volume	Percent, Dry Rodded Unit Weight of Coarse Aggregate per Unit Volume of Concrete	Air-Entrained Concrete: Average Water Content kg/m³	Air-Entrained Concrete With Water Reducing, Set-Controlling Admixture, Average Water Content kg/m³
(1)	(2)	(3)	(4)	(5)	(6)
10	8	60	41	189	177
13	7	50	52	180	168
20	6	42	62	165	156
25	5	37	67	156	147
40	4.5	34	73	145	136
50	4	30	76	136	122
75	3.5	28	81	121	112
150	3	24	87	97	91

NOTE 1 — This table is applicable for concrete containing natural sand with fineness modulus of 2.75 and average coarse aggregate and having a slump of 75 to 100 mm at the mixer.

NOTE 2 — Table 35 is from 'Concrete Manual', (Eighth Edition Revised Reprint, 1981). Bureau of Reclamation, United States Department of the Interior, USA.

TABLE 36 ADJUSTMENT OF VALUES OF WATER CONTENT, PERCENT SAND AND PERCENT OF DRY-RODDED COARSE AGGREGATE

(*Clause* 6.2)

Changes in Material or Mix Proportions	Adjustment Required in		
	Water Content Percent	Sand Percent	Percent, Dry-Rodded Coarse Aggregate
(1)	(2)	(3)	(4)
Each 0.1 increase or decrease in fineness modulus of sand	—	±0.5	±1.0
Each 25 mm increase or decrease in slump	±3	—	—
Each 1 percent increase or decrease in air content	±3	±0.5 to 1.0	—
Each 0.05 increase or decrease in water-cement ratio	—	±1	—
Each 1 percent increase or decrease in sand content	±1	—	±2.0

NOTE — Table 36 is from 'Concrete Manual', (Eighth Edition Revised Reprint, 1981). Bureau of Reclamation, United States Department of the Interior, USA.

TABLE 37 APPROXIMATE COMPRESSIVE STRENGTHS OF CONCRETE MIXES MADE WITH WATER-CEMENT RATIO OF 0.5

(*Clause* 6.3)

Type of Cement	Type of Coarse Aggregate	Compressive Strength (N/mm^2) Age (days)			
		3	7	28	91
(1)	(2)	(3)	(4)	(5)	(6)
Ordinary Portland Cement or Sulphate Resisting Portland Cement	Uncrushed	18	27	40	48
	Crushed	23	33	47	55
Rapid Hardening Portland Cement	Uncrushed	25	34	46	53
	Crushed	30	40	53	60

NOTE — Table 37 is from 'Design of Normal Concrete Mixes (1975)' by D. C. Teychenne, R. E. Franklin and H. C. Erntroy. Contributed by courtesy of the Director, Building Research Establishment and reproduced by the permission of the Controller of Her Britannic Majesty's Stationery Office. Crown copyright.

TABLE 38 APPROXIMATE WATER CONTENTS (kg/m^3) REQUIRED TO GIVE VARIOUS LEVELS OF WORKABILITY

(*Clause* 6.3)

Slump (mm) →		0-10	10-30	30-60	60-180
Vee-Bee (s) →		> 12	6-12	3-6	0-3
Maximum Size of Aggregate (mm)	Type of Aggregate				
(1)	(2)	(3)	(4)	(5)	(6)
10	Uncrushed	150	180	205	225
	Crushed	180	205	230	250
20	Uncrushed	135	160	180	195
	Crushed	170	190	210	225
40	Uncrushed	115	140	160	175
	Crushed	155	175	190	205

NOTE 1 — When coarse and fine aggregates of different types are used, the water content is estimated by the expression:

$$2/3\ W_f + 1/3\ W_c$$

where

W_f = water content appropriate to type of fine aggregate, and

W_c = water content appropriate to type of coarse aggregate.

NOTE 2 — Table 38 is from 'Design of Normal Concrete Mixes (1975)' by D. C. Teychenne, R. E. Franklin and H. C. Erntroy. Contributed by courtesy of the Director, Building Research Establishment and reproduced by the permission of the Controller of Her Britannic Majesty's Stationery Office. Crown copyright.

TABLE 39 SUGGESTED VALUES OF STANDARD DEVIATION

(*Clause* 6.4)

Grade of Concrete	Standard Deviation for Different Degree of Control (N/mm^2)		
	Very Good	Good	Fair
(1)	(2)	(3)	(4)
M 10	2.0	2.3	3.3
M 15	2.5	3.5	4.5
M 20	3.6	4.6	5.6
M 25	4.3	5.3	6.3
M 30	5.0	6.0	7.0
M 35	5.3	6.3	7.3
M 40	5,6	6.6	7.6
M 45	6.0	7.0	8.0
M 50	6.4	7.4	8.4
M 55	6.7	7.7	8.7
M 60	6.8	7.8	8.8

TABLE 40 DEGREE OF QUALITY CONTROL EXPECTED UNDER DIFFERENT SITE CONDITIONS

(*Clause* 6.4)

Degree of Control	Conditions of Production
Very Good	Fresh cement from single source and regular tests, weighbatching of all materials, aggregates supplied in single sizes, control of aggregate grading and moisture content, control of water added, frequent supervision, regular workability and strength tests, field laboratory facilities.
Good	Carefully stored cement and periodic tests, weighbatching of all materials, controlled water, graded aggregate supplied, occasional grading and moisture tests, periodic check of workability and strength, intermittent supervision, experienced workers.
Fair	Proper storage of cement, volume batching of all aggregates allowing for bulking of sand, weighbatching of cement, water content controlled by inspection of mix, occasional supervision and tests.

TABLE 41 APPROXIMATE ENTRAPPED AIR CONTENT

(*Clause* 6.4)

Nominal Maximum Size of Aggregate (mm)	Entrapped Air, as Percent of Volume of Concrete
(1)	(2)
10	3.0
20	2.0
40	1.0

Note — Table 41 is from 'Hardened Concrete; Mechanical Aspects: ACI Monograph No. 6 (1971)' by A. M. Neville with the permission of the American Concrete Institute, USA.

TABLE 42 APPROXIMATE SAND AND WATER CONTENTS PER CUBIC METRE OF CONCRETE

(*Clause* 6.4)

W/C = 0.60

Workability = 0.80 CF

(Applicable for concrete up to grade M 35)

Maximum Size of Aggregate (mm)	Water Content* per Cubic Metre of Concrete (kg)	Sand as Percent of Total Aggregate by Absolute Volume
(1)	(2)	(3)
10	208	40
20	186	35
40	165	30

*Water content corresponding to saturated surface dry aggregate.

TABLE 43 APPROXIMATE SAND AND WATER CONTENTS PER CUBIC METRE OF CONCRETE

(*Clause* 6.4)

W/C = 0.35
Workability = 0.80 CF
(Applicable for concrete above grade M 35)

MAXIMUM SIZE OF AGGREGATE (mm)	WATER CONTENT* PER CUBIC METRE OF CONCRETE (kg)	SAND AS PERCENT OF TOTAL AGGREGATE BY ABSOLUTE VOLUME
(1)	(2)	(3)
10	200	28
20	180	25

*Water content corresponding to saturated surface dry aggregate.

TABLE 44 ADJUSTMENT OF VALUES IN WATER CONTENT AND SAND PERCENTAGE FOR OTHER CONDITIONS

(*Clause* 6.4)

CHANGE IN CONDITIONS STIPULATED FOR TABLES 41 AND 42	ADJUSTMENT REQUIRED IN	
	Water Content	Percent Sand in Total Aggregate
(1)	(2)	(3)
For sand conforming to grading Zone I, Zone III or Zone IV of Table 4, IS : 383-1970	0	+ 1.5 Percent for Zone I − 1.5 Percent for Zone III − 3.0 Percent for Zone IV
Increase or decrease in the value of compacting factory by 0.1	± 3 Percent	0
Each 0.05 increase or decrease in water-cement ratio	0	± 1 Percent
For rounded aggregate	− 15kg/m³	− 7 Percent

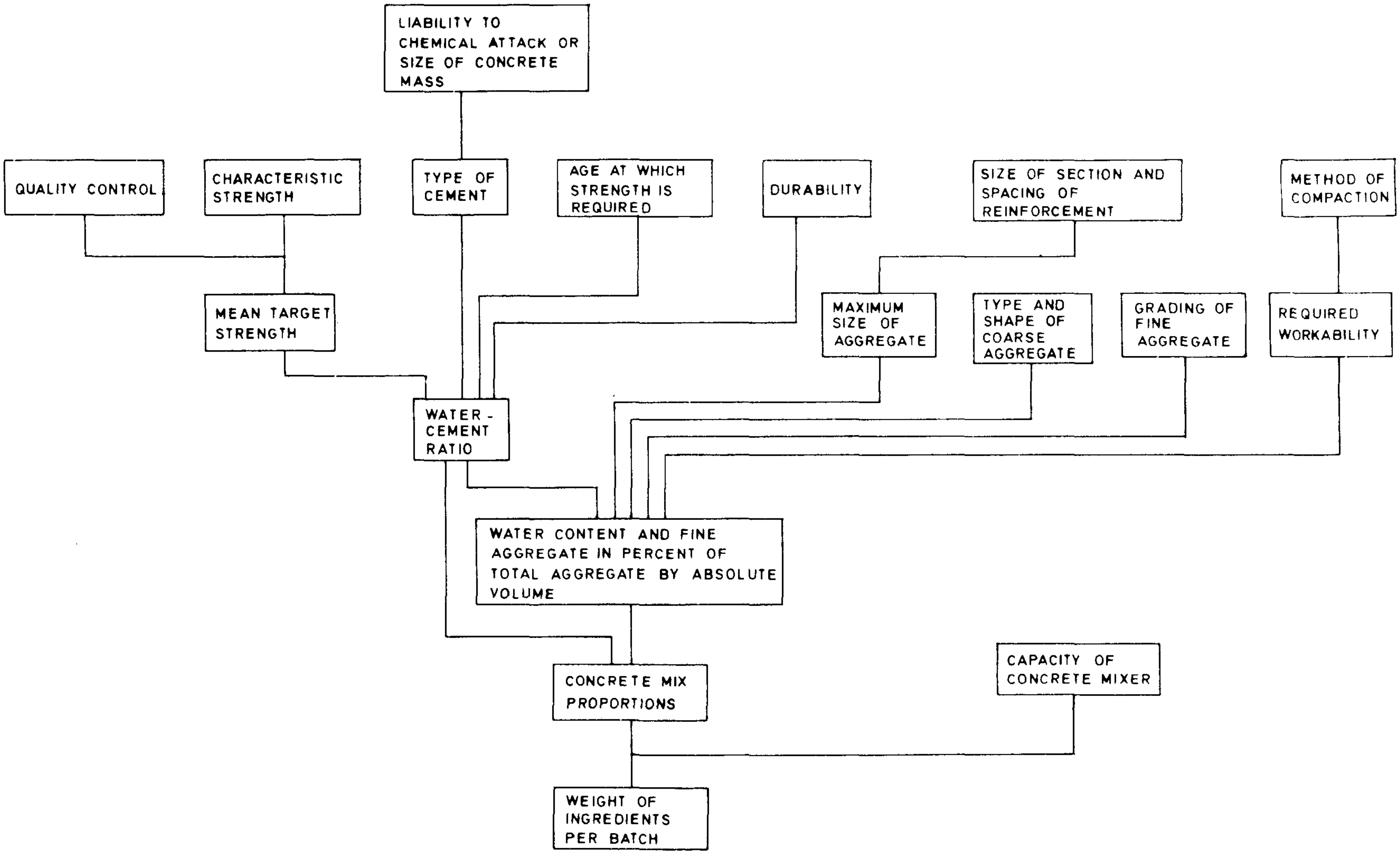


Fig. 42 Procedure of Concrete Mix Design

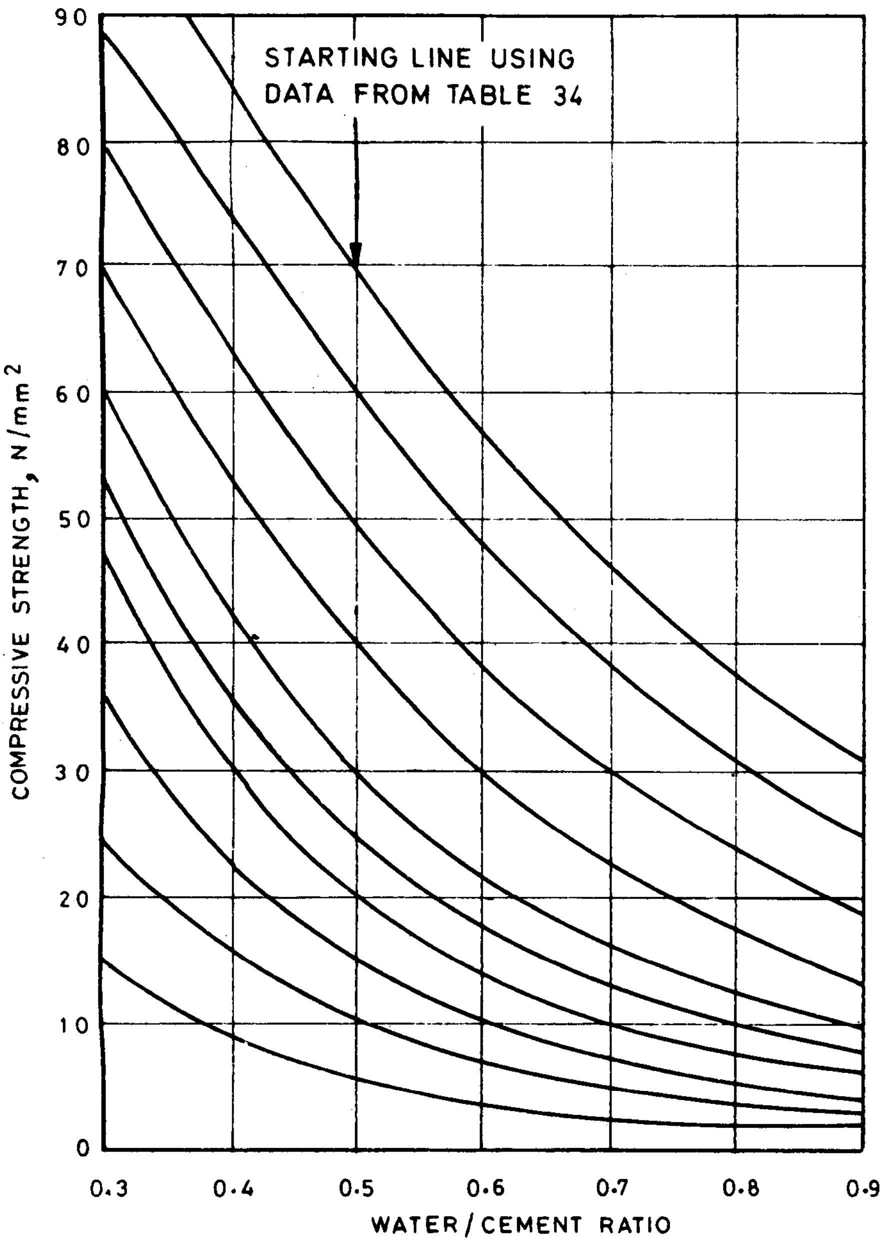


Fig. 43 Relationship Between Compressive Strength and Water-Cement Ratio

NOTE — Figure 43 is reproduced from 'Design of Normal Concrete Mixes (1975)' by D. C. Teychenne, R. E. Franklin and H. C. Erntroy. Contributed by courtesy of the Director, Building Research Establishment and reproduced by the permission of the Controller of Her Britannic Majesty's Stationery Office. Crown Copyright.

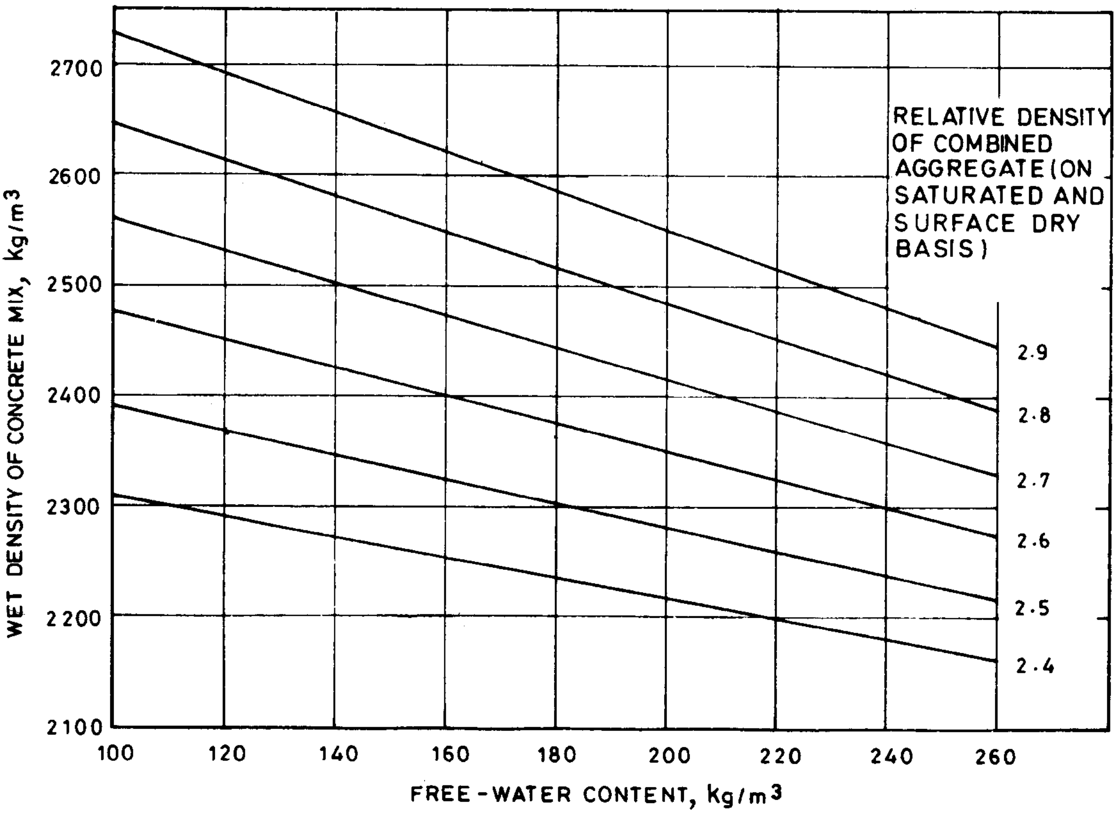


Fig. 44 Estimated Wet Density of Fully Compacted Concrete

NOTE — Figure 44 is reproduced from 'Design of Normal Concrete Mixes (1975)' by D. C. Teychenne, R. E. Franklin and H. C. Erntroy. Contributed by courtesy of the Director, Building Research Establishment and reproduced by the permission of the Controller of Her Britannic Majesty's Stationery Office. Crown Copyright.

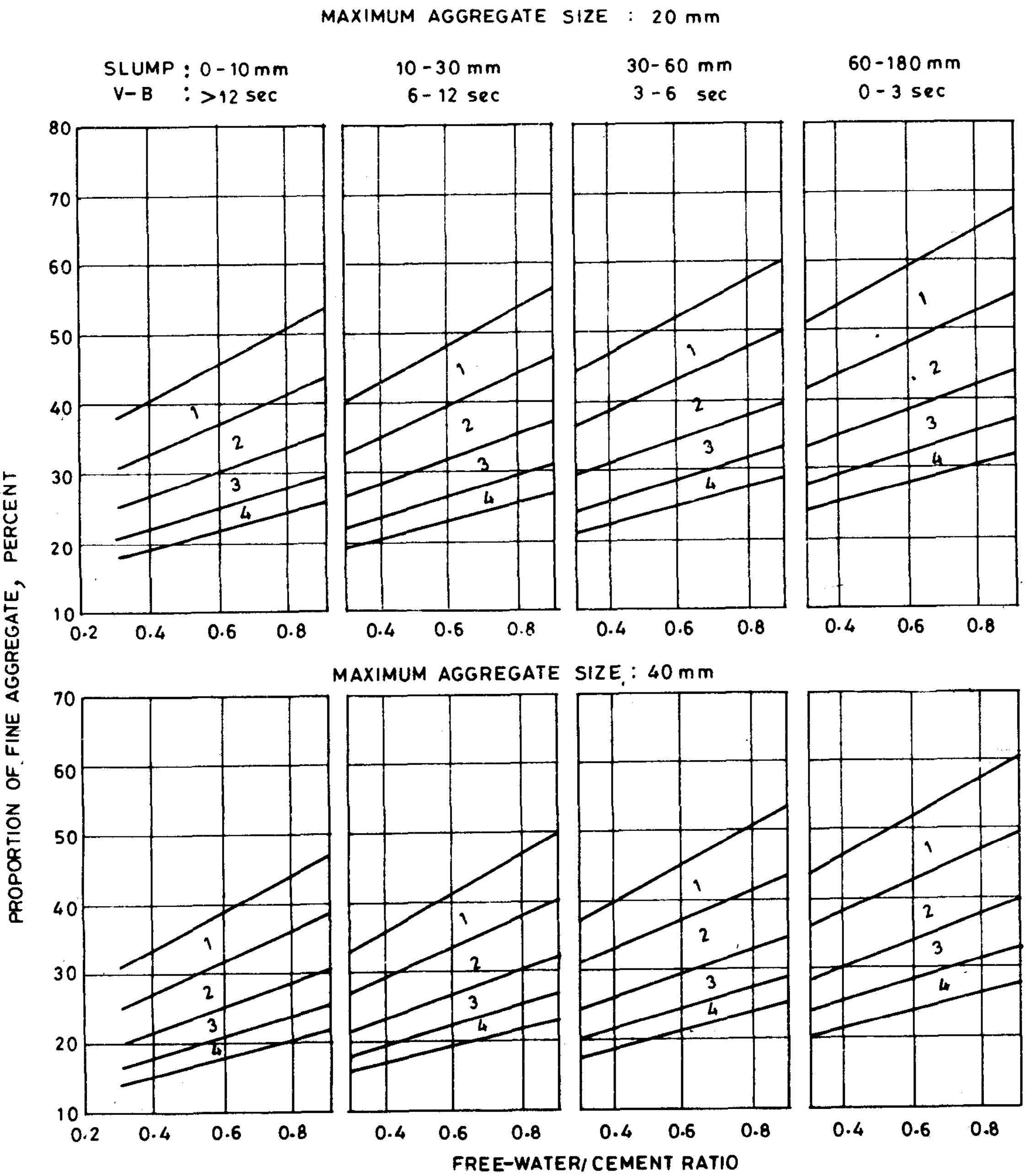


Fig. 45 Recommended Proportions of Fine Aggregate for Grading Zones 1, 2, 3 and 4

NOTE — Figure 45 is from 'Design of Normal Concrete Mixes (1975)' by D. C. Teychenne, R. E. Franklin and H. C. Erntroy. Contributed by courtesy of the Director, Building Research Establishment and reproduced by the permission of the Controller of Her Britannic Majesty's Stationery Office. Crown Copyright.

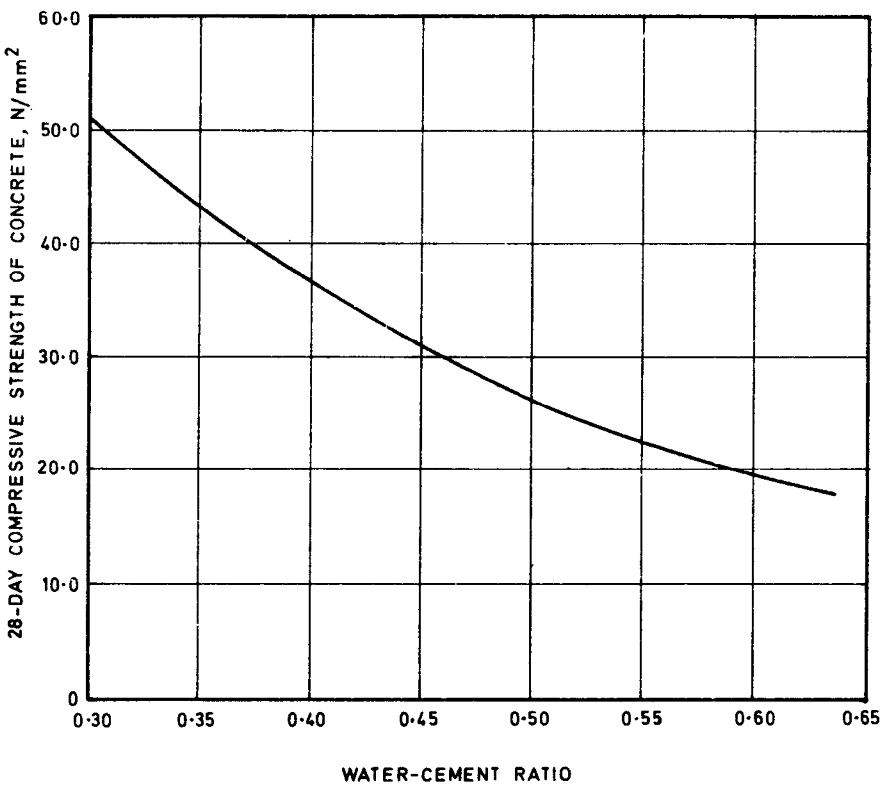
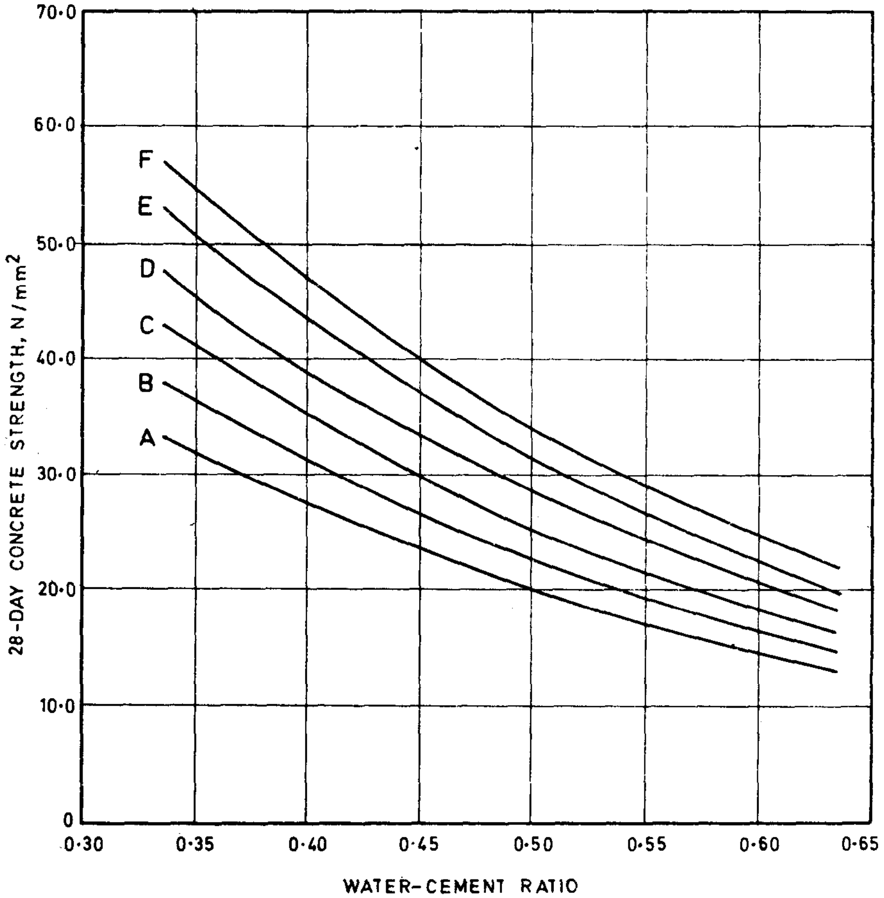


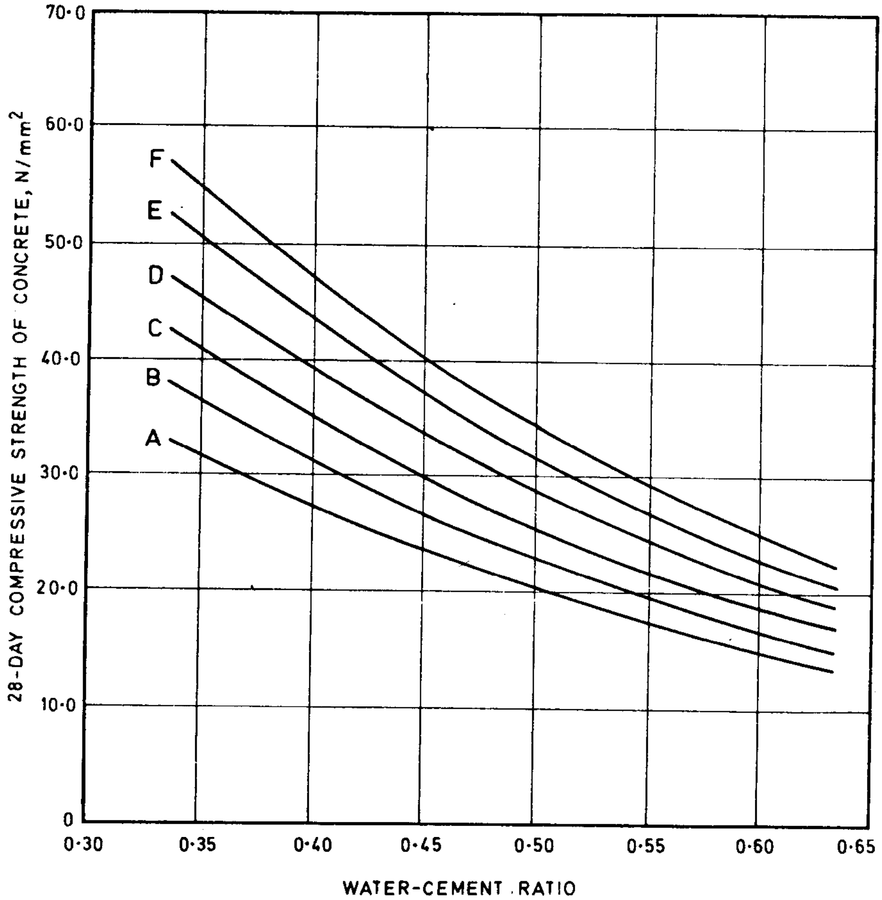
Fig. 46 Generalized Relationship Between Free Water-Cement Ratio and Compressive Strength of Concrete



28-Day Strength of Cement, Tested According to IS : 4031-1968

A — 31.9-36.8 N/mm² (325-375 kg/cm²)
B — 36.8-41.7 N/mm² (375-425 kg/cm²)
C — 41.7-46.6 N/mm² (425-475 kg/cm²)
D — 46.6-51.5 N/mm² (475-525 kg/cm²)
E — 51.5-56.4 N/mm² (525-575 kg/cm²)
F — 56.4-61.3 N/mm² (575-625 kg/cm²)

Fig. 47 Relationship Between Free Water-Cement Ratio and Concrete Strength for Different Cement Strengths



Accelerated Strength (Tested According to IS : 9013-1978) of Reference Concrete Mixes

A —	12.3-15.2	N/mm²	(125-155 kg/cm²)
B —	15.2-18.1	N/mm²	(155-185 kg/cm²)
C —	18.1-21.1	N/mm²	(185-215 kg/cm₂)
D —	21.1-24.0	N/mm²	(215-245 kg/cm²)
E —	24.0-27.0	N/mm²	(245-275 kg/cm²)
F —	27.0-29.9	N/mm²	(275-305 kg/cm²)

Fig. 48 Relationship Between Free Water-Cement Ratio and Compressive Strength of Concrete for Different Cement Strengths Determined on Reference Concrete Mixes (Accelerated Test — Boiling Water Method)

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SECTION 7

EXTREME WEATHER CONCRETING

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SECTION 7 EXTREME WEATHER CONCRETING

7.0 What has been discussed so far in earlier sections refer to properties of concrete for practice of concreting under 'normal' conditions of temperature and humidity. Nevertheless, there can be situations where the temperature of concrete at the time of placing and the environmental temperature during concreting and its subsequent curing periods may be different from those of normal conditions that is, either the temperature is higher or lower. The situation may become further aggravated by decrease of the humidity in the atmosphere, increase of wind or combinations of these. In such situations the properties and performance of concrete are likely to be affected unless due precautions are taken.

The problems of extreme weather concreting stem mainly from the fact that kinetics of hydration of cement, so essential for the development of strength and other intrinsic properties of concrete, are altered. In general, an increase in temperature will accelerate the rate of hydration and decrease in temperature will decelerate it. Therefore, the initial rate of development of strength can be expected to be faster in hot weather and slower in cold weather. Coupled with this is the problem of loss of water due to evaporation, which may affect the workability of fresh concrete and plastic shrinkage that accompany the rapid drying due to consequential loss of water from the fresh concrete. Yet another aspect is the type of microstructure of cement paste that is formed during such accelerated or decelerated hydration; a higher temperature may lead to an accelerated growth of hydrates which may not be as orderly or as compact as can be expected, if the reactions were to proceed at normal rate. In cold weather, although the initial strength may be reduced, the miscrostructure formed is perhaps more orderly and compact.

A comprehensive study on the effects of mixing and curing temperature on the various properties of concrete like compressive strength, flexural strength, workability, air entrainment, etc, was carried out by the Portland Cement Association, which has formed the basis of 'IS: 7861 (Part I)-1975 Code of practice for extreme weather concreting: Part I Recommended practice for hot weather concreting' and 'IS: 7861 (Part II)-1981 Code of practice for extreme weather concreting: Part II Recommended practice for cold weather concreting'.

7.1 Hot Weather Concreting — Any operation of concreting done at atmospheric temperature above 40°C or any operation of concreting (other than steam curing) where the temperature of concrete at the time of its placement is expected to be beyond 40°C may be put under hot weather concreting. Concrete is not recommended to be placed at a temperature above 40°C without proper precautions. The climatic factors affecting concrete in hot weather are high ambient temperature and reduced relative humidity, the effects of which may be more pronounced with increase in wind velocity. The effects of hot weather are most critical during periods of rising temperature and falling relative humidity or both. There are some special problems involved in concreting in hot weather, arising both from temperature rise of the concrete and consequential increase in rate of evaporation from the fresh concrete mix. These problems concern the mixing, placing and curing of the concrete.

7.1.1 Effects of Hot Weather on Concrete — In the absence of special precautions, the effects of hot weather may be described as follows:

a) *Accelerated setting* — A higher temperature of the fresh concrete results in a more rapid hydration and leads to accelerated setting. This reduces the handling time of concrete and also lowers the strength of hardened concrete. Quick stiffening may necessitate undesirable retempering by addition of water. Added water without proper adjustments in mix proportions will adversely affect the ultimate quality of concrete in place, and may adversely influence placement, consolidation and finishing. With the increase in concrete temperature, the slump (workability) decreases and hence the water demand increases with increase in temperature of concrete, for constant consistancy (*see* Fig. 22). As a typical example, it was reported that approximately 25 mm decrease in slump resulted for each 11°C increase in the concrete temperature. Another disadvantage from accelerated setting of concrete is the formation of cold joints.

b) *Reduction in strength* — Concrete mixed, placed and cured at elevated temperatures normally develops higher early strengths than concretes produced and cured at normal temperatures but at 28 days or later strengths are generally lower. Tests have shown that for concretes placed and cured up to 28 days at various temperatures ranging from 49°C to −4°C at 100 percent relative humidity, the initial strength (up to an age of 7 days or so) was generally greater, at higher temperatures. However, the difference in the strength at various temperature tended to be narrower as the age of concrete increased, and indeed at an age of nearly one year the low temperature concrete developed higher strength than those at higher temperatures (*see* Fig. 49[1]). For the influence of simultaneous reduction in the relative humidity, it was shown that specimens moulded and cured in air at 23°C, 60 percent relative humidity and at 38°C, 25 percent relative humidity, produced strength of only 73 and 62 percent, respectively, in comparison with the specimens moist cured at 23°C for 28 days. It was also found that the larger the delay between casting and placing, greater is the strength reduction. High temperature results in greater evaporation loss and hence necessitates increase of mixing water consequently reducing strength.

c) *Increased tendency to crack* — Rapid evaporation may cause plastic shrinkage and cracking, and subsequent cooling of the hardened concrete would introduce tensile stresses. It is generally believed that plastic shrinkage is likely to occur when the rate of evaporation exceeds the rate at which bleeding water rises to the surface, but it has been recently found that cracks also form under a layer of water and merely become apparent on drying. The most important factor influencing plastic shrinkage cracking is the rate of evaporation of water from the surface of the concrete. This depends on the ambient temperature, the relative humidity, the wind speed and the concrete temperature. Results from a road construction indicated that the intensity of cracking was proportional to the maximum day temperature during construction. Cracking was almost confined to the morning work which was attributed to the exposure of concrete to direct sun and higher air temperature for a longer time, than the concrete laid during the afternoon.

d) *Rapid evaporation of water during curing period* — In order to obtain a good concrete the placing of an appropriate mix must be followed by curing in a suitable environment during the early stages of hardening. The necessity of curing arises from the fact that hydration of cement can take place only in water-filled capillaries. For this reason, a loss of water by evaporation from the capillaries must be prevented. Further more, water lost internally by self-desiccation has to be replaced by water from outside. Rapid initial hydration seems to form products of poor physical structure, probably more porous, so that a large proportion of the pores will remain unfilled. This leads to lower strength.

e) *Difficulty in control of air content in air-entrained concrete* — It is more difficult to control air content in air-entrained concrete. This adds to the difficulty of controlling workability. For a given amount of air-entraining agent, hot concrete will entrain less air than concrete at normal temperatures.

7.1.2 RECOMMENDED PRACTICES AND PRECAUTIONS

7.1.2.1 TEMPERATURE CONTROL OF CONCRETE INGREDIENTS — The most direct approach to keep concrete temperature down is by controlling the temperature of the ingredients. The aggregates and mixing water exert the most pronounced effect by virtue of their quantity and specific heat, respectively.

Aggregates may be kept shaded from direct sun rays. They may be sprinkled with water or may be cooled by methods, such as inundating them in cold water or by circulating refrigerated air through pipes or other suitable methods. Mixing water has the greatest effect on lowering the temperature of concrete, because the specific heat of water (1.0) is nearly 5 times that of common aggregates (0.22). The temperature of water is easier to control than that of other ingredients. The use of cold mixing water will affect a moderate reduction in concrete placing temperature. Under certain circumstances, reduction in water temperature may be most economically accomplished by mechanical refrigerator or mixing with crushed ice. To take advantage of the latent heat of fusion, the ice shall be incorporated directly into the concrete as a part of mixing water. Conditions shall be such that the ice is completely melted by the time mixing is completed.

Investigations were carried out on the effects of temperature of cement on the strength of concrete. Cements having temperatures of 23°C, 64°C and 80°C were used in preparing concretes, each at the temperature at the end of mixing period of 24°C and 40°C, the final concrete temperature being attained by adjusting the temperature of concrete ingredients (*see* Fig. 50). Tests showed that so long as the comparison is made on the basis of equal concrete temperatures, the temperature of cement did not exert any significant influence on the strength of concrete.

7.1.2.2 PROPORTIONING OF CONCRETE MIX MATERIALS AND MIX DESIGN — Mix should be designed to have minimum cement content consistent with other functional requirements. As far as possible, cement with lower heat of hydration shall be preferred instead of cements having greater fineness and heat of hydration.

Use of water-reducing and/or set-retarding admixtures are beneficial. Such admixtures should, however, be used after proper evaluation.

7.1.2.3 PRODUCTION AND DELIVERY — Temperature of the aggregates, water and cement shall be maintained at the lowest practical levels so that the temperature of concrete is below 40°C at the time of placement. The temperature of the concrete at the time of leaving the mixer or batching plant should be measured with suitable metal-clad thermometer. In the absence of such measurement, the temperature may be calculated from the following formula:

a) Cold water as mixing water (without ice)

$$T = \frac{S(T_a W_a + T_c W_c) + (T_w W_w + T_{wa} W_{wa})}{S(W_a + W_c) + W_w W_{wa}}$$

b) With ice added to mixing water

$$T = \frac{S(T_a W_a + T_c W_c)}{S(W_a + W_c) + W_w + W_i + W_{wa}} + \frac{(W_w - W_i) T_w + W_{wa} T_{wa} - 79.6 W_i}{S(W_a + W_c) + W_w + W_i + W_{wa}}$$

where

T = temperature of freshly mixed concrete (°C);

T_a, T_c, T_w, T_{wa} = temperature of aggregate, cement, added mixing water, and free water on aggregate, respectively (°C);

$W_a, W_c, W_w, W_{wa}, W_i$ = mass of aggregate, cement, added mixing water, free water on aggregate and ice respectively (kg); and

S = Specific heat of cement and aggregate (Cal/g.°C)

Examples for the calculation of temperature of fresh concrete when cold water or ice is added in place of mixing water at higher temperature are reproduced below from IS : 7861(Part I)-1975[2]:

a) Cold water as mixing water (without ice)

Consider a concrete mix having the following ingredients (per m³), and the initial temperature shown against each:

Cement	336 kg at 35°C
Water	170 kg at 30°C
Aggregates	1 850 kg at 45°C

S = 0.22 Cal/g.°C

It is assumed that the aggregates are dry, that is

$W_{wa} = 0$

The temperature (in °C) of fresh concrete as mixed with these ingredients will be:

$$T = \frac{0.22(45 \times 1\ 850 + 35 \times 336) + 30 \times 170}{0.22(1\ 850 + 336) + 170}$$
$$= 39.9°C$$

Suppose, the mixing water is added at 5°C, then the temperature of concrete (in °C) as mixed will be:

$$T = \frac{0.22(45 \times 1\ 850 + 35 \times 336) + 5 \times 170}{0.22(1\ 850 + 336) + 170}$$
$$= 33.4°C$$

Hence, reduction in concrete temperature is:

$$(39.9 - 33.4)°C = 6.5°C$$

b) With ice added to the mixing water in the example under (a), suppose 50 percent of the mixing water (that is, 85 kg) is replaced by ice.
Then the temperature of fresh concrete as mixed is given by:

$$T = \frac{0.22(45 \times 1\ 850 + 35 \times 336) + (170 - 85) \times 30 - 79.6 \times 85}{0.22(1\ 850 + 336) + 85 + 85}$$
$$= 25.6°C$$

Hence, reduction in concrete temperature is:

$$(39.9 - 25.6)°C = 14.3°C$$

The period between mixing and delivery shall be kept to an absolute minimum. Attention shall be given to coordinate the delivery of concrete with the rate of placement of concrete.

7.1.2.4 PLACEMENT, PROTECTION AND CURING — Formwork, reinforcement and subgrade shall be sprinkled with cool water just prior to placement of concrete. The area around the work shall be kept wet to the extent possible to cool the surrounding air and increase its humidity. Speed of placement and finishing helps to minimize problems in hot weather concreting. Sufficient men and machinery shall be employed to handle and place the concrete immediately on delivery.

Immediately after consolidation and surface finish, concrete shall be protected from evaporation of moisture without letting ingress of external water, by means of wet (not dripping) gunny bags, hessian, cloth, etc. Once the concrete has attained some degree of hardening sufficient to withstand surface damage (approximately 12 hours after mixing), moist curing shall commence. The actual duration of curing shall depend upon the mix proportions, size of the member as well as the environmental conditions; however, in any case it shall not be less than 10 days. Continuous curing is important, because the volume changes due to alternate wetting and drying promote the development of surface cracking. On exposed unformed concrete surfaces, such as pavement slabs, wind is an important factor in the drying rate of concrete. Hence, wind breakers shall be provided as far as possible. On the hardened concrete and on flat surfaces in particular, curing water shall not be much cooler than the concrete because of the possibilities of thermal stressess and resultant cracking.

7.2 Cold Weather Concreting — As was pointed out before, the production of concrete in cold weather introduces special problems which do not arise while concreting at normal temperatures. The problems are mainly due to slower development of concrete strength, the damage that can happen if concrete in the plastic state is exposed to low temperature which cause ice lenses to form and expansion to occur within the pore structure, and subsequent damage due to alternate freezing and thawing when the concrete has hardened. From the tests carried out at PCA laboratories[1] referred to earlier it was found that, there is a temperature during the early life of concrete which could be considered optimum with regard to its satisfactory performance at later stages. Mainly on the basis of these tests most of the codes do not advocate concreting to be done at an atmospheric temperature below 5°C. Accordingly, any concreting operation done at a temperature below 5°C is termed as cold weather concreting.

7.2.1 EFFECTS OF COLD WEATHER ON CONCRETING — In the absence of special precautions, effects of cold weather concreting may be described as follows:

a) *Delayed setting* — When the temperature is falling to about 5°C or below, the development of concrete strength is retarded compared with the strength development at normal temperatures. The hardening period necessary before removal of formwork is thus increased and the experience from concreting at normal temperature cannot be used directly. Effects of temperature of concrete on the strength development can be expressed as in Fig. 51[3]. Although the initial strengths of concrete are lower, it has now been found that the long-term strength of concrete will not be severely affected provided that the concrete has been prevented from freezing during its early life [*see* 7.2.1(b) below]. The combined effects of time and temperature as expressed in terms of 'maturity' of concrete was discussed earlier (*see* Section 3). However, such concept has not been found to be strictly applicable for winter concreting under actual site conditions[4]. One of the reasons may be that the actual temperatures of concrete, which could indeed be different from the ambient temperature, were not taken into account in such comparisons.

b) *Freezing of concrete at early stages* — The permanent damage that can be expected when the concrete still in fresh stage is exposed to freezing temparature can be seen from Fig. 52[5]. It is generally felt that if concrete is allowed to freeze before a certain 'pre-hardening period' concrete may suffer irrepairable loss in its properties so much so that even one cycle of freezing and thawing during the pre-hardening period may reduce the compressive strength to 50 percent of what could be expected for normal temperature concrete. Opinions differ as to the length of such 'pre-hardening period' which indeed depends upon the type of cement and the environmental conditions. While some specify it in terms of the time required to attain a compressive strength of the order of 35 to 70 kgf/cm^2, others have specified it in terms of a period varying from 24 hours to even 3 days depending upon the degree of saturation and water-cement ratio.

c) *Stresses due to temperature differentials* — Large temperature differentials within the concrete member may promote cracking and has a harmful effect on the durability. Such differentials are likely to occur in cold weather at the time of removal of formwork.

7.2.2 RECOMMENDED PRACTICE

7.2.2.1 TEMPERATURE CONTROL OF CONCRETE AGGREGATES — The most direct approach to keep concrete temperature above the permissible minimum is by controlling the temperature of the ingredients. All available means shall be used for maintaining these materials at as high a temperature as practicable. Heating of the aggregates shall be such that frozen lumps, ice and snow are eliminated and at the same time overheating is avoided. The average temperature of an aggregate for an individual batch shall not exceed 65°C. The mixing water shall be heated under such a control and in sufficient quality as to avoid appreciable fluctuation in temperature from batch to batch. The required temperature of mixing water to produce specified concrete can be obtained from Fig. 53[6] which are also included in IS : 7861 (Part II)-1981[7]. The heated water shall come in direct contact with aggregate first and not in contact with cement. Water having temperature up to the boiling point may be used provided the aggregate is cold enough to reduce the temperature.

7.2.2.2 USE OF INSULATING FORMWORK — Sufficient amount of heat is generated during hydration of cement. Such heat can be gainfully conserved by having insulating formwork covers which may maintain the concrete temperature above the desirable limits up to the first 3 days and (may be even up to 7 days), even when the ambient temperatures are lower. The formwork covers can be of timber, clean straw, blankets, sacking, tarpaulins, plastic sheeting, etc, in conjunction with air gap as insulation, efficiency of which depends upon the thermal conductivity of the medium as well as on the ambient temperature conditions. Tests on 90 cm concrete cubes made with ordinary Portland cement (310 kg/m^3) and insulated either with 50 mm lumber or 20 mm plywood showed that the concrete temperatures were nearly 30 to 40°C above the air temperature (*see* Fig. 54[5]). For moderately cold weather, timber formworks alone are sufficient and are preferable to steel formworks. The following comparison of different insulating materials, taken from ACI 306-1966[8] indicates how the efficiency of different combination varies due to their coefficient of thermal conductivity; the insulating values are indicated with reference to 25 mm commercial blankets as reference:

Insulating Material	*Equivalent Thickness* mm
25 mm commercial blanket	25
25 mm loose-fill insulation of fibrous type	25
25 mm insulating board	20
25 mm saw dust	15
25 mm timber	8
25 mm damp sand	0.6

In addition to insulated formworks, the concreting operations may sometimes require to be carried out in heated enclosures. The size of the section should be taken into account while using insulated formworks because of the thermal gradient that may result across the cross-section; usually the temperature at the centre of such massive section will be somewhat higher than near the formwork and the temperature around the corner are usually the lowest.

7.2.2.3 PROPORTIONING OF CONCRETE INGREDIENTS — Since the quantity of cement in the mix affects the rate of increase in temperature, additional quantity of ordinary Portland cement, rapid hardening Portland cement or accelerating admixtures used with proper precaution can help in getting the required strength in a shorter period. Air-entraining agents are generally recommended for use in cold weather. Air-entrainment increases the resistance of hardened concrete to freezing and thawing and normally at the same time improves the workability of fresh concrete. In cold weather concrete constructions, calcium chloride has been used as an accelerating admixture. However the matter is a subject of big controversy in so far as incidence of corrosion of reinforcing steel is considered. The winter condition as expected in India may not be as severe as elsewhere, where the successful use of calcium chloride is cited. In a study to stimulate the strength development of concrete under wintry condition as in India, it was found that perhaps air entrained concrete with insulated formwork is sufficient to ensure the minimum concrete temperature as well as to prevent the fresh concrete from damage during the prehardening period[9]. In any case, calcium chloride shall not be used in prestressed concrete construction.

7.2.2.4 PLACEMENT, PROTECTION AND CURING — Before any concrete is placed, all ice, snow and frost shall be completely removed. It should be remembered that no amount of insulation can supply heat at below freezing temperature. Care should be taken to see that the surface on which the concrete is to be placed and the steel and all eminent parts are sufficiently warm. It will be wrong to expect that the heat from the fresh concrete will thaw a frozen surface or mould inside the formwork without damaging the concrete. Whenever it is proposed to place concrete at or below 2°C, it is essential to know that the time taken for the concrete temperature to fall to freezing point is at least equal to the minimum prehardening period. For weather conditions below −1°C, the temperature of the concrete as mixed should be 15.5°C and as placed 10°C. During periods of freezing or near-freezing conditions, water curing is not necessary.

7.2.2.5 DELAYED REMOVAL OF FORMWORK — Because of slower rate of gain of strength during the cold weather, the formwork and props have to be kept in place for longer time than in usual concreting practice. The appropriate time for removal of formwork may be ascertained from the strength of test cubes left at site under the same conditions of temperature and humidity as the structural element concerned. The time of removal of formwork thus arrived should not be less than the duration of necessary protection afforded by the formwork (*see 7.2.2.2*). As a general guidance, the minimum time limits for stripping of formwork of members (carrying only its own weight), and at air temperature about 3°C as given in Reference 7 is reproduced below:

	With OPC Days	*With RHC* Days
Beam sides, walls, columns	5	3
Slabs (props left under)	7	4
Beams soffits (props left under)	14	8
Removal of props to slabs	14	8
Removal of props to beams	28	16

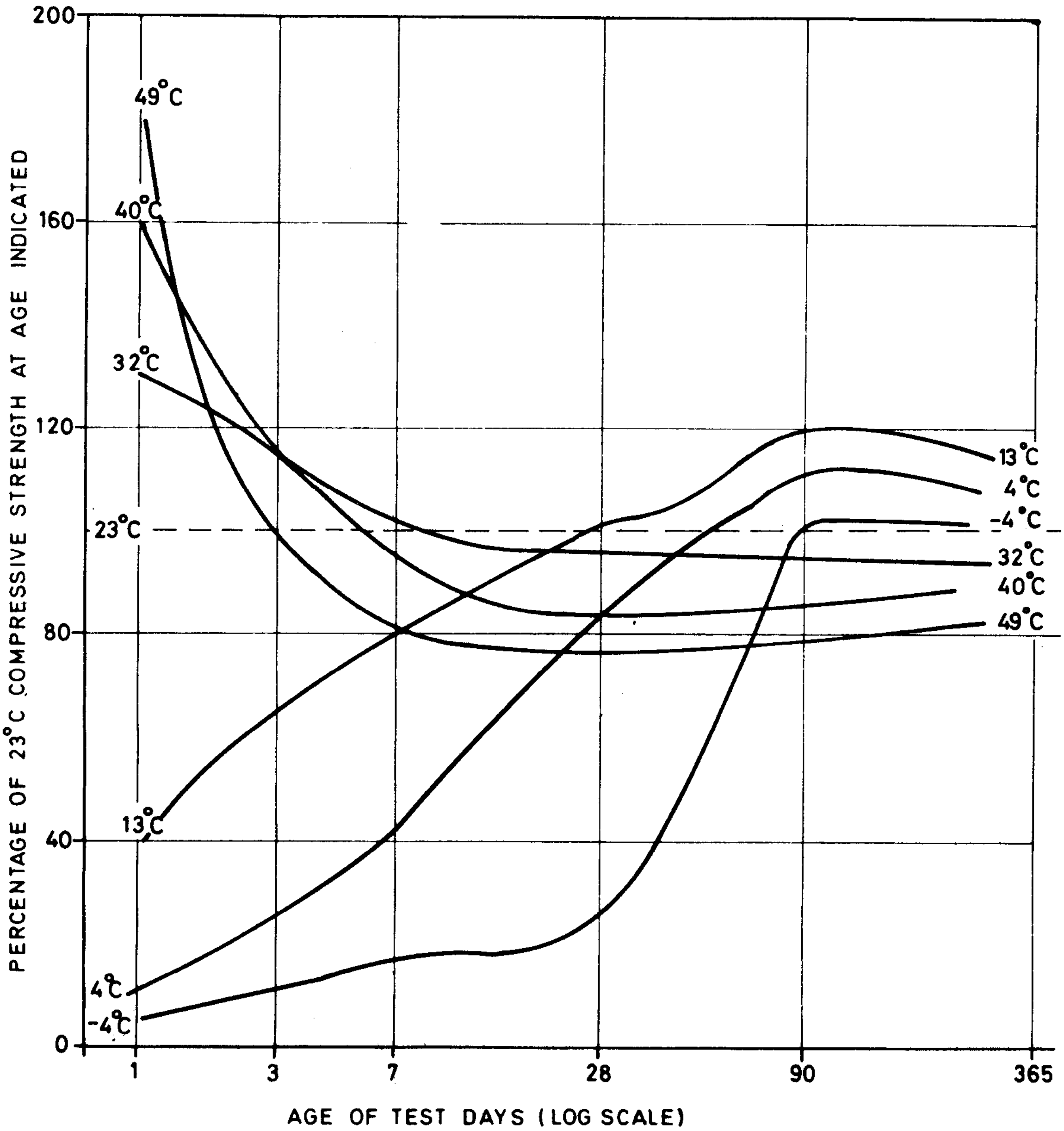


Fig. 49 Effect of Temperature on Compressive Strength of Concrete Made with Ordinary Portland Cement

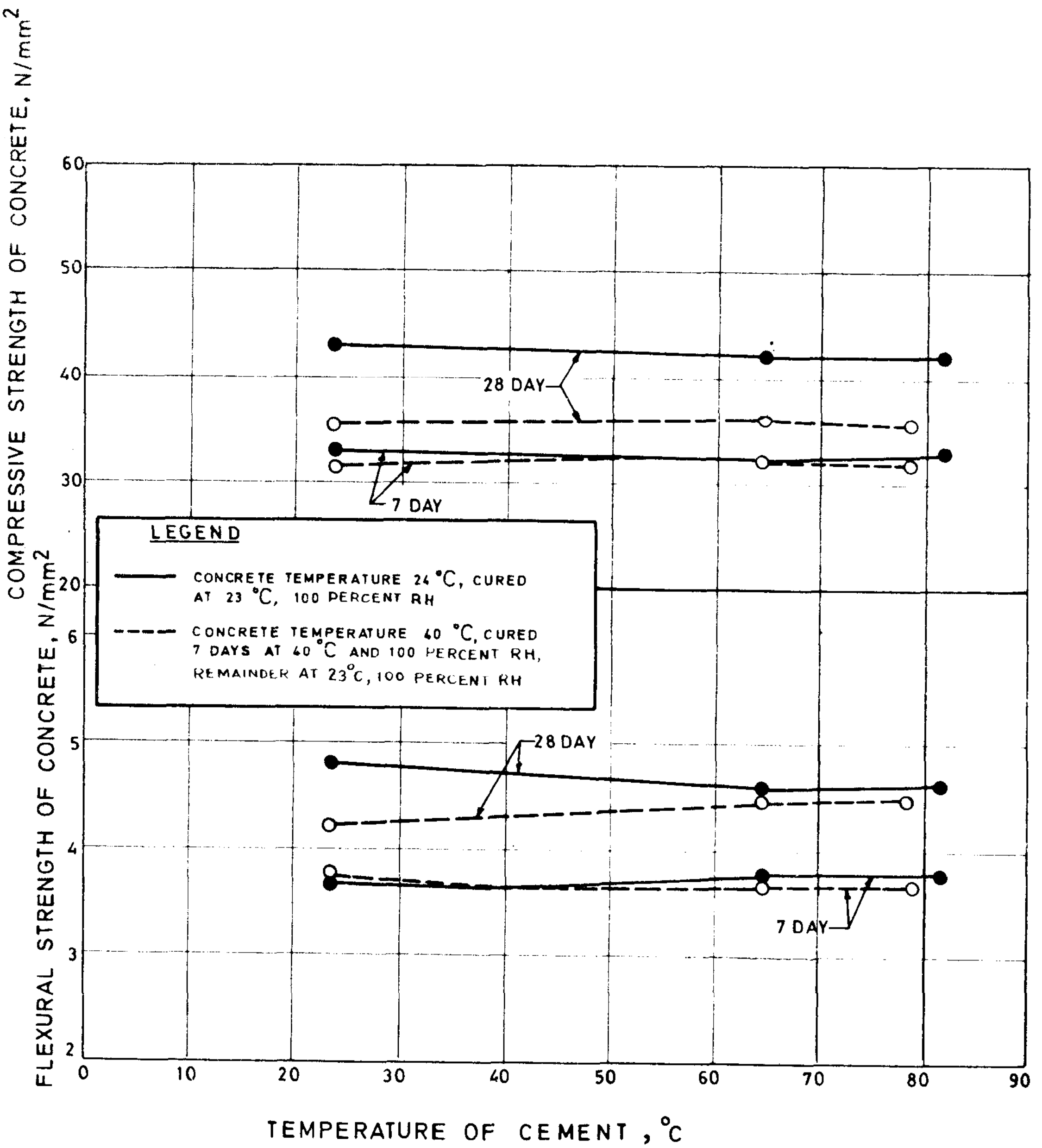


Fig. 50 Effect of Temperature of Cement on Concrete Strength

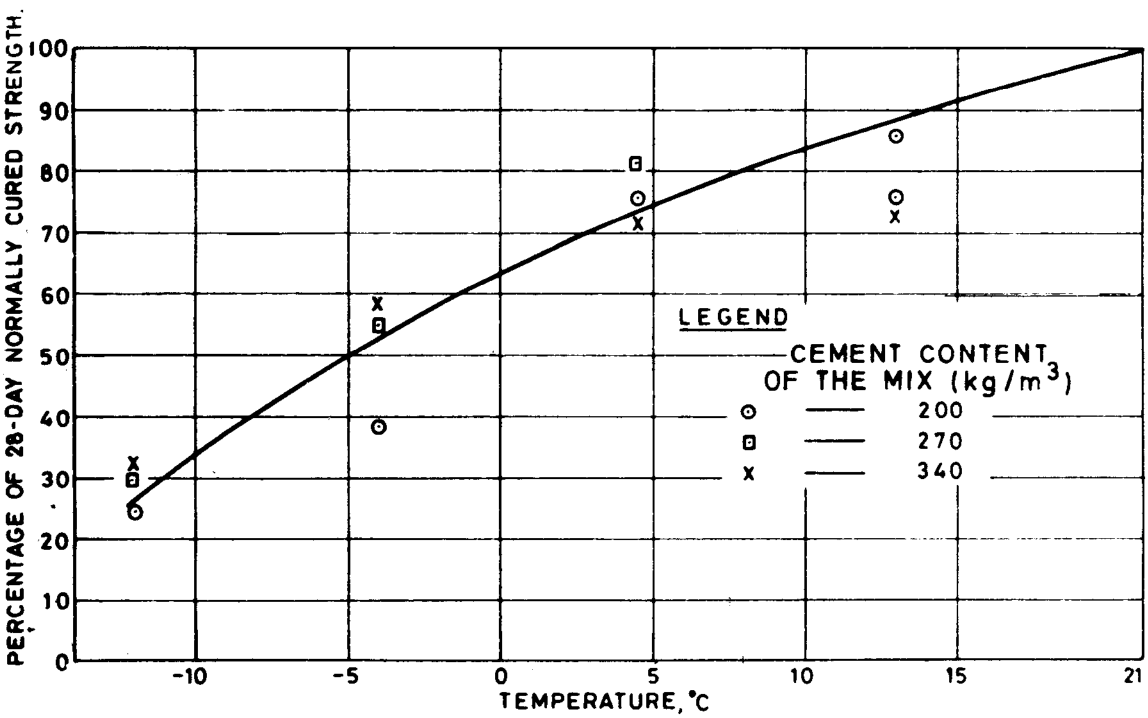


Fig. 51 Effect of Low Temperature on Compressive Strength of Concrete

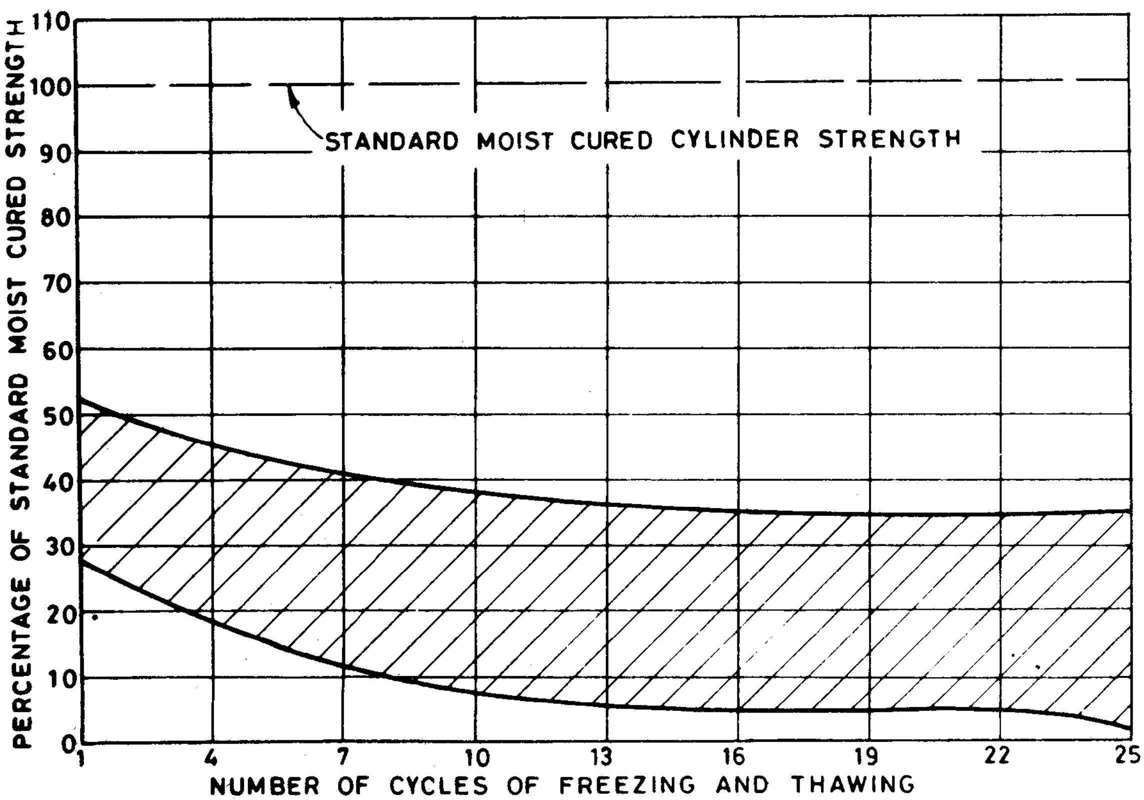
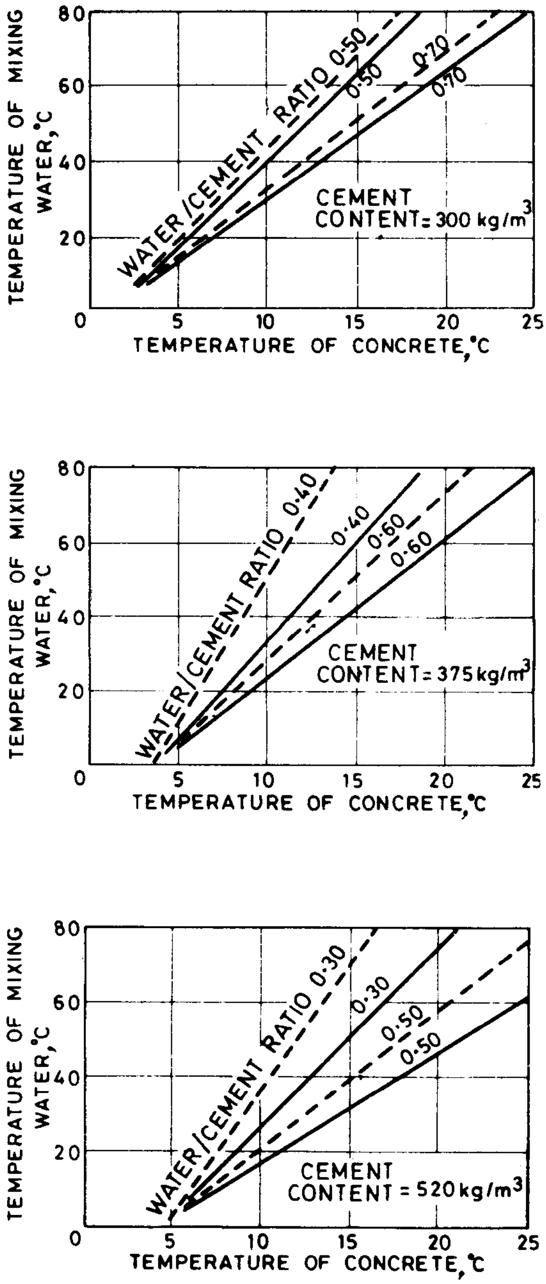


Fig. 52 Effect of Cycles of Freezing and Thawing of Fresh Concrete During Prehardening on Compressive Strength



MOISTURE CONTENT OF AGGREGATE:

DAMP (4% IN FINE, 1% IN COARSE)

WET (8% IN FINE, 2% IN COARSE)

TEMPERATURE OF AGGREGATE AND CONTAINED MOISTURE = 1°C

TEMPERATURE OF CEMENT = 5°C

Fig. 53 Required Temperature of Mixing Water to Produce Heated Concrete

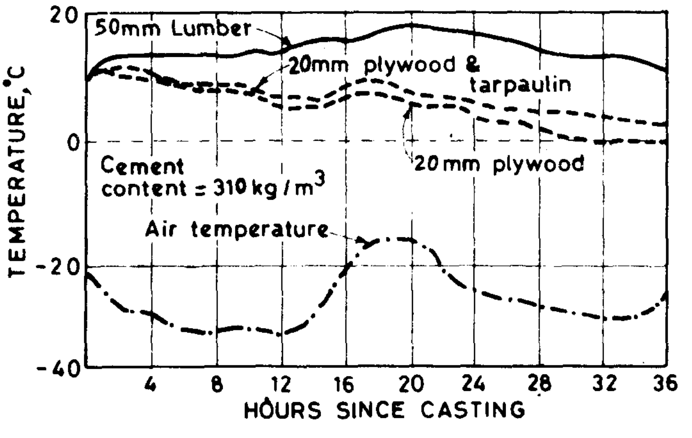


Fig. 54 Form Protection — 90 cm Concrete Cube, Thermocouple at Centre of Face (Next to Form)

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SECTION 8

TESTING OF CONCRETE MIXES

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SECTION 8 TESTING OF CONCRETE MIXES

8.0 Concrete is required to be tested in both fresh and hardened states. Fresh concrete is tested for workability to determine its capacity for satisfactory placing. The workability test also gives information about the consistency of the concrete mix such that the concrete can be transported, placed, compacted and finished easily and without segregation. The analysis of fresh concrete is required to judge the stability that is to identify segregation of the concrete mix, uniformity in mixing and to determine the proportions of the ingredients of concrete actually used. Tests on setting time of concrete are required to determine the time up to which it can be vibrated and to decide the rate of movement of formwork, for example, in slip form construction.

The testing of hardened concrete specimens is required for checking the quality and compliance with the specifications. Compression test on concrete cubes at the age of 28 days is done for these purposes. Additional tests include those for flexural strength, tests by accelerated curing and tests on cored samples.

8.1 Sampling and Testing of Concrete — A 'sample' is a statistical term which is used in the engineering sense to mean a small portion of concrete taken to represent the whole. Compliance with the specified characteristic strength of concrete (defined in *4.1.3*) is judged by compression test on standard cubes ($15 \times 15 \times 15$ cm) prepared from such sample of concrete. The method of sampling fresh concrete for the preparation of cubes is given in IS : 1199-1959[1]. In order to get a relatively quicker idea of the quality of concrete, optional tests, namely, modulus of rupture on beams at 72 ± 2 hours or at 7 days; or compressive strength test at 7 days on cubes may be carried out in addition to 28-days compressive strength test. The requirements of these tests are given in Table 5 of IS : 456-1978[2].

8.2 Frequency of Sampling — Taking into account the theory of probability, sampling frequency of fresh concrete is based on statistical principles and is adopted to ensure that each batch of concrete shall have a reasonable chance of being subjected to test. The sampling should be spread over the entire period of concreting and cover all the mixing units. IS : 456-1978[2] lays down the frequency of sampling which is related to the quantity of concrete involved and the specified frequency is to be applied separately for each grade of concrete. It is necessary that at least one sample shall be taken from each shift of concreting. Higher rates of sampling may be required in the beginning of the work in order to establish the levels of quality quickly. Also, in critical elements/portions of the structures, a higher rate of sampling is desirable. 'Critical elements/portions' are those elements which are regarded as critical by the designer. However, for relatively small works and unimportant buildings and works in which quantity of concrete is less than 15 m³, the strength test may be waived by the engineer-in-charge at his discretion[2].

8.3 Workability Tests of Fresh Concrete — Workability of concrete should be measured at frequent intervals (at least once for each shift) during the progress of work in the field, by means of slump test or compacting factor test or Vee-Bee time consistency test as appropriate (*see* IS : 1199-1959[1]). Out of these, in actual site conditions slump test is more common for medium strength concretes and compaction factor test for high-strength concretes. The significance of these test methods has already been discussed in detail in Section 3. The Kelly ball penetration test as given in ASTM Standard C 360-1963[3] may also be used, in addition to the slump test.

If the proportions of materials are properly followed and the workmanship is satisfactory, the results should not differ by more than the following tolerances for different workability tests:

Slump:	± 25 mm or $\pm$ one-third of the required value, whichever is less
Ball penetration:	± 12 mm
Compacting factor:	± 0.03, where the required value is 0.90 or more;

	±0.04, where the required value is less than 0.90 but more than 0.80;
	±0.05, where the required value is 0.80 or less.
Vee-Bee time:	±3 seconds or ± one-fifth of the required value, whichever is greater.

8.4 Analysis of Fresh Concrete — Analysis of fresh concrete enables the determination of mix proportions of concrete that is proportions of cement, aggregate and water content. Although it has potentials of becoming useful tools of quality control, at present IS : 4634-1968[16] specifies such a test only for determining the efficiency of concrete mixers.

In the method of analysis described in IS : 1199-1959[1] (Buoyancy method), the analysis has to be supplemented by tests of specific gravity and water absorption on fine and coarse aggregates. The coarse and fine aggregates are separated by wet sieving through sieves of appropriate openings. The cement content of a sample of concrete is determined by difference in weight in water of the concrete and aggregates so separated. The water content in the mix is determined by the difference in the weight of concrete in air and the sum of weights of coarse and fine aggregates and cement as determined. Although there are not enough data on the accuracy of this method, in case of similar buoyancy methods, it is claimed that the solid components of fresh concrete can be measured to ±1 percent of the total concrete and the water to ±2 percent of the total concrete[4]. The method takes about 2 hours and requires fairly high degree of experience and skill.

8.5 Measurement of Air Content — Air in concrete is of two types:

a) that purposely entrained to promote workability and reduce segregation and bleeding and to increase frost-resistance of concrete, and

b) unwanted air due to imperfect compaction or the drying out of excessive mixing water.

The air content of non-air-entrained concrete is the casual air which is automatically entrained even though concrete is compacted as thoroughly as possible, the amount depending on maximum size of aggregate used.

There are three methods of measuring the total air content of fresh concrete:

a) the gravimetric method,

b) the volumetric method, and

c) the pressure method.

The pressure method (*see* IS : 1199-1959[1]) provides the most dependable and accurate method of determining the air content of concrete.

The air content of concrete is usually required to be determined for air-entrained concrete. The percentage of air content determined from individual samples taken at the point of placing the concrete and representative of any given batch of concrete should be within ± 1.5 of the required value. The average percentage air content from any four consecutive determinations from separate batches should be within ± 1.0 of the required value.

8.6 Setting Time of Concrete — The setting time of concrete is determined by the penetration resistance apparatus (spring reaction type). The details of the test method are given in IS : 8142-1976[5]. The method measures the load necessary for a needle to penetrate a fixed distance into the cement mortar. Because of the interference from large particles of coarse aggregate, the method is not directly applicable to normal concrete. For determining the setting time of concrete, this method is used indirectly on cement mortar sieved from concrete. When there is a strict correspondence of mixing procedure, the penetration resistance values of cement mortar sieved from concrete have been shown to correspond to those obtained from equivalent mortars where the coarse aggregate is absent[6].

Experiments[6] suggest that a penetration resistance of 3.5 N/mm² on the sieved mortar corresponds to the so called 'vibration limit' that is the point during the hardening of the concrete at which the concrete can no longer again be made plastic by revibration. The time elapsed after adding water to reach a penetration resistance value of 3.5 N/mm² can, therefore, give an approximate guide to the time available for avoiding the formation of cold joints between successive layers of concrete. At a penetration resistance value of about 27.6 N/mm², the concrete has a compressive strength of almost 0.7 N/mm² and is considered to have hardened. Thus the initial and final setting times have been defined by IS : 8142-1976[5], by respective penetration resistance value as follows:

> *Initial setting time* — The elapsed time, after initial contact of cement and water, required for the mortar (sieved from the concrete) to reach a penetration resistance of 3.43 N/mm².

> *Final setting time* — The elapsed time, after initial contact of cement and water, required for the mortar (sieved from the concrete) to reach a penetration resistance of 26.97 N/mm².

8.7 Tests for Strength — Tests for compressive strength of concrete should be conducted on the samples (cube specimens) obtained from the frequency of sampling. Usually, it is necessary to test at 28 days, samples (each sample consisting of three 15 cm cubes) prepared and cured in accordance with IS : 516-1959[7]. Additional tests may be carried out by accelerated curing of concrete as in accordance with IS : 9013-1978[8] or under normal curing at 7 days as specified by IS : 456-1978[2], in order to get a quicker idea of the quality, but the acceptance of concrete will be only on the basis of 28-day compressive strength.

The test strength of the sample shall be the average of the strength of three specimens, provided the individual variation is not more than ± 15 percent of the average. Taking the test specimen in sets of three from a batch and using the average value, reduces the testing error considerably, but usually the effect of testing error on the standard deviation value is relatively smaller.

In the field, samples of fresh concrete can be obtained either from the mixers or at the time of deposition. Approximately, equal portions of concrete from three different times during its discharge from the mixer or from at least five well distributed positions during deposition are required to be taken and test specimens prepared from the composite sample of fresh concrete. The samples are required to be demoulded after storage for 24 ± ½ hour at temperature within the range of 22 to 32°C, and thereafter stored in clean water at a temperature of 24 to 30°C, until they are transported to the laboratory for testing. Since temperature of moist curing influences the compressive strength of concrete, storage at temperatures beyond this range is likely to affect the test results. Specimens, when received in the laboratory are to be stored in water at a temperature of 27 ± 2°C until the time of test. It is necessary to test the specimens in a saturated conditions because concrete specimens tested in a dry state are likely to show an increase in strength[9].

Two criticisms can be levelled against the usefulness of the 28-day cube tests. The first is that the results being available only after 28 days, they do not offer scope of quality control of concrete at the fresh stage and soon after placing. Consequently, it does not allow a judgement to be made before more concrete is subsequently placed on top of it and any remedial or corrective measures, if required, is not possible without affecting the subsequent placement. Mainly from these considerations, accelerated strength tests are becoming popular in which the concrete specimens are subjected to moist curing at elevated temperatures (warm water method) or curing in water at its boiling point (boiling water method) under predetermined curing regimes. IS : 9013-1978[8] prescribes two such methods for quality control purposes and it is found that statistically significant correlation exists between compressive strength of such accelerated cured concrete and those cured under normal 28-day curing. Such accelerated test results become available within about 24 to 28 hours after casting. The correlation is likely to be affected by the materials and mix proportions used and it is advisable to establish appropriate correlation curves for the materials and mix proportions to be used at the site.

Tests with Indian cements show that the relationship between strength of concrete specimens with accelerated curing and normal curing for 28 days is not significantly affected by variations in the physical properties and chemical composition of ordinary Portland cement[10,11,12].

Another limitation of the results of tests on 28-day cube specimens is that even when cured under controlled conditions of temperature and humidity, it does not indicate the actual compressive strength of concrete in the structures. This is because of a number of intrinsic and environmental parameters which are different in case of concrete in the cube specimens and in the structures. It is held that the most reliable measure of the actual strength of concrete in the structures can be obtained by testing specimens of hardened concrete obtained from the structure by core drilling[13]. The procedure of obtaining cored samples from hardened concrete is described in IS : 1199-1959[1] and method of their testing in IS : 516-1959[7]. Such core testing is usually resorted to when the 28-day cube results do not meet the specified acceptance criteria, when the cube tests had not been taken or the results are disputed, or when the design load on the structure is to be enhanced sometimes during its service life to accommodate some modifications in the usage and occupancy of the structure than originally proposed for.

There are a number of factors which influence the measured strength of core specimens, which have been discussed in detail in Reference 13. Cored samples are generally cylinderical in shape, with 150 mm diameter; alternatively 100 mm diameter is also permitted, but the diameter should not be less than three times the nominal maximum size of aggregates. The length to diameter (L/D) ratio is required to be 2. For the ratio less than 2 but not less than 0.95 (this being permitted), the indicated strength increases and necessary correction factors are to be applied to express the indicated strength to that of $L/D = 2$. Such a set of correction factors are given in IS : 516-1959. The influence of the size of the specimens having identical L/D ratio on the compressive strength is not very clear; generally smaller diameter specimens are expected to indicate higher strength. The earlier version of the code for plain and reinforced concrete (IS : 456-1964) stated that compressive strength of 100 mm cubes was expected to be 10 percent higher than those of 150 mm cubes; but in case of core specimens, no correction due to size of core is envisaged in IS : 516-1959[7]. Direction of drilling, presence of reinforcement and age of concrete all influence the indicated strength but precise estimation of these effects is difficult to make. The core strength is ultimately required to be expressed in terms of equivalent cube strength for acceptance purposes. For this purpose, the strength of 150 mm diameter and 300 mm high core specimens can be taken to be 0.8 times that of 150 mm cube specimens.

Apart from those factors intrinsic differences in the qualities of concrete in the structure from which core specimens are obtained and that in the companion cube specimens, exist because of differences in the manner of placing, compaction and curing. As a result, the strength of hardened concrete in the structure is expected to be within a range of 55 to 80 percent of the cube strength[13]. IS : 456-1978[2] stipulates that for acceptance purposes, the average equivalent cube strength of three core samples shall be at least 85 percent of the characteristic strength for the grade of concrete at the corresponding age.

The above discussion pertains to compressive strength of concrete. In some instances, tensile strength of concrete is also required to be measured. The tensile strength is determined either by flexure test (*see* IS : 516-1959[7]) or split cylinder test (*see* IS : 5816-1970[14]). Their relationships with compressive strength of concrete has been discussed earlier in Section 3.

8.8 Analysis of Hardened Concrete — Analysis of cement content in hardened concrete is frequently required, whenever concrete has either exhibited low strength or any other inadequate performance. Although different methods like microscopic, petrographic and even nucleonic are available and new methods are being developed, only chemical methods are recognized in the national standards of most countries, including IS : 1199-1959[1].

In all methods of analysis of hardened concrete, sample selection plays an important part. Since only a small quantity goes in the analysis, it should be as representative of the concrete in question as possible. For this reason, several samples of 4-5 kg are taken and the representative sample is obtained by repeated quartering. Samples of cement and aggregate used in the constructions, if available, aid in the analysis and yield more accurate results.

The procedure prescribed in IS : 1199-1959[1] involves determination of soluble silica (SiO_2) in the mortar fraction which is supposed to be resulting from cement alone. The sample is broken up, crushed and then reduced to a fineness of IS Sieve of aperture size 106 micron to 75 micron and soluble silica content determined by treating it with hydrochloric acid and sodium hydroxide solutions. Assuming that the soluble silica content in the cement to be 21.40 percent by mass, the quantity of cement is calculated.

The relevant ASTM method C85[15] also permits determination of soluble lime (calcium oxide) by leaching with acid solutions and the cement content being determined on the assumption that cement contains lime to the extent of 63.5 percent. In addition, soluble silica basis as above is also permitted.

The above methods involve two assumptions, none of which may be strictly correct. First method assumes the relative contents of silica and lime in the cement which vary considerably between cements; secondly the methods assume that aggregates do not contain soluble silica (on silica basis) or soluble lime (lime basis). If the samples of cement and aggregates are available, both these assumptions can be checked and necessary corrections applied which improve the accuracy. In the absence of these, the estimated cement content may be in error by 10 to 20 percent from the true cement content. These methods are applicable only in case of ordinary Portland cement and are not applicable to Portland slag cement or Portland pozzolana cement, unless blank samples of such blended cements actually used are available at the time of analysis. This is because slags and some reactive pozzolana can release silica, some other fly ashes can release lime.

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