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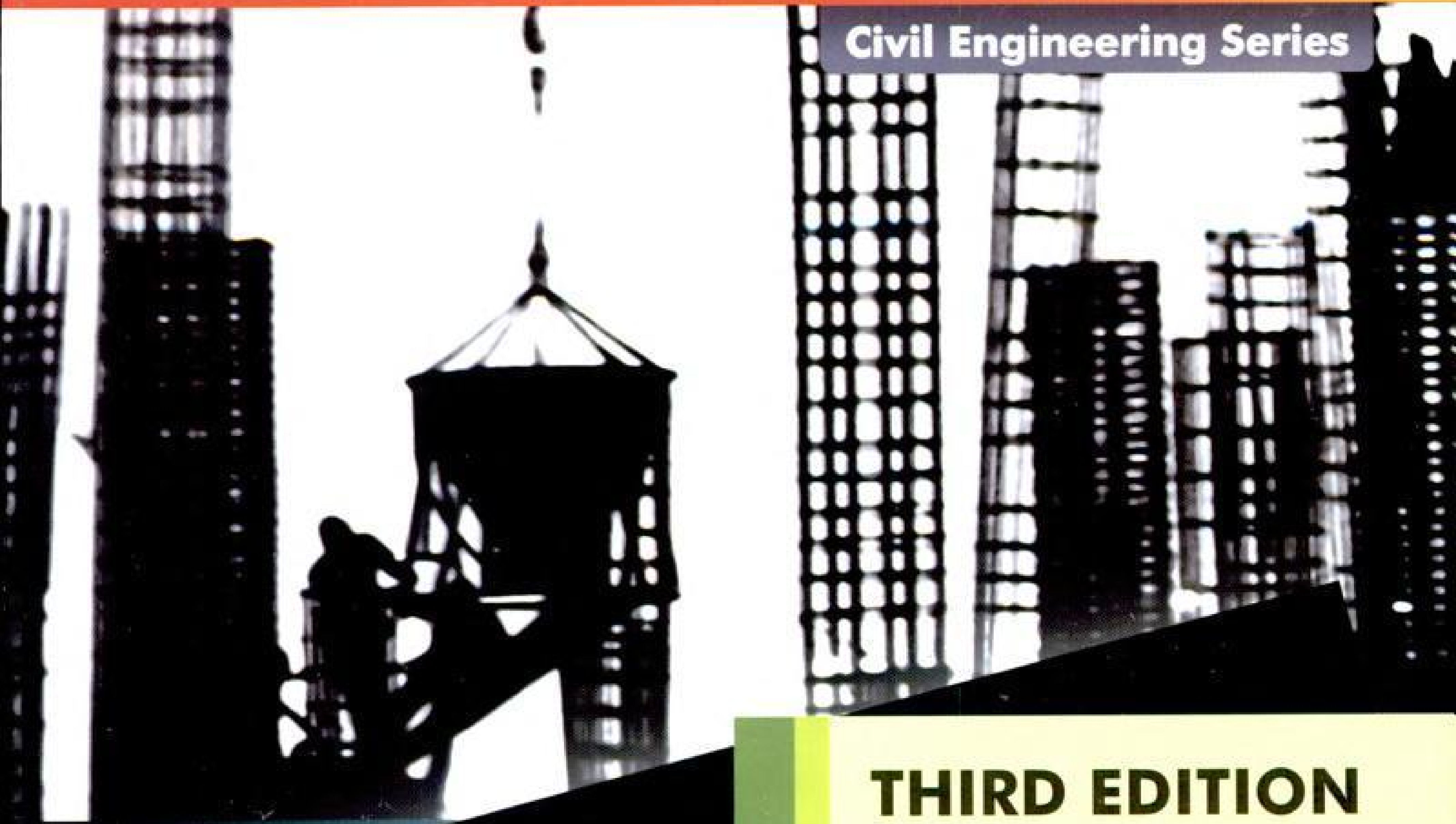

THIRD EDITION

Concrete Technology

M L GAMBHIR

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Preface to the Third Edition

Encouraged with the universal acceptance of the second edition by the engineering community, and excellent positive feedback from the readers, the author has endeavoured to revise the book. The book has been completely rewritten and updated in the light of the latest Indian Standards Codal provisions. The aim has been to bring it to conformity with the rapidly changing field of concrete technology and to attain a state-of-the-art status. The long-awaited and much-demanded new chapter on High Performance Concrete has now been added. In the chapter on Special Concretes, detailed information on roller compacted, and slurry infiltrated fibre concretes have been provided. In the chapter on Admixtures and Mineral Additives emphasis is placed on superplasticizers, fly ash and silica fume. The present day methods of production of concrete namely pumped and ready-mixed concretes have been described in detail. Methods for proportioning of High Performance Concrete are also given. In the chapter on Properties of Hardened Concrete, more information is given on the durability aspects of hardened concrete whereas in the chapter on Inspection and Testing of concrete structures, testing of the durability parameters of hardened concrete has been included. The ever-increasing need to strengthen the deficient, earthquake-devasted and fire-damaged structures has prompted the author to include the state-of-the-art technologies of strengthening with composite laminates.

To help the readers gain an in-depth understanding of the principles of concrete technology, the objective-type question bank has been enlarged. A model question paper has been added for the students preparing for competitive examinations.

It is hoped that this enlarged edition will be as acceptable to the engineering fraternity as its previous editions have been. The author feels that the book will serve as a standard text for the graduate and postgraduate engineering students and as an essential source of reference material to the researchers and practising engineers.

This new edition owes much to the useful feedback from the students, researchers and practising engineers to whom the author expresses his deep sense of gratitude. The author is especially grateful to his daughter Er. Neha Gambhir for her useful contribution, and his wife for continued cooperation and help in bringing out this edition.

M L Gambhir

Preface to the First Edition

The rapid progress and changes in all areas of concrete technology in the recent past have motivated me to prepare this text to suit the needs of undergraduate and graduate courses of various engineering colleges and universities. Most of these institutions have introduced separate courses in concrete technology. The main purpose of this text is to provide the readers with the basic principles, and sufficient information on the state of art relating to all facets of manufacturing and production processes in the making of structural concrete. I am of the opinion that although undergraduate students can depend upon one textbook, at the same time they must realize that a lot more literature exists on the subject. A short bibliography is included at the end of the book which will provide the discerning student with the source for more detailed information and will encourage students' interest in supplementary reading. To cater to the needs of the undergraduate level course more emphasis is laid on the fundamentals and practice. A little bit of repetition has been intentionally allowed to facilitate the understanding of the subject matter.

In addition to the traditional concrete engineering topics dealing with the principles of concrete and concrete-making materials, the current state of art of special concretes and concreting techniques has been included. This will introduce the engineering faculties, practising engineers, architects, research workers, and contractors to the new developments in the production of quality concrete. Many topics of particular interest have been included because they represent areas of recent technological advances like fibre-reinforced concrete, ferrocement, fibre-reinforced ferrocement, polymer impregnated concrete, nondestructive testing, modern concepts of quality concrete, etc. The final chapter introduced the repair techniques used for concrete structures as a logical extension of the principles discussed in the preceding chapters.

As most of the information has been derived from many sources including those specially mentioned in the bibliography at the end of text, I wish to express my indebtedness to these individuals and publications contributing the information. Acknowledgments and thanks are recorded to the Bureau of Indian Standards (erstwhile Indian Standards Institution) to quote certain regulations.

I record my gratitude to Dr Jai Krishna, FNA, formerly Vice Chancellor, University of Roorkee, and Ex-president of International Association for Earthquake Engineering (IAEE) for kindly writing the foreword to the book and for his encouragement. I acknowledge with gratitude the assistance received from many quarters in the preparation of the book. Foremost thanks are due to Dr S Krishnamurthy, Professor of Civil Engineering at the Indian Institute of Technology, New Delhi, for critically reviewing the manuscript and giving valuable suggestions for the improvement of the text. Special thanks go to Dr N C Nigam, Director, Thapar Institute of Engineering and Technology, Patiala, for his encouragement and motivations.

Finally, I am grateful to my wife and children for their forbearance and patience in putting up with the inconveniences when I was engaged in the preparation of this text.

ML Gambhir

CHAPTER

1

Concrete as Construction Material

1.1 INTRODUCTION

Concrete is the most widely used man-made construction material in the world, and is second only to water as the most utilized substance on the planet. It is obtained by mixing cementitious materials, water and aggregates (and sometimes admixtures) in required proportions. The mixture when placed in forms and allowed to cure, hardens into a rock-like mass known as concrete. The hardening is caused by chemical reaction between water and cement and it continues for a long time, and consequently the concrete grows stronger with age. The hardened concrete may also be considered as an artificial stone in which the voids of larger particles (*coarse aggregate*) are filled by the smaller particles (*fine aggregate*) and the voids of fine aggregates are filled with cement. In a concrete mix the cementitious material and water form a paste called *cement–water paste* which in addition to filling the voids of fine aggregate, coats the surface of fine and coarse aggregates and binds them together as it cures, thereby cementing the particles of the aggregates together in a compact mass.

The *strength*, *durability* and other characteristics of concrete depend upon the properties of its ingredients, on the proportions of mix, the method of compaction and other controls during placing, compaction and curing. The popularity of the concrete is due to the fact that from the common ingredients, it is possible to tailor the properties of concrete to meet the demands of any particular situation. The advances in concrete technology have paved the way to make the best use of locally available materials by judicious mix proportioning and proper workmanship, so as to produce concrete satisfying *performance requirements*.

The key to producing a strong, durable and uniform concrete, i.e. *high performance concrete* lies in the careful control of its basic and process components. These are:

Cement Portland cement, the most widely used cementitious ingredient in present day concrete comprises phases that consist of compounds of calcium, silicon, aluminum, iron, and oxygen.

Aggregate These are primarily naturally-occurring, inert granular materials such as sand, gravel, or crushed stone. However, technology is broadening to include the use of recycled materials and synthetic products.

Water The water content and the minerals and chemicals dissolved in it are crucial to achieving quality concrete.

Chemical Admixtures These are the ingredients in concrete other than Portland cement, water, and aggregates that are added to the mixture immediately before or during mixing to reduce the water requirement, accelerate/retard setting, or improve specific durability characteristics.

Supplementary Cementitious Materials Supplementary cementitious materials, also called mineral additives, contribute to the properties of hardened concrete through hydraulic or pozzolanic activity. Typical examples are natural pozzolans, fly ash, ground granulated blast-furnace slag, and silica fume. After concrete is placed, these components must be cured at a satisfactory moisture content and temperature must be carefully maintained for a sufficiently long time to allow adequate development of the strength of the concrete.

The factors affecting the performance of concrete are shown in Fig. 1.1. The concept of treating concrete in its entity as a building material rather than its ingredients is gaining popularity. The user is now interested in the concrete having the desired properties without bothering about the ingredients. This concept is symbolized with the progress of *ready mixed concrete* industry where the consumer can specify the concrete of his needs, and further in the pre-cast concrete industry where the consumer obtains finished structural components satisfying the performance requirements.

The various aspects covered in the following chapters are—materials, mix proportioning, elements of workmanship, e.g. placing, compaction and curing, methods of testing and relevant statistical approach to quality control. The discussions on these aspects are based on the appropriate provisions in the Indian Standard Codes.

Concrete has high compressive strength, but its tensile strength is very low. In situations where tensile stresses are developed the concrete is strengthened by steel bars or short randomly distributed fibers forming a composite construction called *reinforced cement concrete (RCC)* or *fiber reinforced concrete*. The concrete without reinforcement is termed *plain cement concrete* or simply as concrete. The process of making concrete is called *concreting*. Sometimes the tensile stresses are taken care of by introducing compressive stresses in the concrete so that the initial compression neutralizes the tensile stresses. Such a construction is known as *prestressed cement concrete* construction.

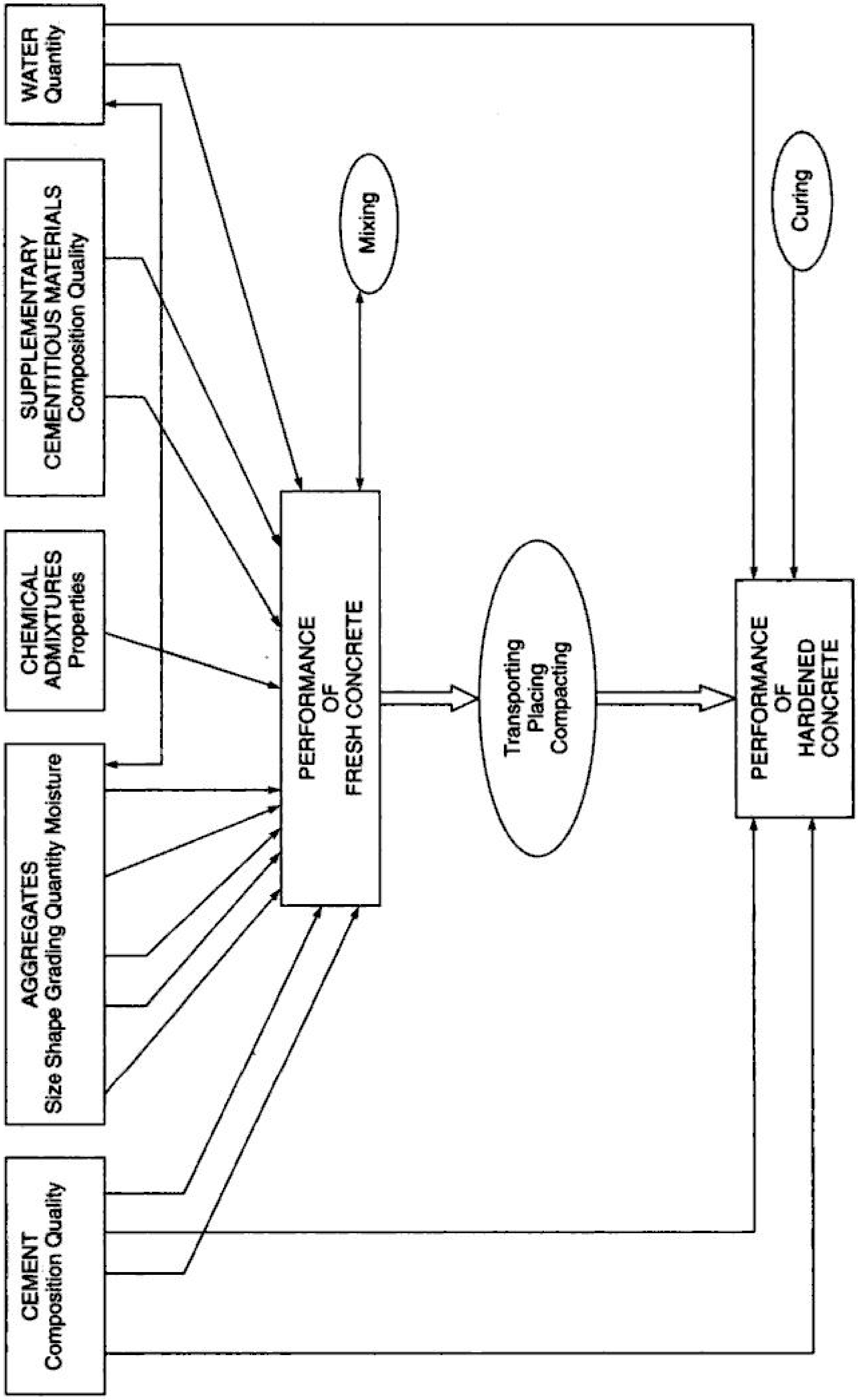

Fig. 1.1 Factors affecting performance of concrete

1.2 CLASSIFICATION OF CONCRETE

As mentioned earlier the main ingredients of concrete are cement, fine aggregate (sand) and coarse aggregate (gravel or crushed rock). It is usual to specify a particular concrete by the proportions (by weight) of these constituents and their characteristics, e.g. a 1:2:4 concrete refers to a particular concrete manufactured by mixing cement, sand and broken stone in a 1:2:4 ratio (with a specified type of cement, water-cement ratio, maximum size of aggregate, etc.). This classification specifying the proportions of constituents and their characteristics is termed as *prescripitive specifications* and is based on the hope that adherence to such prescripitive specifications will result in satisfactory performance. Alternatively, the specifications specifying the requirements of the desirable properties of concrete such as strength, workability, etc. are stipulated, and these are termed as *performance oriented specifications*. Based on these considerations, concrete can be classified either as *nominal mix concrete or designed mix concrete*. Sometimes concrete is classified into *controlled concrete* and *ordinary concrete*, depending upon the levels of control exercised in the works and the method of proportioning concrete mixes. Accordingly, a concrete with ingredient proportions fixed by designing the concrete mixes with preliminary tests are called *controlled concrete*, whereas ordinary concrete is one where *nominal mixes* are adopted. In IS:456–2000 there is nothing like uncontrolled concrete: *only the degree of control varies from very good to poor or no control*. In addition to mix proportioning, the *quality control* includes *selection 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*.

1.3 PROPERTIES OF CONCRETE

Concrete making is not just a matter of mixing ingredients to produce a plastic mass, but *good concrete* has to satisfy *performance requirements* in the *plastic* or *green* state and also the hardened state. In the plastic state the concrete should be workable and free from *segregation* and *bleeding*. Segregation is the separation of coarse aggregate and bleeding is the separation of cement paste from the main mass. The segregation and bleeding result in a *poor quality concrete*. In its *hardened state concrete* should be *strong*, *durable*, and *impermeable*; and it should have *minimum dimensional changes*.

Among the various properties of concrete, its *compressive strength* is considered to be the most important and is taken as an *index of its overall quality*. Many other properties of concrete appear to be generally related to its compressive strength. These properties will be discussed in detail later in the book.

1.4 GRADES OF CONCRETE

Concrete is generally graded according to its compressive strength. The various *grades of concrete* as stipulated in IS:456–2000 and IS:1343–1980 are given in Table 1.1. In the designation of concrete mix, the letter M refers to the mix and the number to the specified characteristic strength of 150 mm work cubes at 28 days, expressed in MPa (N/mm^2). The concrete of grades M5 and M7.5 is suitable for lean concrete bases, simple foundations, foundations for masonry walls and other simple or temporary reinforced concrete constructions. These need not be designed. The concrete of grades lower than M15 is not suitable for reinforced concrete works and grades of concrete lower than M30 are not to be used in the prestressed concrete works.

Table 1.1 *Grades of Concrete*

Group	*Ordinary concrete*			*Standard concrete*							*High strength concrete*				
Grade designation	M 10	M 15	M 20	M 25	M 30	M 35	M 40	M 45	M 50	M 55	M 60	M 65	M 70	M 75	M 80
Specified characteristic strength at 28 days, MPa	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80

1.5 ADVANTAGES OF CONCRETE

Concrete as a construction material has the following advantages.

(i) Concrete is economical in the long run as compared to other engineering materials. Except cement, it can be made from locally available coarse and fine aggregates.

(ii) Concrete possesses a high *compressive strength*, and the corrosive and weathering effects are minimal. When properly prepared its strength is equal to that of a hard natural stone.

(iii) The green or newly mixed concrete can be easily handled and moulded or formed into virtually any shape or size according to specifications. The formwork can be reused a number of times for similar jobs resulting in economy.

(iv) It is strong in compression and has unlimited structural applications in combination with steel reinforcement. Concrete and steel have approximately equal *coefficients of thermal expansion*.

(v) Concrete can even be sprayed on and filled into fine cracks for repairs by the *guniting* process.

(vi) Concrete can be pumped and hence it can be laid in difficult positions also.

(vii) It is durable and fire resistant and requires very little maintenance.

These qualities explain why concrete is extensively used in the construction of skyscrapers, superhighways, railways, airfields, buildings, water-retaining structures, docks and harbours, dams, bridges, bunkers and silos, etc.

1.6 DISADVANTAGES OF CONCRETE

Following are the disadvantages of concrete.

(i) Concrete has low *tensile strength* and hence cracks easily. Therefore, concrete is to be reinforced with steel bars or meshes or fibers.

(ii) Fresh concrete shrinks on drying and hardened concrete expands on wetting. Provision for contraction joints has to be made to avoid the development of cracks due to *drying shrinkage* and *moisture movement.*

(iii) Concrete expands and contracts with the changes in temperature. Hence expansion joints have to be provided to avoid the formation of cracks due to *thermal movement.*

(iv) Concrete under sustained loading undergoes *creep,* resulting in the reduction of *prestress* in the prestressed concrete construction.

(v) Concrete is not entirely impervious to moisture and contains soluble salts which may cause *efflorescence.*

(vi) Concrete is liable to disintegrate by *alkali* and *sulphate attack.*

(vii) The lack of *ductility* inherent in concrete as a material is disadvantageous with respect to *earthquake resistant design.*

1.7 CONCEPT OF QUALITY CONTROL

Quality in general terms is totality of features and characteristics of a product or service that bear on its ability to satisfy the stated or implied needs. The stated or implied needs are those derived by balanced excellence and equity within the sustainable regime and in the given socio–techno–economic scenario. The quality management has evolved over the period through:

- Policing quality—acceptance and Rejection through inspection and assessment by user.
- Judging quality—confidence building through third party judgement.
- Fostering quality—ensuring quality of the final product by attending to quality at all intermediary stages such as in Certification Marking Schemes.

Concrete, generally manufactured at the site, is likely to have variability of performance from batch to batch and also within the batch. The magnitude of this variation depends on several factors, such as the variation in the quality of constituent materials, variation in mix proportions due to batching process, variations in the quality of batching and mixing equipment available, the quality of overall workmanship and supervision at the site, and variation due to sampling and testing of concrete specimens.

The above variations are inevitable during production to varying degrees. For example, the cements from different batches or sources may exhibit different strengths. The *grading* and *shape of aggregates* even from the same source varies widely. Considerable variations occur partly due to the quality of the plant available and partly due to the efficiency of operation. Some of the variations in test results are due to variations in sampling, making, curing and testing the specimen even when carried out in terms of relevant specifications.

The *quality control* of concrete is thus to reduce this variation and to produce *concrete of uniform quality* consistent with specified minimum performance requirements which can be achieved by good workmanship and maintainance of the plant at peak efficiency.

The concrete industry strives at making 'quality', a way of life and a way of management through *Quality Systems Approach* covering all aspects of ISO 9000 series.

1.8 CONCRETE INDUSTRY

Since concrete is an affordable and reliable material, which is extensively used throughout in the infrastructure of a nation's construction, industrial, transportation, defense, utility, and residential sectors, it has become a huge industry. India produces about 170 million cubic meters of concrete annually. Slightly more than a tonne of concrete is produced each year for every human being on earth.

In addition to cement and concrete manufacturing, the concrete construction industry includes aggregate and material suppliers, designers, haulers, constructors, and repair and maintenance companies. Over 40 million jobs relate to the concrete industry alone. While there is significant diversity of services within this industry, all facets of the concrete industry share a common objective—a sincere *desire to deliver a high-quality, long-lasting, competitive, eco-friendly* and *sustainable product.*

Sustainability

Sustainable Regime may be defined as **that regime in which the endeavors are towards meeting the needs of the present generation without compromising the needs of the future generations**.

So far as construction industry is concerned, it has to work within the following strategic framework if it has to be 'sustainable'.

- Responsiveness to environmental regulations, i.e. environmental protection including ecological balancing.
- Material conservation including performance maximization.
- Energy efficiency or conservation.
- Cost effectiveness or cost reduction both initial and life cycle costs.
- Safety assurance.

- Durability and serviceability considerations.
- Manpower development and optimization.
- Ergonomic and aesthetic concerns.
- Total quality management.
- Creation of proper interface with computer-integrated knowledge based systems for technology transfer.

1.9 CHALLENGES FACED BY THE CONCRETE INDUSTRY

Portland cement is the most energy-intensive material produced after steel and aluminum. More than 7 per cent of world's carbon dioxide emissions are attributed to Portland cement. In addition to CO_2 emissions, the burning of Portland cement at high temperature (1450°C) is costly in terms of fossil fuel usage. Moreover, by some estimate concrete industry is largest consumer of natural resources such as water, sand, gravel and crushed rock. Thus for sustainable development it is recognized that considerable improvements are essential in *productivity*, *product performance*, *energy efficiency*, and *environmental performance*. To achieve these objectives will require a concerted and focused effort.

Research in new materials, processing technologies, delivery mechanisms, and applications of information technology, could transform the industry. Greater materials improvements will enable the industry to demonstrate clearly the full spectrum of performance benefits of concrete.

A number of government agencies—NCB, CRI, SERC, CBRI—and CRRI focus on wide variety of concrete research topics. Programmes of several universities and technical institutions are involved in concrete related research. Sponsoring agencies include DST, AICTE, UGC, etc. These agencies sponsor broad-spectrum *concrete research*—basic and applied—to improve concrete and *repair materials* technologies. This research is designed to enable cost-effective application of high-performance concrete with extended service life, and to advance concrete technology by providing a sound materials science base. Additionally, there are numerous other state and central programmes that strive to advance the nation's knowledge of concrete.

To make concrete the most efficient and cost-effective material of construction, will require processing improvements throughout the life cycle of concrete including design, production, transportation, construction, maintenance and repair. The concrete industry is unique in that *process improvements* can crosscut many other industries. Foundry sand, fly ash, silica fume, slag, and other by-products from industries such as aluminum, metal casting, and steel and power generation can be and are used as ingredients in the manufacture of cement and concrete. The concrete industry will have to commit to changes in practices in the materials, design, and construction arenas through the use of materials and systems that improve function, durability, and sustainability.

There is no central resource for performance data and service life of current concrete products. This limits the ability of designers and constructors to communicate life-cycle benefits of concrete products to the user community. Computer-integrated knowledge systems can provide a practical basis for *optimizing concrete* for specific applications by taking technical, economic, and environmental factors into account. Advanced systems models must be developed to show the prediction of performance for any mixture design under a range of environmental conditions lasting over a long period, i.e. over decades and even centuries. Aggregates, cement, repair and maintenance, materials transportation, life-cycle analysis, and other areas can all be readily addressed under this concept.

Due to the fear of failure to meet design criteria, producers, users, and designers are reluctant to shift from tried and proven processes and materials to adopt promising new technologies until long use histories have been substantiated. It is estimated that it takes 15 to 20 years to get a new technology from concept to adoption. Thus the advances in materials and *process technologies* needed to produce *high-performance concrete* are advancing slowly, and are not entering the marketplace quickly.

Portland cement production is the most energy-intensive phase of the concrete production chain as its production requires high process temperatures to produce the necessary chemical transformations. Cement-manufacturing accounts for about 80 per cent of the total concrete industry's power consumption. In addition a large amount of energy is required to transport aggregate and other ingredients to manufacturing sites and to deliver finished products to market. The material transportation costs account for 20 per cent to 50 per cent of the final cost of *ready-mixed concrete*, and delays in material or concrete delivery can create significant labour downtime in concrete construction. Both of these phases of concrete production offer significant opportunities for improvement.

Currently, the industry operates in a *prescriptive* rather than *performance-based environment*. Thus, the full potential of concrete is often not realized. Accordingly, the procurement process for concrete construction and products typically favors the low-cost bidder because no incentives are provided for improved performance. This forces concrete companies to keep costs down and creates a disincentive to investing in research and development. Improved technology can reduce service life costs, prevent premature repairs, and also use less energy.

Process Improvements

The industry can achieve significant improvements in process over the next quarter of a century by:

- using a variety of by-products from other industries as well as *re-cycled concrete* as constituent materials for concrete production;

- using a geomimetic approach to tailor mixture design to specific structural environments;
- achieving *optimal particle size distribution* of the constituent materials;
- manufacturing the cement with less energy and fewer emissions such as nitrous oxide and carbon dioxide, with decreased production of by-product cement kiln dust;
- using accepted techniques and processes to produce *lighter-weight, higher-strength products*, thereby reducing volumetric requirements and making transportation easier and less expensive;
- using advanced systems modeling to predict the performance of concrete for users; and
- adopting automation as standard practice in concrete placement.

Product Performance

As explained earlier concrete is one of the most *durable* and *cost-effective construction materials* used in civil engineering. However, more needs to be done *to improve its performance, reliability, and life-cycle cost-effectiveness*. The diverse applications for concrete have a wide variety of performance requirements. The industry needs critical research to produce *high-performance, cost-effective concrete.*

The industry can pass on the product performance benefits of concrete to the users by:

- using effective, consistent quality assurance/quality control standards throughout the industry;
- making full use of *nondestructive measurements*, sensors, *intelligent curing techniques*, and other technology advances to continuously monitor property performance and to maintain *durability*;
- producing concrete products having concrete strengths of 5 to 10 times that of current levels leading to a reduction in the overall volume of concrete required;
- having a system of shared, consolidated data such as materials, structures, design, and performance databases and using them with *computer-integrated knowledge systems* to demonstrate product quality to customers; and
- making concrete reinforcement more durable through the use of advanced fibers and composites, enhancing the life-cycle benefits of concrete.

Energy Efficiency

Energy efficiency can be improved in all stages of the concrete life cycle. The concrete industry should aim at reducing energy consumption from current levels by 50 per cent per unit of output during the next quarter of a century. This can be achieved by:

- using *bio-based raw materials as fuel sources* in cement making;
- using aggregates that are less energy-intensive to produce;
- using advanced technology to improve process heating for cement making;
- utilizing *cementitious materials* that require less process heating and produce fewer emissions; and
- saving energy by making increased use of recycled waste and by-products, from within the concrete industry and from other industries in concrete manufacturing.

Environmental Performance

Approximately 7 per cent of the world's carbon dioxide (CO_2) emissions are attributable to Portland cement. Carbon dioxide belongs to the so-called greenhouse gases, which contribute to global warming. Out of 450 million tonnes of fly ash that is suitable for use in cement, only a less than 8 per cent is used for cement production. Only a small fraction of the 100 million tonnes of slag produced worldwide each year is utilized as a cement substitute. Nearly 90 per cent of coal ash and metallurgical slag produced today end up in low-value applications or simply undergo disposal by ponding and stockpiling.

To meet sustainable development and environmental goals, *responsiveness to environmental regulations*, and *waste management* should be the part of daily operations in the concrete industry. The industry continually should seek to identify how it can increase its use of environmentally friendly practices and processes. Moreover, the concrete industry must consume a wider range of by-products from other industries to evolve novel concretes for *tailored waste isolation*.

Manpower Development

Since construction industry involves multi-disciplinary inputs, the *manpower development* for such an industry is a challenging task. It involves identification of training needs at each level and discipline, the training methodology, preparation of instruction material, mode of training delivery system, hands-on-experience, etc. which are of highly variable nature depending upon the target group considered. The manpower comprises various levels of understanding or education from top management to skilled and semi-skilled workers. Hence a holistic approach is required for MPD which is currently available through National Council for Cement & Building Materials (NCB) and RTCs. The Construction Infrastructure Development Council (CIDC) dealing with construction, lays emphasis on training of construction craftsman through Construction Traders Training Council (CTTC). CTTC may in turn indentifies network agencies like NCB, NAC, ICI, NICMAR, ACC-RCD, etc.

In order to *achieve* desired *objectives of securing improved quality, productivity* and *efficiency in cement* and *concrete construction*, it is necessary to improve quality, content, nature of education and training within *sustainable* *regime*. It is necessary to look into the objective of designing and placing concrete mix and to compare this ideal with experience. The aim is to minimize the gap between the ideal and the practice. For the given technical quality of the batching and mixing system this objective can be achieved by proper training of personnel.

CHAPTER

2

Concrete Making Materials–I: Cement

2.1 INTRODUCTION

Cement is a well-known building material and has occupied an indispensable place in construction works. There are a variety of cements available in the market and each type is used under certain conditions due to its special properties. A mixture of cement and sand when mixed with water to form a paste is known as *cement mortar* whereas the composite product obtained by mixing cement, water, and an inert matrix of sand and gravel or crushed stone is called *cement concrete*. The distinguishing property of concrete is its ability to harden under water.

The cement commonly used is Portland cement, and the fine and coarse aggregates used are those that are usually obtainable, from nearby sand, gravel or rock deposits. In order to obtain a strong, durable and economical concrete mix; it is necessary to understand the characteristics and behaviour of the ingredients. The ingredients of concrete can be classified into two groups, namely *active* and *inactive*. The active group consists of cement and water, whereas the inactive group comprises *fine* and *coarse aggregates*. The inactive group is also sometimes called the *inert matrix*. In this chapter the ingredients of the active group will be discussed.

Although all materials that go into a concrete mixture are essential, cement is by far the most important constituent because it is usually the delicate link in the chain. The function of cement is, first to bind the sand and coarse aggregates together, and second to fill the voids in between sand and coarse aggregate particles to form a compact mass. Although cement constitutes only about 10 per cent of the volume of the concrete mix, it is the active portion of the *binding medium* and the only scientifically controlled ingredient of concrete.

2.2 PORTLAND CEMENT

Portland cement is an extremely ground material having adhesive and cohesive properties, which provide a binding medium for the discrete ingredients. It is obtained by burning together, in a definite proportion, a mixture of naturally occurring *argillaceous* (containing alumina) and *calcareous* (containing calcium carbonate or lime) materials to a partial fusion at high temperature (about 1450°C). The product obtained on burning, called *clinker*, is cooled and ground to the required fineness to produce a material known as *cement*. Its inventor, Joseph Aspdin, called it Portland cement because when it hardened it produced a material resembling stone from the quarries near Portland in England. During grinding of clinker, gypsum or plaster of Paris ($CaSO_4$) is added to adjust the setting time. The amount of gypsum is about 3 per cent by weight of clinker. It also improves the *soundness* of cement.

Depending upon the location of the cement-manufacturing plant, available raw materials are pulverized and mixed in proportions such that the resulting mixture will have the desired chemical composition. The common calcareous materials are limestone, chalk, oyster shells and marl. The argillaceous materials are clay, shale, slate and selected blast-furnace slag. When limestone and clay are the two basic ingredients, the proportions will be approximately four parts limestone to one part of clay. Certain clays formed during volcanic eruption, known as volcanic ash or pozzolana, found near Italy, have properties similar to that of Portland cement.

Since the raw materials consist mainly of lime, silica, alumina and iron oxide, these form the major constituents of Portland cement also. Depending upon the wide variety of raw materials used in the manufacture of cements, the oxide composition of ordinary Portland cement may be expressed as given in Table 2.1.

Table 2.1 *Oxide Composition of Ordinary Portland Cement*

Oxide	*Percentage*	*Average*
Lime, CaO	60–65	63
Silica, SiO_2	17–25	20
Alumina, Al_2O_3	3–8	6.3
Iron oxide, Fe_2O_3	0.5–6	3.6
Magnesia, MgO	0.5–4	2.4
Sulphur trioxide, SO_3	1–2	1.5
Alkalis, i.e. soda and/or potash, $Na_2O + K_2O$	0.5–1.3	1.0

These oxides interact with each other to form a series of more complex products during fusion. The *compound composition* will be discussed later in the chapter.

The processes used for the manufacture of cement can be classified as *dry* and *wet*. When the basic raw material is rock, it is transported to a large gyratory, or jaw crusher for primary reduction in size (to about 150 mm). It then passes through a smaller crusher or hammer mill where further reduction takes place to a 40 mm size aggregate, and from there it goes to rock storage or stacker. From stacker the crushed rock is fed to a vertical ball mill along with clay or crushed shale. In the *wet process* water is added at this point to obtain a blended mixture of very finely ground raw materials and water, called slurry. The slurry is stored in tanks under constant agitation and fed into huge firebrick-lined rotary kilns. In the *dry process* the raw powdered materials (also called raw meal) which are mixed, and homogenized pass through a series of sophisticated precalcining systems with each system consisting of separate strings of 5 or 6-stage precalcining units and fed to the kiln in the dry state. In the recent past economical, efficient and environmental friendly dry process of cement manufacturing has replaced the more power oriented wet process. In the recent years significant advancements have been made in raw material grinding, pyro-processing, controlled clinker cooling, cement grinding and packing technologies with advanced process control and instrumentation system. The coal consumption in the dry process is approximately one-fourth to that in wet process. Typically, the total consumption of coal in the dry process is 100 kg as against 350 kg in wet process for producing a ton of cement.

The *kilns* are fired with crushed coal or gas from the discharge end under forced draft so that material being fed in advances against the heat blast as the kiln rotates. The kilns are mounted with the longitudinal axis inclined in such a way that the raw material or slurry is fed at the higher end. At about 425 °C, excess water is driven off, and then further along the kiln, at 875 °C, limestone breaks down into calcium oxide and carbon dioxide. Finally, at 1400 °C to 1450 °C, about 10 m from the discharge end, the initial melting stage of material, known as the point of incipient fusion, is reached. Sintering takes place at this point, and a substance having its own physical and chemical properties called *clinker* is formed. *The rate of cooling influences the mineralogy of clinker, i.e. the degree of crystallization, the size of crystals and the amount of amorphous materials.* The mineralogy of clinker influences the hydration and strength properties of cement considerably. A moderate rate of cooling in rotary kiln from 1200 °C to 500 °C in about 15 minutes and from 500° C to normal atmospheric temperature in about 10 minutes results in high strength cements. The cooled clinker is crushed, mixed with about 3 per cent crushed gypsum, and fed into a tube mill and processed through closed circuit grinding where proper particle size distribution is ensured by high efficiency *cyclonic separator*. After initial grinding in tube mill the material moves into high efficiency cyclonic separator, which assures that the ground material has the ideal surface and ideal proportion of particles of sizes between 5 to 30 micron (to the extent of 50 per cent). The finished product known as Portland cement is taken to the storage silos where it is finally bagged in high density polyethylene (HDPE) woven sacks, double

Hessian bitumenized sacks, polyethylene lined jute bags, and four-ply paper bags and transported to stockists and construction sites.

The variations in the chemical and physical properties of cement specially the strength and fineness can be minimized or consistency in quality can be ensured by installation of proper quality control *monitoring systems* and modern sophisticated instrumentation control systems. Computers are widely used for controlling the operations and quality at various stages in cement manufacturing process, i.e. on-line analysers for raw meal quality control and expert systems for controlling operations of raw materials grinding mill, preheaters, precalcinators and clinker coolers, cement grinding and packing sections for achieving smooth operation, uniform quality and maximum output.

The *composition of Portland cement* is rather complicated but basically it consists of the following four main compounds.

Tricalcium silicate (C_3S),	$3CaO{\cdot}SiO_2$
Dicalcium silicate (C_2S),	$2CaO{\cdot}SiO_2$
Tricalcium aluminate (C_3A),	$3CaO{\cdot}Al_2O_3$
Tetracalcium alumino ferrite (C_4AF),	$4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$

The symbols in parentheses are the abbreviations generally used. To the above ingredients is added about 3 per cent gypsum ($CaSO_4$). Depending upon the wide variety of raw materials used in the manufacture of cements, typical ranges of these compounds in ordinary Portland cements may be expressed as given in Table 2.2.

Table 2.2 Compound Composition of Ordinary Portland Cement

Compound	*Percentage by mass in cement*
C_3S	25–50
C_2S	20–45
C_3A	5–12
C_4AF	6–12

Differences in the various types of ordinary Portland cements arise due to the variations in the relative proportions of these compounds in the cement.

2.2.1 Basic Properties of Cement Compounds

The two silicates, namely C_3S and C_2S, which together constitute about 70 to 80 per cent of the cement control the most of the strength giving properties. Upon hydration, both C_3S and C_2S give the same pcoduct called calcium silicate hydrate ($C_3S_2H_3$) and calcium hydroxide. Tricalcium silicate (C_3S) having a faster rate of reaction (Fig 2.1) accompanied by greater heat evolution develops early strength. On the other hand, dicalcium silicate (C_2S) hydrates and hardens slowly and provides much of the ultimate strength. It is likely that both C_3S and

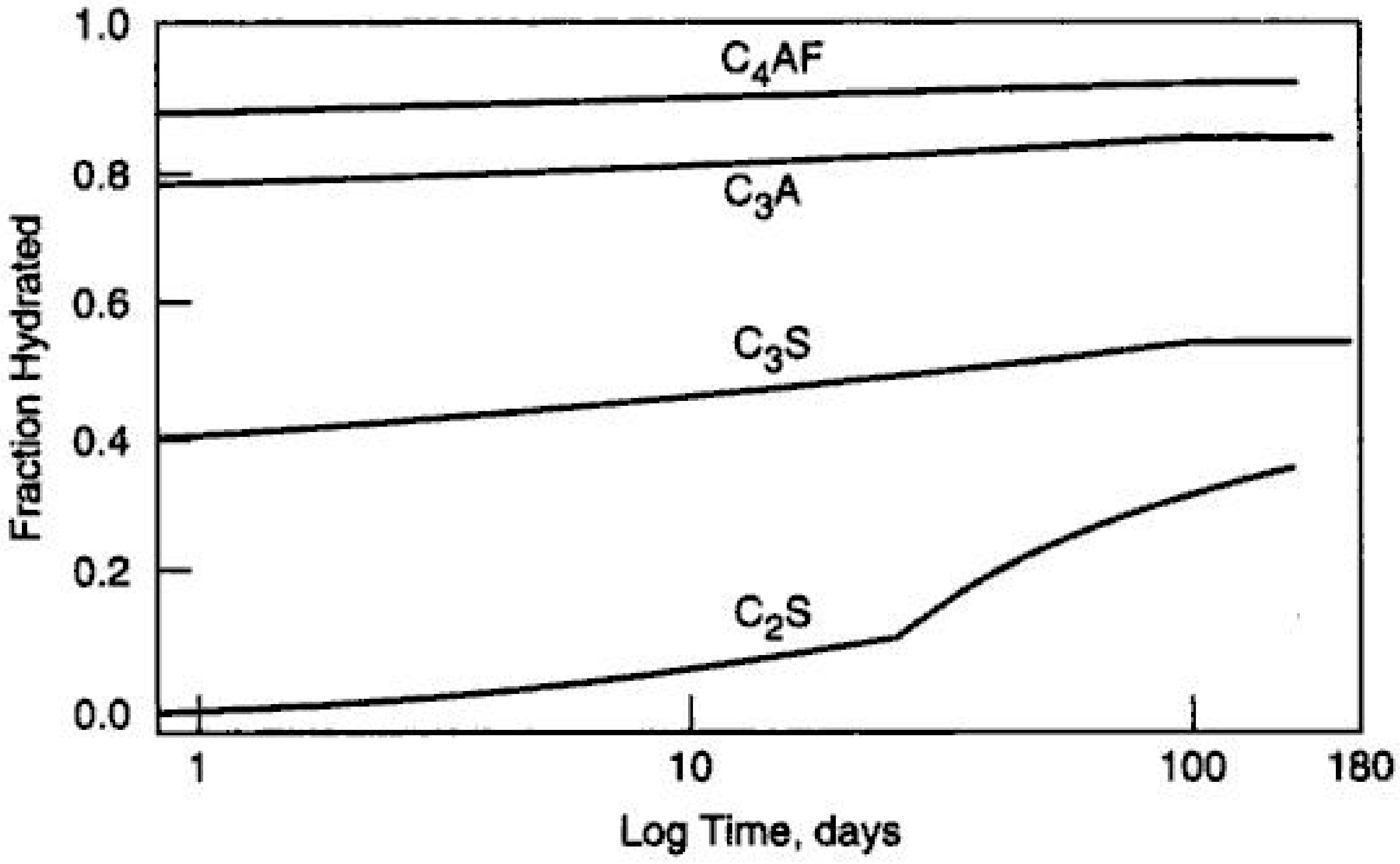

Fig. 2.1 Rate of hydration of pure cement compounds

C_2S phases contribute equally to the eventual strength of the cement as can he seen in Fig. 2.2. C_3S and C_2S need approximately 24 and 21 per cent water by weight, respectively, for chemical reaction but C_3S liberates nearly three times as much calcium hydroxide on hydration as C_2S. However, C_2S provides more resistance to chemical attack.

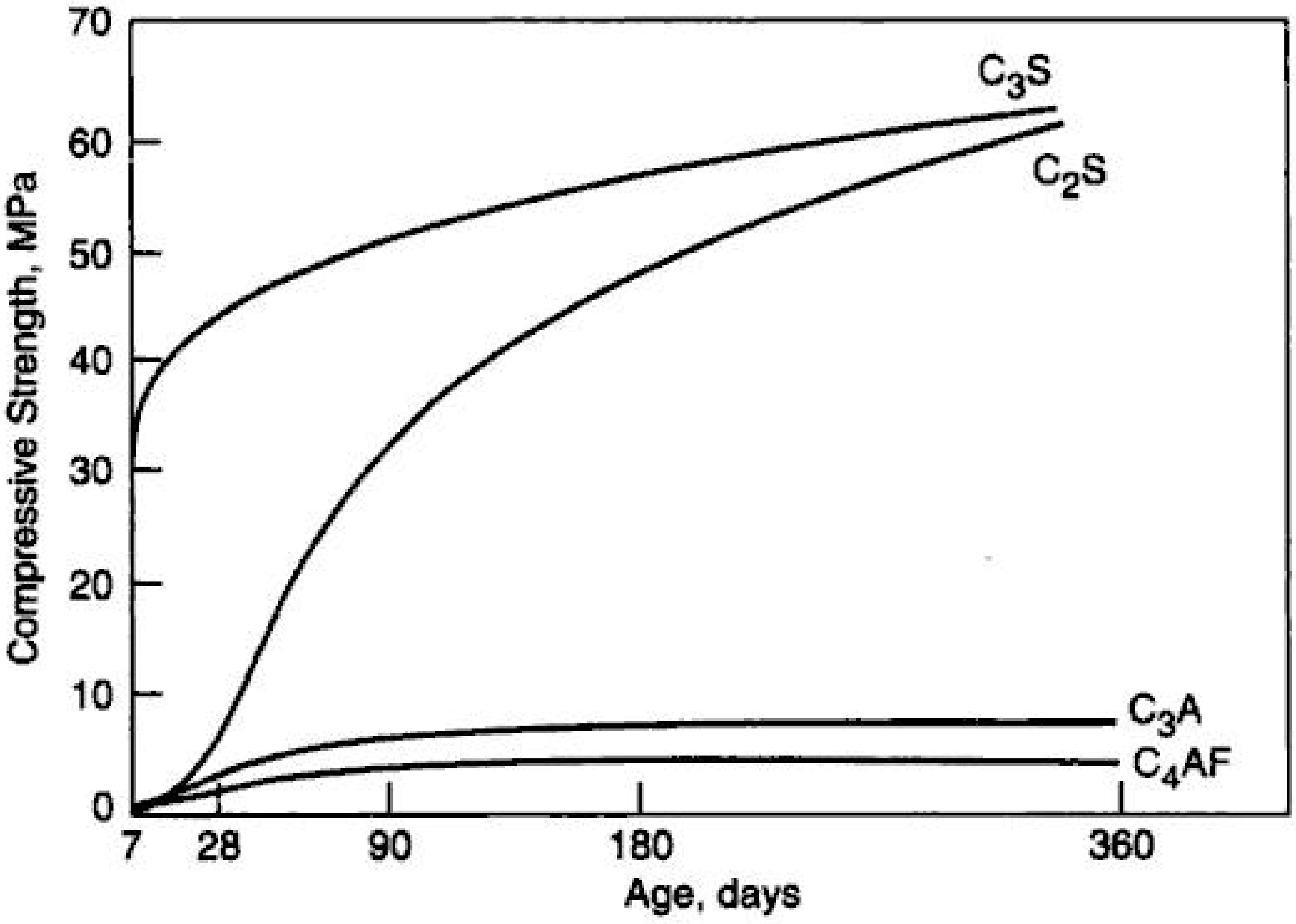

Fig. 2.2 Contribution of cement compounds to the strength of cement

Thus a higher percentage of C_3S results in rapid hardening with an early gain in strength at a higher heat of hydration. On the other hand, a higher percentage of C_2S results in slow hardening, less heat of hydration and greater resistance to chemical attack.

The compound tricalciumaluminate (C_3A) is characteristically fast-reacting with water and may lead to an immediate stiffening of paste, and this process is termed *flash set.* The role of gypsum added in the manufacture of cement is to prevent such a fast reaction. C_3A reacts with 40 per cent water by mass, and this 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. It provides weak resistance against sulphate attack and its contribution to the development of strength of cement is perhaps less significant than that of silicates. In addition, the C_3A phase is responsible for the highest heat of evolution both during the initial period as well as in the long run. Like C_3A, C_4AF hydrates rapidly but its individual contribution to the overall strength of cement is insignificant. However, it is more stable than C_3A.

In terms of oxide composition, a high lime content generally increases the setting time and results in higher strengths. A decrease in lime content reduces the strength of concrete. A high silica content prolongs the setting time and gives more strength. The presence of excess unburnt lime is harmful since it results in delayed hydration causing expansion (unsoundness) and *deterioration of concrete.* Iron oxide is not a very active constituent of cement, and generally acts as a catalyst and helps the burning process. Owing to the presence of iron oxide the cement derives the characteristic grey colour. Magnesia, if present in larger quantities, causes unsoundness.

2.2.2 Hydration of Cements

The extent of hydration of cement and the resultant microstructure of hydrated cement influences the physical properties of concrete. The microstructure of hydrated cement is more or less similar to that of silicate phases. When the cement comes in contact with water, the *hydration of cement* proceeds both inward and outward in the sense that the hydration products get deposited on the outer periphery and the nucleus of the unhydrated cement inside gets gradually diminished in volume. The reaction proceeds slowly for 2–5 hours (called *induction* or *dormant period*) before accelerating as the surface skin breaks. At any stage of hydration, the cement paste consists of gel (a finely-grained product of hydration having large surface area collectively called gel), the remnant of unreacted cement, calcium hydroxide $Ca(OH)_2$, and water, besides some other minor compounds. The crystals of various resulting compounds form an interlocking random three-dimensional network gradually filling the space originally occupied by the water, resulting in stiffening and subsequent *development of strength.* Accordingly, the hardened cement paste has a porous structure, the pore size varying from very small (4×10^{-4} µm) to a much larger value, the pores being called *gel pores* and *capillary pores*, respectively. The pore system inside the hardened cement paste may or may not be continuous. As the hydration proceeds, the deposit of hydration products on the original cement grain makes the diffusion of water to unhydrated nucleus more and more difficult thus reducing the rate of hydration with time.

The reactions of compounds of cement and their products may be represented as:

$$2(3CaO.SiO_2) + 6\ H_2O \rightarrow 3CaO.\ 2SiO_2\ .3H_2O + 3\ Ca(OH)_2$$

or symbolically

$$2C_3S + 6H \rightarrow C_3S_2H_3 + 3\ Ca\ (OH)_2$$

$$2\ (2CaO.SiO_2) + 4\ H_2O \rightarrow 3CaO.2SiO_2.3H_2O + Ca\ (OH)_2$$

or

$$2C_2S + 4\ H \rightarrow C_3\ S_2\ H_3 + Ca\ (OH)_2$$

$$2C_3A + 21\ H \rightarrow C_4\ A\ H_{13} + C_2AH_8 \rightarrow 2C_3\ AH_6 + 9H$$

$$C_4AF + 7\ H \rightarrow C_3AH_6 + CFH$$

Reactions in the presence of gypsum:

$$C_3A + 32\ H + 3CaSO_4 \rightarrow C_3A.3C\bar{S}H_{32} \rightarrow C_6AS_3H_{32}$$

(tri-sulphate hydrate or ettringite)

$$C_3A + 18\ H + CaSO_4 \rightarrow C_3A.C\ \bar{S}H_{18} \rightarrow C_4A\ \bar{S}H_{18}$$

(monosulphate hydrate)

Above equations (with $C = CaO$; $S = SiO_2$ and $H = H_2O$) only refer to the process in which the cement compounds react with water to form a strong hydrated mass. The hydrated crystals are extremely small, varying from colloidal dimensions (less than 2 μm) to 10 μm or more. The calcium hydroxide, $Ca(OH_2)$, liberated during the reaction of *silicate phase* crystallizes in the available free space. The product $C_3S_2H_3$ representing *calcium silicate hydrate, a gel structure*, is normally expressed by hyphenated C-S-H, which signifies that it is not a well-defined compound.

The hydration of C_3S produces a comparatively lesser quantity of C–S–H than that produced by C_2S. On the other hand C_3S liberates nearly three times as much calcium hydroxide on hydration as C_2S. However, $Ca(OH)_2$ is not a desirable product in the concrete mass as it is soluble in water and gets leached out making the concrete porous. The only advantage of $Ca(OH)_2$ is its being alkaline in nature and maintaining a pH value of around 13 in the concrete. A pH value at this level passivates reinforcing steel against corrosion. In general, the quality and density of C-S-H produced due to hydration of C_3S is slightly inferior to that formed by hydration of C_2S. The hydration product of C_2S is rather dense and its specific surface is higher.

On hydration of C_3A, a calcium aluminate system $CaO–Al_2O_3–H_2O$ is formed. The cubic compound C_3AH_6 is probably the only stable product. Hydration of C_4AF is believed to form a system $CaO–Fe_2O_3–H_2O$. A hydrated calcium ferrite of the form C_3FH_6 is comparatively more stable. In the presence of gypsum, depending upon the concentrations of aluminate and sulphate ions in the solution phase, the precipitating crystalline product is either calcium aluminate trisulphate hydrate ($C_6S_3H_{32}$) or calcium aluminate monosulphate hydrate. The product calcium aluminate trisulphate is known as *ettringite*, which crystallizes as short prismatic needles on account of high sulphate–aluminate ions ratio in the solution phase during first hour of hydration. When sulphate solution gets depleted, aluminate ions concentration increases due to renewed hydration and the aluminate is gradually converted into monosulphate which is the final product of hydration of Portland cement containing more than 5 per cent C_3A.

Rate of Hydration

As mentioned earlier, the reaction of the compound C_3A with water is very fast in that *flash setting,* i.e. siffening without strength development, can occur because the C—A—H phase prevents the hydration of C_3S and C_2S. However, some of the $CaSO_4$ ground in the clinker dissolves immediately in water and the sulphate ions in the solution react with C_3A to form insoluble calcium sulphoaluminate which deposits on the surface of the C_3A to form a protective colloidal membrane and thus retard the direct hydration reaction. When all the sulphate is consumed, hydration can accelerate. The amount of sulphate must, therefore, be carefully controlled to leave little excess C_3A to hydrate directly. The hardening of C_3S appears to be catalyzed by C_3A so that C_3S becomes almost solely responsible for the gain of strength up to about 28 days by growth and interlocking of C—S—H gel. The later age increase in strength is due to the hydration of C_2S. The *rate of strength development* can, therefore, be modified by changes in the relative quantities of these compounds.

Mechanism of Hydration

C_3A reacts from beneath the thin membrane of calcium sulphoaluminate formed on the C_3A surface. Owing to the larger volume of calcium sulphoaluminate, pressure develops and the membrane eventually bursts, allowing the sulphate in solution to come in contact with unreacted C_3A to reform the membrane. The cyclic process continues until all the sulphate in solution is consumed, whereupon the C_3A can hydrate directly at a faster rate and the transformation of calcium sulphoaluminate into needle like monosulphate crystals leads to the loss of *workability* and to setting. This gives rise to the *induction period* which ends when the protective membrane is disrupted. Although the reaction between C_3S and water proceeds at the same time, in a properly retarded cement the end of induction period of C_3S hydration coincides with the point at which the sulphate in solution is no longer available for reaction. Setting, now, is due to the simultaneous growth of aluminate hydrate, monosulphate and silicate hydrate in the inter-particle space. The above theory is termed as *protective membrane* layer theory.

Effect of Admixtures on Hydration

Some *admixtures* may reduce the electric repulsion between the individual positively charged hydrating cement particles, so that they approach closer and stick to form agglomerates which grow and eventualy settle out. This process is termed *flocculation* and the agglomerates floc. The anions may flocculate the *colloidal membrane* thus making it more permeable. The rapid diffusion of water through the permeable membrane increases hydrostatic pressure beneath the membrane till it reaches a level sufficient to rupture it at an earlier stage in hydration, thus accelerating the hydration of cement.

2.2.3 Water–Cement Ratio and Compressive Strength

A cement of average composition requires about 25 per cent of water by mass for chemical reaction. In addition, an amount of water is needed to fill the gel pores. The total amount of water thus needed for chemical reaction and to fill the gel pores is about 42 per cent. The general belief that a *water–cement ratio* of less than 0.40 or so should not be used in concretes—because for the process of hydration, the gel pores should be saturated—is not valid. This is because, even in the presence of excess water, the complete hydration of cement never takes place due to the decreasing porosity of the hydration products. As a matter of fact, a *water–cement ratio* of less than 0.40 is quite common in high-strength structural concretes.

In concrete, as explained earlier, the hardened cement paste is a porous ensemble. Also, the concentration of the solid products of hydration in the total space or volume available (the original water and hydrated cement) is an index of *porosity*. Like any other porous solid, the compressive strength of cement paste (or concrete) is related to the parameter *gel–space ratio* or *hydrate–space ratio*. The *water–cement ratio* which governs the compressive strength, is really an expression of the concentration of hydration products in the total volume at a particular age for the resultant degree of hydration.

2.2.4 Physical Properties of Portland Cement

The cement to be used in construction must have certain given qualities in order to play its part effectively in a structure. When these properties lie within a certain range, the engineer is confident that in most of the cases the cement performance will be satisfactory. Also, based on these properties, it is possible to compare the quality of cement from different sources. Frequent tests are carried out on the cement either on dry powder or hardened cement paste, and sometimes on the concrete made from the cement, to maintain quality within specified limits. The important physical properties of a cement are as follows.

Fineness

The fineness of a cement is a measure of the size of particles of cement and is expressed in terms of *specific surface of cement*. It can be calculated from particle size distribution or one of the air permeability methods. It is an important factor in determining the *rate of gain of strength* and *uniformity* of quality. For a given weight of cement, the surface area is more for a finer cement than for a coarser cement. The finer the cement, the higher is the rate of hydration, as more surface area is available for chemical reaction. This results in the early development of strength. The effect of fineness on the compressive strength of cement is shown in Fig. 2.3. If the cement is ground beyond a certain limit its cementative properties may be adversely affected due to prehydration by atmospheric moisture. As per Indian Standard Specifications the residue of cement should not exceed 10 per cent when sieved on a 90-micron IS sieve. In addition, the amount of water required for constant slump concrete decreases with the increase in the fineness of cement.

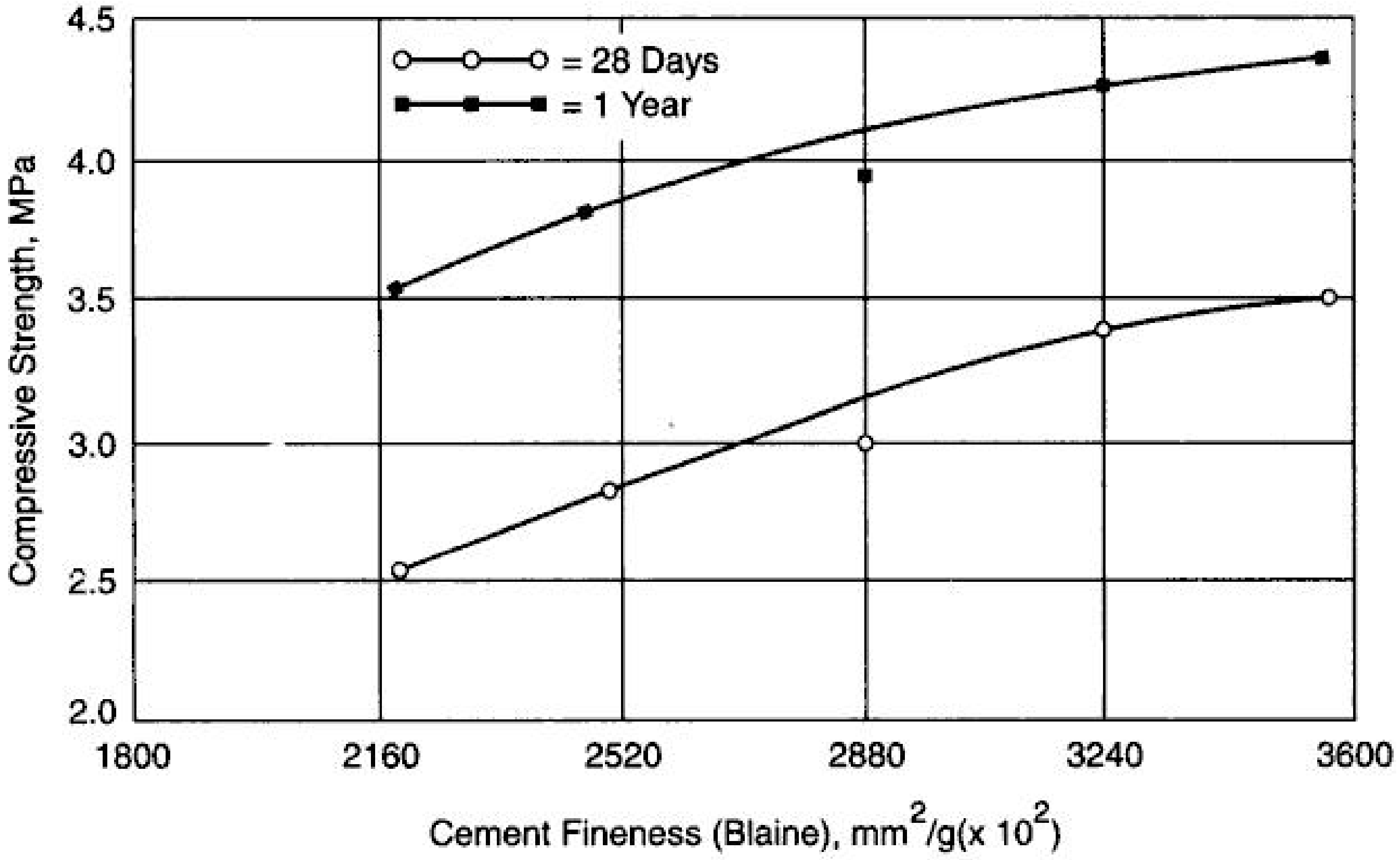

Fig. 2.3 The effect of fineness of cement on the compressive strength of concrete

Setting Time

Cement when mixed with water forms paste which gradually becomes less plastic, and finally a hard mass is obtained. In this process of setting, a stage is reached when the cement paste is sufficiently rigid to withstand a definite amount of pressure. The time to reach this stage is termed setting time. The time is reckoned from the instant when water is added to the cement. The setting time is divided into two parts, namely, the *initial* and the *final setting times.* The time at which the cement paste *loses* its plasticity is termed the *initial setting time.* The time taken to reach the stage when the paste becomes a hard mass is known as the *final setting time.*

It is essential for proper concreting that the initial setting time be sufficiently long for finishing operations, i.e. transporting and placing the concrete. The setting process is accompanied by temperature changes. The temperature rises rapidly from the initial setting to a peak value at the final setting. The setting time decreases with rise in temperature up to 30°C and vice versa. The setting times specified for various types of cements are given in Table 2.4. For an ordinary portland cement, the initial setting time should not be less than 30 minutes and final setting time should not be more than 600 minutes. A phenomenon of abnormal premature hardening within a few minutes of mixing the water is termed *false set.* However, not much heat is evolved and remixing the paste without water restores the plasticity and then the cement sets in the normal manner with no appreciable loss of strength.

In practice, the length of time for which a concrete mixture will remain plastic is usually more dependent on the amount of mixing water used and atmospheric temperature than on the setting time of cement.

Soundness

The unsoundness of cement is caused by the undesirable expansion of some of its constituents, sometimes after setting. The large change in volume accompanying expansion results in disintegration and severe cracking. The unsoundness is due to the presence of free lime and magnesia in the cement. The free lime hydrates very slowly because it is covered by the thin film of cement which prevents direct contact between lime and water. After the setting of cement, the moisture penetrates into the free lime resulting in its hydration. Since slaked lime occupies a larger volume, the expansion takes place resulting in severe cracking. The unsoundness due to the presence of magnesia is similar to that of lime. The unsoundness may be reduced by:

(i) limiting the MgO content to less than 0.5 per cent,
(ii) fine grinding,
(iii) allowing the cement to aerate for several days, and
(iv) thorough mixing.

The chief tests for soundness are the Le Chatelier and Autocalve tests. The expansion carried out in the manner described in IS: 269–1989 should not be more than 10 mm in the Le Chatelier test and 0.8 per cent in Autoclave test.

Compressive Strength

It is one of the important properties of cement. The strength tests, generally carried out in tension on samples of neat cement, are of doubtful value as an indication of ability of the cement to make concrete strong in compression. Therefore, these are largely being superseded *by the mortar cube* crushing tests and *concrete compression tests.* These are conducted on standardized aggregates under carefully controlled conditions and therefore give a good indication on strength qualities of cement. Cement mortar cubes (1:3) having an area of 5000 mm^2 are prepared and tested in compression testing machine. For ordinary portland cement the compression strength at 3 and 7 days curing shall not be less than 16 MPa and 22 MPa, respectively. The graded standard sand used for preparing the cubes should conform to IS: 650–1991.

Standard Sand A particular variety of sand available at Ennore in Tamil Nadu is used as standard sand which closely resembles the Leighton Buzzard sand (the British Standard Sand) in its properties The imported Leighton sand has been replaced by Ennore sand. The standard sand has following properties.

(i) The standard sand shall be of quartz, of light grey or whitish variety and shall be free from silt.
(ii) The sand grains shall be angular with shape approximating to spherical forms.
(iii) The sand shall pass through IS: 850-μm sieve and not more than 10 per cent shall pass through IS: 600-μm sieve.
(iv) It shall be free from organic impurities.

Heat of Hydration

The *silicates* and *aluminates* of *cement react* with *water to form a binding medium, which solidifies into a hardened* mass. This reaction is termed hydration, which is exothermic with approximately 120 cal/g being liberated. In the interior of mass concrete constructions like dams, etc. the temperature can be as high as 50°C above the initial temperature of concrete mass at the time of placing the concrete. This high temperature is found to persist for a prolonged period. At the same time, the exterior of the concrete mass loses some heat so that a *steep temperature gradient* may be established, and during the subsequent cooling of the interior, severe *cracking* may occur. On the other hand, the heat of hydration may be advantageous in preventing the freezing of water in the capillaries of freshly placed concrete in cold weather.

The heat of hydration is defined as the quantity of heat, in calories per gram of hydrated cement, liberated on complete hydration at a given temperature. The different cement compounds hydrate at different rates and liberate different quantities of heats. On adding water to cement, a rapid heat of evolution lasting for few minutes is due to reaction of aluminates. However, this initial heat evolution ceases quickly as solubility of aluminates is restrained by C_3S. The total heat generated in the complete hydration process will depend upon the relative quantities of major compounds of cement. A normal cement generally produces approximately 90 cal/g of heat in 7 days and 90 to 100 cal/g in 28 days. It is determined by measuring the quantities of heat liberated by unhydrated and hydrated cements in a mixture of nitric and hydrofluoric acids, the difference between the two values represents the heat of hydration. The heat of hydration for low-heat Portland cement should not be more than 66 and 75 cal/g for 7 and 28 days, respectively.

The heat of hydration increases with temperature at which hydration takes place. For ordinary portland cement (OPC) it varies from 37 cal/g at 5 °C to 80 cal/g at 40 °C. For common types of Portland cements, about 50 per cent of the total heat is liberated between 1 and 3 days, about 75 per cent in 7 days and 83 to 91 per cent in six months. By restricting the quantities of compounds C_3A and C_3S in cement, the high rate of heat liberation in early ages can be controlled. The rate of hydration and the heat liberation increases with the fineness of cement but the total amount of heat liberated is unaffected by the fineness.

Specific Gravity

The specific gravity of portland cement is generally about 3.15, but that of cement manufactured from materials other than limestone and clay, the value may vary. Specific gravity is not an indication of the quality of cement. It is used in calculation of mix proportions.

2.2.5 Chemical Properties of Cements

The *loss on ignition* test is carried on portland cement to determine the loss of weight when the sample is heated to 900–1000 °C. The loss in weight occurs as the moisture and carbon dioxide which are present in combination with free lime or magnesia evaporate. The presence of mositure causes prehydration of cement and may be absorbed from atmosphere during manufacturing or afterwards. The carbon dioxide is also taken from the atmosphere. The loss in weight is a measure of the freshness of cement. Since the hydroxides and carbonates of lime and magnesium have no cementing property, they are termed inert substances. Lesser the loss on ignition, lesser is the quantity of these inert substances and better is the cement.

The loss on ignition is determined by heating one gram of cement sample in a platinum crucible at a temperature of 900 °C–1000 °C for minimum of 15 minutes.

Normally the loss will be in the neighbourhood of 2 per cent. Maximum allowable loss is 4 per cent.

Insoluble Residue

The insoluble material is an inactive part of cement. It is determined by stirring one gram of cement in 40 ml of water and adding 10 ml of concentrated HCl. The mix is boiled for 10 minutes maintaining constant volume. Any lump, if present, is broken and the solution filtered. The residue on filter is washed with Na_2CO_3 solution, water and HCI in the given order and, finally, again with water. The filter paper is dried, ignited, and weighed to give an insoluble residue. The minimum the residue, the better is the cement. The maximum allowable value is 0.85 per cent.

2.3 TYPES OF CEMENTS

By using additives, changing the chemical composition of the Portland cement—by varying the percentage of the four basic compounds through the use of different raw materials—it is possible to obtain several types of cements, each with some unique characteristics for the required performance. A gradual increase in the C_3S content and fineness has enabled general-purpose Portland cements to develop very high strength at early ages. The oxide and compound compositions of some of the commonly used Portland cements are given in Table 2.3. The compound composition of cements is determined by quantitative microscopy.

Table 2.3 *Oxide and Compound Compositions of Portland Cements*

Type of cement	*Oxide composition, (per cent)*				*Compound composition (per cent)*			
	CaO	SiO_2	Al_2O_3	Fe_2O_3	C_3S	C_2S	C_3A	C_4AF
Normal or ordinary cement	63	20.6	6.3	3.6	40	30	11	11
Rapid-hardening cement	64.5	20.7	5.2	2.9	50	21	9	9
Low-heat cement	60	22.5	5.2	4.6	25	45	5	14
Sulphate-resisting cement	64	24.5	3.7	3.0	40	40	5	9

The following are the main types of Portland cement.

2.3.1 General-purpose Portland Cements

The commonly used Portland cement in India is branded as 33-grade (IS: 269–1989), 43-grade (IS: 8112–1989) and 53-grade (IS: 12269–1987) having 28-days mean *compressive strengths* exceeding 33 MPa, 43 MPa and 53 MPa, respectively. All the three *grades of ordinary Portland* cement are produced from the same materials as explained earlier. The higher strengths are achieved by increasing the tricalcium silicate (C_3S) content and also by finer grinding of the *clinker*. The fineness of 53-grade cement obtained by *Blaine's air permeability* test is specified to be of the order of 350 000 mm^2/g. The requirements of the initial and final *setting times* are same as that of conventional OPC. The conventional OPC, i.e. 33-grade cement has virtually disappeared and has been displaced by high strength 43-grade cement. The minimum compressive strengths of the 43-grade cement are 23 MPa and 33 MPa at the end of three days and seven days, respectively. The use of this cement was originally restricted to the production of railway sleepers and generally referred to as *sleeper cement*. The railway specifications require that the initial setting time should not be less than 90 minutes. At higher *water–cement ratios*, the concrete produced with high-strength cement has about 80 per cent higher strength and at lower *water–cement ratios*, it has 40 per cent higher strength than that of concrete using 33-grade OPC. The cost of high-strength Portland cements is only marginally higher than the OPC. The use of this cement in the usual 1:2:4 nominal mix, with a water–cement ratio of 0.60 to 0.65 can easily yield M25 concrete. Its composition and properties are governed by IS: 8112–1989. Greater fineness of 43 and 53 grade cements increase workability due to reduction of friction between aggregates. Moreover, due to shorter setting time and *faster development of strength*, the *stripping time* is shorter. Although high strength cements of grades 43 and 53 are desirable for economical design of *high-grade concretes*, but they can also be used for lower grade concretes. However, to make *high strength concrete* a *high performance concrete*, will require extremely careful batching, mixing, transportation, placing, compaction and curing.

IS: 10262–1982 has classified the OPC grade-wise from A to F depending upon the 28 days compressive strength as: A (32.5–37.5 MPa), B (37.5–42.5 MPa), C (42.5–47.5 MPa), D(47.5–52.5 MPa), E(52.5–57.5 MPa), F(57.5–62.5 MPa). Accordingly, the 33, 43 and 53 grades of cement correspond to categories A, C and E, respectively. However, most of the 43-grade cements available in the market generally fall in the category D, and the 53-grade cements available are generally in the category F or above. The actual strength of cement must be ascertained either from the manufacturer or through laboratory tests before it is used in *concrete mix design* to get the maximum benefit of the additional strength and superior quality.

2.3.2 Special-purpose Cements

The special-purpose cements are manufactured for the specific *performance requirements*. The frequently used ones are the following.

(i) OPC-based cements
(ii) Non-OPC cements

These cements have some further classifications, which are described below.

OPC-based Cements

Rapid-hardening Portland Cement This cement is similar to OPC but with higher C_3S content and finer grinding. A higher fineness of cement particles provides greater surface area (not less than 325 000 mm^2/g) for action with water. It gains strength more quickly than OPC, though the final strength is only slightly higher. The one-day strength of this cement is equal to the three-day strength of 33-grade OPC with the same *water–cement ratio*. This cement is used where a *rapid strength development* is required. The rapid gain of strength is accompanied by a higher rate of heat development during the *hydration of cement*. This may have advantages in cold weather concreting, but a higher concrete temperature may lead to cracking due to subsequent *thermal contraction*, and hence should not be used in mass concreting or thick structural sections. The composition, fineness and other properties are governed by IS: 8041–1990. It is only about 10 per cent costlier than OPC. It is recommended for prefabricated concrete construction, road repairs and in applications requiring early stripping of forms.

Low-heat Portland Cement This cement is less reactive than OPC and is obtained by increasing the proportion of C_2S and reducing C_3S and C_3A. This reduction in the content of more rapidly hydrating compounds C_3S and C_3A

results in a slow development of strength but the ultimate strength is the same. In any case, to ensure a sufficient rate of development of strength, the *specific surface* of cement must not be less than 320 000 mm^2/g. The *initial setting time* is greater than OPC. The properties and composition are governed by IS: 12600–1989. This cement is recommended for the use in mass concrete construction such as dams where temperature rise by heat of hydration can become excessive.

Sulphate-resisting Cement A Portland cement with low C_3A (less than 5 per cent) and C_4AF contents is very effective against *sulphate attack*. Such a cement having high silicate content is called sulphate-resisting cement. The content of tetra-aluminoferrite C_4AF in OPC varies between 6 to 12 per cent. As it is not feasible to reduce the Al_2O_3 content of raw material, Fe_2O_3 is added to the raw materials mixture to increase C_4AF content at the expense of C_3A. IS: 456–2000 limits the total content of C_4AF and C_3A such that $2C_3A + C_4AF$ shall not exceed 25 per cent. Such a cement with low C_3A content is effective against sulphate attack whereas the ordinary Portland cement is susceptible to attack of sulphates in solution which permeate in the hardened concrete and react with free $Ca(OH)_2$, hydrate of calcium aluminate and even hydrated silicates to form calcium sulphoaluminate having a volume of approximately 227 per cent of the volume of original aluminates. This expansion within the hardened structure of cement paste results in cracks and subsequent disruption. This phenomenon is called sulphate attack, which is greatly accelerated if accompanied by an *alternate wetting* and *drying* as in the case of *marine environment.*

The use of sulphate resisting cement is recommended for concretes to be used in the marine environment, foundations in chemically aggressive soils, for pipes to be buried in marshy regions or sulphate bearing soils, and construction of sewage treatment plants.

Masonry Cement This cement is manufactured by intimately grinding a mixture of OPC clinker and gypsum with mineral additives (pozzolanas) or inert (non-pozzolanic) materials such as limestone, dolomite, carbonated sludge, etc. and air-entraining agents in suitable proportions, generally to a fineness greater than that of OPC. Masonry cement conforming to the standard requirements can be produced by intergrinding 3 parts of Portland cement clinker, 5 parts of fly ash and 5 parts of granulated blast furnace slag or 1 part of Portland cement clinker, 2 parts of fly ash and 2 parts of hydrated lime with suitable quantity of gypsum and an air-entraining admixture. Masonry cement mortar is considered superior to lime mortar, lime-cement mortar and cement mortar. It combines the desirable properties of cement mortar relating to strength and setting, and lime mortar relating to workability and water-retention. Thus a masonry cement produces, a smooth, plastic, cohesive and strong, yet workable mortar. The cracks due to shrinkage and temperature movement are considerably reduced. Its composition and properties are governed by IS: 3466–1988. The physical requirements of the masonry cement are:

(i) *Fineness*	
Residue on 45-micron sieve, max	15 per cent
(ii) *Setting times*	
(a) Initial setting time, min	90 minutes
(b) Final setting time, max	24 hours
(iii) *Soundness*	
(a) Le-Chatelier expansion, max	10 mm
(b) Autoclave expansion, max	01 per cent

The following properties are measured on the mortar composed of 1 part of masonry cement and 3 parts of standard sand by volume.

(iv) *Compressive strength*	
(a) at 7 days, min	2.5 MPa
(b) at 28 days, min	5.0 MPa
(v) *Air content*, min	6 per cent
(vi) *Water-retention*	
Flow after suction as compared to the original flow, min	60 per cent

Waterproof Portland Cement Waterproof cement is manufactured by adding a waterproofing substance to ordinary Portland cement during mixing. The common admixtures are calcium stearate, aluminium stearate and the gypsum treated with tannic acid.

White Portland Cement The process of manufacturing white cement is the same as of ordinary Portland cement but the amount of *iron oxide, which is responsible for greyish colour*, is limited to less than 1 per cent. This is achieved by careful selection of raw materials and often by the use of refined furnace oil (RFO) or gas fuel in place of pulverized coal in the kiln. The suitable raw materials are chalk and high purity limestones having 95 per cent $CaCO_3$ and less than 0.1 per cent iron oxide contents, and white clays. Its composition and properties are governed by IS: 8042–1989. Generally white cement is ground finer than the grey cement.

Coloured Portland Cement These are basically Portland cements to which pigments are added in quantities up to 10 per cent during the process of grinding the cement clinker. A good pigment should be permanent, i.e. colour should be durable under exposure to light and weather, and chemically *inert* when mixed with cement. For lighter colours *white cement* has to be used as basis.

Hydrophobic Cement This type of cement is obtained by adding water repellant film forming substances like stearic acid, boric acid, oleic acid and pentachlorophenol to OPC during grinding of cement clinker. These acids form a *film around the cement particles*, which prevent the entry of atmospheric moisture, and the film breaks down when the concrete is mixed, and then the normal hydration takes place. The film forming materials also entrain certain amount of air in the body of concrete which improves its workability. Its composition and properties are governed by IS: 8043–1991. This cement is useful for the places having high humidity, poor transportation system and perforce storage for long time. In such situations OPC gets deteriorated and loses significant part of its strength.

The physical and chemical requirements for some of the commonly used cements are summarized in the Tables 2.4 and 2.5.

Air-entraining Cement This cement is manufactured by mixing small quantity of *air-entraining* agent like alkali salts of wood resins; synthetic detergents of alkyl-aryl sulphate type and calcium lignosulphate with ordinary Portland cement. These agents in powder or in liquid forms are added to the extent of 0.025 to 0.100 per cent by weight of OPC cement clinker at the time of grinding. At the time of mixing, these cements produce tiny, discrete non-coalesceing air bubbles in the concrete mass which enhances *workability* and reduces tendency to *segregation* and *bleeding*.

Expansive Cement Cement which does not shrink while hardening and thereafter, but expands slightly with time is called expansive cement. This cement does not suffer any overall change in volume on drying. Expansive cement is obtained by mixing about 8 to 20 parts of the sulphoaluminate clinker with 100 parts of the OPC and 15 parts of the stabilizer. In one type of expansive cement called *shrinkage compensating cement*, the restraint to the expansion induces compressive stress which approximately offsets the tensile stress induced by shrinkage. In another type called *self-stressing cement*, the concrete induces significant compressive stresses after the occurrence of *drying shrinkage*. In addition to neutralizing the shrinkage they provide prestressing effects in the tensile zone of a flexural member. This cement is commonly used for grouting anchor bolts or grouting machine foundations or prestressed concrete ducts wherein drying shrinkage may otherwise defeat the purpose of grout.

Oil-well Cement The annular space between steel casting and sedimentary rock formation through which oil well has been drilled, is sealed off by *cement slurry* to prevent escape of oil or gas. The cement slurry also seals off any other fissure or cavities in the rock layer. For this purpose cement slurry has to be pumped down to points located in the annulus around the casting, at considerable depth where prevailing temperature may be as high as 350 °C under pressure up to 150 MPa. The slurry used for this purpose must remain mobile to be able to flow under these conditions for periods up to several hours and then harden fairly rapidly to give sufficient strength to support the casting. It may also have to resist corrosive conditions from sulphur gases and water containing dissolved salts. The type of cement suitable for above conditions is called oil-well cement. The cement produced by inter-grinding Portland cement clinker, fly ash, gypsum and certain admixtures (retarders) in suitable proportions has been found to conform to the requirements of an oil-well cement. These retarders prevent quick setting and retain slurry in mobile condition to facilitate penetration to all fissures and cavities. The composition and properties are governed by IS: 8229–1986.

Very High Strength Cements The cements of this category can be obtained by improving particle packing density and microstructure of cement pastes as follows.

Removing entrapped air In the conventionally mixed cement paste relatively large voids or defects are usually present due to entrapped air which limit the strength. In one of the systems, water soluble polymer is added as a rheological aid to permit cement to be mixed with a very small amount of water and at final processing stage entrapped air is removed by application of modest pressure of 5 MPa. This process has resulted in a strength of 300 MPa for calcium aluminate system and 150 MPa for OPC. This system is called *macro-defect free cement.*

Providing densely packed system OPC and ultra fine silica fume (5 to 20 per cent) are mixed to obtain a densified system containing homogeneously arranged particles. A compressive strength of 270 MPa has been obtained with silica fume substituted paste.

Achieving densification with warm pressing By the method of warm pressing, i.e. applying heat and pressure simultaneously to cement paste results in reduction of porosity and generation of very homogeneous fine microstructure with small porosity. By warm pressing of mixture of Portland and calcium cements has resulted in compressive strength of 650 MPa.

Non-OPC Cements

High-alumina Cement This cement is basically different from OPC and concrete made with it has properties different from OPC concrete. *High-alumina cement* (HAC) is very reactive and produces very *high early strength.* About 80 per cent of the ultimate strength is developed at the age of 24 hours and even at 6 to 8 hours. High-alumina cement has an *initial setting time* of about 4 hours and the final setting time of about 5 hours. Generally no *additives* are added to alumina cement. For the same *water–cement ratio*, the alumina cement is more workable than Portland cement. The strength is adversely affected by rise in temperature. HAC is extremely resistant to chemical attack and is suitable for under sea water applications.

The raw materials used for its manufacture are limestone or chalk and *bauxite* which are crushed into lumps not exceeding 100 mm. These raw materials with appropriate proportion of coke are charged into the furnace which is fired with pulverized coal or oil. The fusion takes place at temperature about 1600 °C. The solidified material is fragmented and then ground to a fineness of 250 000–320 000 mm^2/g. The very dark grey powder is passed through magnetic separators to remove metallic iron. The alumina cement is considerably more expensive. The pozzolana additives are not useful in concrete made with HAC because it does not produce calcium hydroxide that would react with pozzolanas. Its composition and properties are governed by IS: 6452–1989. The approximate chemical oxide composition is as follows:

Alumina ($A1_2O_3$)	39 per cent
Ferric Oxide (Fe_2O_3)	10 per cent
Lime (CaO)	38 per cent
Ferrous Oxide (FeO)	4 per cent
Silica (SiO_2)	6 per cent

During hydration of HAC initially monocalcium aluminate decahydrate (CAH_{10}), dicalcium aluminate octahydrate ($C_2A\ H_8$) and alumina gel (AH_3) are formed. However, these compounds of hydration are metastable and at normal temperature convert gradually to a more stable tricalcium aluminate hexahydrate (C_3AH_6). This conversion is accompanied by a loss in strength and change in crystal form from hexagonal to cubical shape resulting in a release of water with consequent reduction in the volume of solids and an increase in the *porosity*. The increase in porosity enhances its vulnerability to *chemical attack*. The rate of conversion increases with the rise in temperature. The hydration and conversion processes can be symbolically represented as:

$$CA + 10\,H \rightarrow CA\,H_{10}$$

$$3CA\,H_{10} \rightarrow C_3AH_6 + 2AH_3 + 18\,H$$

High alumina cement concrete loses considerable strength when subjected to humid conditions and high temperature. Desiccated high alumina cement concrete when subjected to high temperature, undergoes insignificant conversion and has significant residual strength. A completely desiccated alumina cement has very high resistance to dry heat. A concrete made using this cement and crushed firebricks as aggregate can withstand temperatures up to 1350 °C. A *refractory concrete* for withstanding temperatures up to 1600 °C may be produced by using aggregates such as dead-burnt magnesite, carborundum, silimanite, etc. Since high alumina cement is slow setting but rapid hardening certain proportions of OPC may be added to reduce *setting time*.

Lithium salts have been effectively used as accelerator in high alumina cement to obtain high early strength cement. This has resulted in strength as high as 4 MPa in one hour, 25 MPa in three hours time and 50 MPa in 24 hours time.

Magnesium Phosphate Cement A very high early strength mortar and concrete developed by CRRI, consists of a pre-packed mixture of dead-burnt magnesite and fine aggregate mixed with phosphate. It sets rapidly and yields durable high strength cement mortar. The dead-burnt magnesite is obtained by calcining $MgCO_3$ at or above 1500 °C and grinding the product to fineness of 300000–350000 mm^2/g (Blains). The ground dead-burnt magnesite is mixed with commercially available crystalline mono-ammonium phosphate after grinding it into a fine powder passing 600 μm sieve, and other ingredients like sodium tripolyphosphate in the form of fine powder, di-sodium tetra borate (borax), fine aggregate (crushed dolomite sand) and water, mixed for one minute. After application in repair of road and subsequent air curing the traffic can be opened in a short period of about 4–5 hours.

2.3.3 Composite or Multiple Blended Cements

With the development of Portland cements having very high strength at early ages, there is general trend now to produce correspondingly *high early strength concrete* mixtures containing large proportions of these cements. These modern concretes tend to crack more easily due to lower *creep*; higher *thermal* and *drying shrinkages* and higher *elastic modulus*. On the other hand some mineral industrial by-products when added to the normal concretes are highly effective in reducing the *heat of hydration*, *strength*, and *elastic modulus* of concrete. These concretes when properly cured are generally less *crack-prone* and therefore less permeable in service.

Thus a *composite* or *blended cement* can be optimized with a synergistic effect, allowing component ingredients to compensate for any mutual shortcomings. Therefore, resource-efficient cements with tailor-made properties can be developed to achieve the needed balance between the industry's quest for *high-performance concrete* and increasingly restrictive *environmental regulations*. Developing cements involving high-volume replacement of OPC with industrial by-products is perhaps the most promising venture for cement industry to meet its environmental obligations.

This synergy between modern Portland cements and *mineral additives* is being systematically exploited by the cement/concrete industry to meet sustainable development and environmental goals by progressively choosing *supplementary cementitious materials* often industrial by-products called *pozzolanas* such as *fly ash* (FA), a waste by-product from coal burning thermal power plants; *ground granulated blast furnace slag* (GBFS), a by-product of iron and steel manufacturing; *silica fume* (SF), a waste by-product of the manufacture of silicon or ferro-silicon alloys from high purity quartz and coal in a submerged-arc electric furnace; and *rice husk ash*, a waste by-product from co-generation electric power plants burning rice husk *as partial replacements for cement*. A pozzolana is a finely ground siliceous glassy material which as such does not possess cementitious property in itself, but reacts in the presence of water with lime (calcium hydroxide) at normal temperature to form compounds of low solubility having cementitious properties. This action is called *pozzolanic action*. The pozzolanic activity is due to the presence of finely divided glassy silica and lime which produce calcium silicate hydrate as is produced in Portland cement hydration. The growth and interlocking of this hydrate gives mechanical strength. The lime produced in Portland cement hydration provides the right environment for *pozzolanic action* to proceed. Since similar hydrates are

Table 2.4 Physical Requirements for Different Types of Ordinary Portland Cements

Characteristics	*Ordinary Portland cement with grade of*			*Portland–Pozzolana (silicate) cement (IS: 1489 –Part 1–1991)*	*Rapid-hardening Portland cement (IS: 8041 –Part 1–1990)*	*Low-heat Portland cement (IS: 12600 –1989)*	*Portland-slag cement* (IS: 455 –1989)
	33 (IS: 269 –1989)	*43 (IS: 8112 –1989)*	*53 (IS: 12269 –1987)*				
1	2	3	4	5	6	7	8
Fineness,							
Specific surface (Blain) (mm^2/g) min	225000	225000	225000	300000	325000	320000	225000
Setting Time, (minutes)							
Initial setting time, min	30	30	30	30	5	60	30
Final setting time, max	600	600	600	600	30	600	600
Soundness,							
Expansion, max							
Le-Chatelier method, (mm)	10.0(5.0)$^+$	10.0(5.0)$^+$	10.0(5.0)$^+$	10.0(5.0)$^+$	10.0(5.0)$^+$	10.0(5.0)$^+$	10.0(5.0)$^+$
Autoclave Method, (%)	0.80(0.60)$^+$	0.80(0.60)$^+$	0.80(0.60)$^+$	0.80(0.60)$^+$	0.80(0.60)$^+$	0.80(0.60)$^+$	0.80(0.60)$^+$
Heat of hydration (cal/g), max							
7 days	–	–	–	–	–	65	–
28 days	–	–	–	–	–	75	–
Compressive strength, (MPa), min							
24+0.5h, (1 day)	–	–	–	–	16.0	–	–
72+1h, (3 days)	16.0	23.0	27.0	16.0	27.5	10	16.0
168+2h, (7 days)	22.0	33.0	37.0	22.0	–	16	22.0
672+4h, (28 days)	33.0	43.0	53.0	33.0	–	35	33.0
Drying shrinkage, (%) max	–	–	–	0.15	–	–	–

Notes: + For aerated sample

Table 2.5 Chemical Requirements for Different Types of Ordinary Portland Cements

Characteristics	*Ordinary Portland cement with grade of*			*Portland–Pozzolana (silicate) cement (IS: 1489 –Part 1–1991)*	*Rapid-hardening Portland cement (IS: 8041 –Part 1–1990)*	*Low-heat Portland cement (IS: 12600 –1989)*	*Portland-slag cement (IS: 455 –1989)*
	33 (IS: 269–1989)	*43 (IS: 8112–1989)*	*53 (IS: 12269–1987)*				
1	2	3	4	5	6	7	8
Lime saturation factor, (LSF)	0.66 to 1.02	0.66 to 1.02	0.8 to 1.02	–	0.66 to 1.02	–	–
Ratio of percentage of alumina to that of iron oxide, (Al_2O_3/Fe_2O_3), min	0.66	0.66	0.66	–	0.66	0.66	–
Insoluble residue, (% by mass), max	4.00	2.00	2.00	α**	2.00	2.00	2.50
Magnesia, (% by mass), max	6.00	6.00	6.00	6.00	6.00	6.00	6.00
Sulphuric anhydride, SO_3 (% by mass), max	2.50 (3.00)*	2.50 (3.00)*	2.50 (3.00)*	2.50 (3.00)*	2.50 (3.00)*	2.50 (3.00)*	2.50 (3.00)*
Slag content, (%)	–	–	–	–	–	–	25 – 65
Permitted additives, (%) (other than gypsum), max	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Content of Pozzolana, (%)	–	–	–	10.00 – 25.00	–	–	–
Total chlorides, (% by mass) max	0.05	0.05	0.05	0.05			
Total alkalies #, (Na_2O) (%), max	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Total loss on ignition, max	5.00	5.00	4.00	5.00	5.00	5.00	4.00

* **Notes:** when the content of tricalcium aluminate (C_3A) is more than 5 per cent where C_3A=2.65 (Al_2O_3)-1.69 (Fe_2O_3)

LSF-Lime saturation factor = $\frac{CaO - 0.7\,SO_3}{2.8\,SiO_2 + 1.2\,Al_2O_3 + 0.65\,F_2O_3}$

** Insoluble residue in Portland–pozzolana cement $\alpha = p + \frac{4.0(100 - p)}{100}$ wherein p is declared percentage of Pozzolana in cement

\# In case of reactive aggregates

produced, the combination of two reactions in mixed cement-cementitious material concrete results in improved mechanical strength. These cementitious materials which participate in the hydration reaction significantly improve the *strength*, *permeability* and *durability* of concrete. The oxide composition of the typical commonly used cementitious materials is compared in Table 2.6. The concrete industry can fulfill its environmental obligations by advantageously using these materials, which would otherwise have to be disposed off in landfill sites, and consequently creating problems with ground water, air and land.

Table 2.6 *Oxide Compositions of Cementitious Materials*

Material	*Oxide composition, per cent by weight*							
	CaO	*SiO_2*	*Al_2O_3*	*Fe_2O_3*	*MgO*	*SO_3*	*Na_2O*	*K_2O*
Ordinary Portland cement (OPC)	63.00	20.00	6.30	3.60	2.40	1.50	0.15	0.50
Ground-granulated blast-furnace slag (GGBS)	42.40	32.30	13.30	0.30	6.40	2.10	–	–
Fly ash (FA)	2.50	52.50	28.20	10.50	1.60	0.20	0.04	0.90
Silica fume (SF)	4.15	93.00	0.20	0.05	0.51	0.05	0.20	0.22

Hydration of Composite Cements

As explained earlier about 40 per cent of the Portland cement is composed of the primary mineral tricalcium silicate, which on hydration forms *calcium silicate hydrate* (C-S-H) and *calcium hydroxide*, $Ca(OH)_2$. In Portland–pozzolana blended cement the pozzolana can be represented by silica (SiO_2) because non-crystalline silica glass is the principal reactive constituent of pozzolana. This silica combines with the calcium hydroxide released on the hydration of Portland cement. Calcium hydroxide in hydrated Portland cement as such does not contribute to development of strength, but in composite cements it is utilized with *reactive silica*.

Slowly and gradually it forms additional *calcium silicate hydrate* which is a binder and fills up the space, and gives *impermeability* and ever-increasing strength. The hydration process of a composite cement can be expressed as:

Portland cement only	$C_3S + H_2O \rightarrow \text{C-S-H} + Ca(OH)_2$
Portland cement +pozzolana	$\underset{\text{(Silica)}}{SiO_2} + Ca(OH)_2 \rightarrow \text{C-S-H}$

The national codes have placed a very heavy emphasis on the chemistry of cementitious material or pozzolana, i.e. on the total amount of oxides of silica, alumina, and iron. There is really no direct relation between the chemistry of pozzolana and the properties of cement. Most of the properties of pozzolana in concrete are determined by the pozzolana *mineralogy* and *particle size* *distribution*, and not by chemistry. Except for calcium, pozzolana chemistry has little influence on reactivity.

Particle Size Distribution of Cementitious Materials

There are two parameters that determine the reactivity of pozzolana, one is the *mineralogy*, and the second is the size distribution of particles which are mostly glassy, solid and spherical. In case of fly ash the particles range in size from 1 to 100 microns (0.1 mm). The average size is about 20 microns, which is similar to Portland cement average particle size. More than 40 per cent of the particles which are under 10 microns, regardless of the type of fly ash, are the ones that contribute to the early age (7 and 28 day) strengths. And particles of the size 45 microns or above which do not participate in *pozzolanic reactions*, even after one year, are considered inert and behave like sand. Particles of size between 10 and 45 microns are the ones that slowly react between the period of 28 days and one year or so. Most of the fly ashes have less than 15 to 20 per cent particles which are above 45 microns.

In addition to the *physical filler effect*, the synergistic action in composite cements continues for the long-term. The calcium hydroxide, $Ca(OH)_2$, produced by early *hydration* of OPC would be consumed by the hydration of the *highly reactive pozzolana* in the blend, such as silica fume, rice husk ash, and metakaolin, to yield more desirable C–S–H phase. The calcium hydroxide produced by the later hydration of OPC would be consumed by the less-reactive component in the blend, such as fly ash, slag, and natural pozzolana, to provide further refinement of *porosity* and improvement of *microstructure*. The overall impact of this sequence of hydration reactions on the decreased *permeability* and increased *durability* of concrete is considerable.

Some of the significant performance parameters of supplementary *cementitious materials* used in composite cements are the following.

(i) No interaction between fly ash and slag occurs when used simultaneously in a composite cement, and each component manifests its own cementitious properties as hydration proceeds.
(ii) Highly reactive pozzolanas enhance the early age strength.
(iii) The effectiveness of pozzolana on *durability* depends on the its characteristics. High calcium pozzolanas provide higher resistance to *sulphate attack* and *chloride-ion penetration* or *diffusion*.
(iv) Replacement of large volumes of OPC with pozzolanic cementitious materials results in significant drop in pH of pore solution and consequent increase in the risk of *depassivation* of steel in reinforced cement concrete.
(v) *Pozzolanic activity* refines *pore structure* which increases electrolytic resistances of concrete.
(vi) The expansion due to *alkali–silica reaction* can be controlled by high level replacement (as high as 60 per cent) of OPC with high-calcium pozzolana.

(vii) High replacement cements have higher accelerated *carbonation* depths compared with OPC.
(viii) In addition to physical filler effect, the replacement of OPC widens the *particle size distribution* of the solid suspension and results in better *rheological (workability) properties*.
(ix) Finer pozzolanas such as *silica fume* or *rice husk* ash can inhibit *bleeding* problems.
(x) Use of pozzolana prevents calcium hydroxide leaching.
(xi) Large volume of blended cements increase powder content of concrete and thus provide a high colloidal volume that combats *segregation* without increasing *heat of hydration*.

2.3.4 Binary Cements

Binary cements are two cementitious constituent systems, in which one constituent is OPC and the other is one of the *cementitious pozzolanas* like *fly-ash* (FA), *ground granulated blast furnace slag* (GBFS), *silica fume* (SF), and *rice husk ash* (RHA). These *environment friendly* cements have been successfully used in the construction of important projects, meeting demanding design criteria in some of the most hostile environments. However, these cements are often associated with shortcomings, such as the need for extended moist-curing, low-early-age strengths, increased use of *admixtures*, increased cracking tendency due to *plastic shrinkage* and as such these cements remain largely underutilized.

Most of the current codal practices limit fly ash usage to 15 to 40 per cent which gives about 7 to 15 per cent reduction in water. Using 50 per cent fly ash may give 20 to 25 per cent water reduction. Reduction in the water content controls the cost and ensures good performance at the same time. With 50 per cent replacement of cement with a fly ash that has fine particles—mostly less than 45 microns—water requirements are reduced by about 30 per cent as compared to the reduction in the range of 25–30 per cent obtained with an expensive *super-plasticizer*. Fly ash improves workability, apparently due to glass beads acting like ball bearings, but the most important reason for fly ash working as a plasticizer for cement, is that the cement particles are electrically charged due to broken bonds and they tend to *flocculate*. Like normal plasticizers, e.g. lignosulfonates fly ash particles get adsorbed on the surface of the cement grains and act as a very powerful *dispersant* to the cement particles.

Addition of mineral additives enhances the intrinsic properties of cement by slow conversion of calcium hydroxide in hydrated cement paste into cementitious product. The major advantages currently recognized are: *improved and dense pore structure* which reduces and micro cracks in the *transition zone* in concrete; *reduced permeability enhances resistance to chemical attack, low diffusivity to chloride ions* and *hence better resistance to corrosion of steel reinforcement and low heat of hydration*.

The general-purpose cements of this category are Portland–pozzolana cement (OPC–FA), Portland slag cement (OPC–GBFS) and super-sulphated cement. The early strength is due to cement clinker fraction and later strength is due to FA and slag fractions. Alkali-silica-reaction (ASR) can be minimized by using by using adequate quantities of these additives.

Portland–pozzolana Cement (OPC–FA)

Portland–pozzolana cement can be produced either by intergrinding the predetermined quantities of Portland cement clinker and pozzolana (15 to 35 per cent by mass of Portland–pozzolana cement) together with small amounts of gypsum, or by intimately and uniformly blending Portland cement having predetermined fineness and fine *pozzolana*. While intergrinding two materials together presents no difficulty, blending of dry powders intimately is extremely difficult. The blending should be resorted to only when the intergrinding techniques prove uneconomical in a particular case and requisite machinery to ensure homogeneity or uniformity (± 3 per cent) of production is available. If the blending is not uniform, it is reflected in the performance tests.

Portland–pozzolana cement produces less *heat of hydration* and offers greater resistance to the *sulphate attack* and *chloride-ion penetration* due to impurities in water than normal Portland cement. Hence it can be conveniently used for sewers and sewage disposal works. It is particularly useful in marine and hydraulic constructions, and other *mass concrete* structures like dam, bridge piers and thick foundations. The Portland–pozzolana cement can generally be used wherever ordinary Portland cement is usable under normal conditions. However, as explained earlier all the pozzolanas need not necessarily contribute to strength at early ages. IS: 1489–1991 gives the specifications for the production of Portland–pozzolana cement equivalent to 33-grade ordinary Portland cement on the basis of seven-day *compressive strength*. The compressive strength of Portland–pozzolana cement at 28 days also has been specified to enable the Portland–pozzolana cement to be used as substitute for ordinary Portland cement in plain and reinforced concrete works. The Portland–pozzolana cement should conform to the requirements specified in IS: 1489–1991. The average compressive strength of mortar cubes (area of face 50 cm^2) composed of one part of cement, three parts of standard sand (conforming to IS: 650–1991) by mass and $(p^*/4) + 3.0$ per cent (of combined mass of cement and sand) water obtained in the manner described in IS: 4031 (Part-6)–1988 should be as follows:

(i) at 168 ± 2h	22 MPa, minimum and
(ii) at 672 ± 4h	31 MPa, minimum

where p^* is the percentage of water to produce a paste of *standard consistency*.

The Portland cement/clinker for blending/intergrinding with fly ash should conform to IS: 269–1989. While the fly ash used in the manufacture of

Portland–pozzolana cement should conform to IS: 3812–1981. The average compressive strength in *lime reactivity* test of fly ash should not be less than 4.0 MPa. The fineness of fly ash to be used in blending should not be less than 320000 mm^2/g.

To achieve almost equal strength at 28 days, a mix with 20 per cent of cement replaced with 27.5 per cent fly ash by weight along with consequential adjustments in fine and coarse aggregates is recommended. On these lines in terms of equivalent strength a nominal mix 1:0.5:2.0:5.0 (C:FA:Sand:CA) can be used in lieu of nominal mix 1:2:4. Similarly the nominal mix 1:0.5:2.0:4.0 is equivalent to 1:1.5:3.0.

High Volume Fly Ash Portland Cement (OPC–HVFA)

The HVFA blended cement is produced by intergrinding approximately 55 per cent of a *low-calcium fly ash* and 45 per cent ordinary Portland cement clinker together with small amounts of gypsum and *high-range water-reducing admixture* (HRWRA), e.g. sulphonated naphthalene formaldehyde condensate in a dry powder form. The incorporation of HRWRA in the HVFA-blended cement helps the mortars made with the HVFA-blended cement achieve the desired compressive strength as a result of the reduction in the *water-to-blended cements ratio*. The use of HRWRA in HVFA blended cements, however, retards their setting times.

The *entrained air content* of the concrete made with HVFA-blended cements normally maintained between 5 and 7 per cent results in a satisfactory *bubble-spacing factor* in the hardened concrete. The dosage of the *air-entraining admixture* required for obtaining this air content is also strongly influenced by the type of fly ash used in the blended cement and whether or not the blended cement contains the HRWRA. The concrete made with the blended cements containing a HRWRA usually requires a lower dosage of the air-entraining admixture than that made with the blended cements without the HRWRA. The *bleeding* of concrete made with the HVFA-blended cements ranges from very low to negligible due to the low water content in the concrete. The *setting time* of the concrete is generally longer than that of concrete made with Portland cement only. In general, however, the HVFA concrete does not show unacceptable retardation in setting time and demonstrates enough strength development to result in adequate strength at one day.

Because of the low cement content, the maximum *autogenous temperature* rise in concrete made with the HVFA-blended cements is rather low. Thus, HVFA cements are ideal for concrete structures where high *heat of hydration* is a concern, e.g. *mass concrete* and *thick structural concrete member*. The cost of the blended cements depends on the cost of the cement clinker, fly ash, and the energy required for intergrinding. It is believed that the cost of the blended cements should be lower than the cost of normal Portland cement.

2.3.5 Ternary Cements

These are three cementitious constituent systems. Substantial improvements in concrete performance have been reported with ternary cements, compared with that obtained by using both binary cements and OPC. The *ternary* and *multiple blended cement* of OPC–SF–GBFS or FA have shown desired balance of *mechanical* and *durability* properties in marine and offshore environments. The need to develop ternary cements and to optimize their blends has been recognized. The ternary blended cements developed and produced are (OPC–SF–FA) and (OPC–SF–GBFS).

2.3.6 Synergistic Actions in Ternary and Quaternary Cements

The rationale behind developing ternary and quaternary cements is to evolve blends in which the various *mineral additives* combine to provide tailor-made properties by compensating for their mutual shortcomings. Various aspects of the performance of such composite cements are discussed below.

Particle Packing Density and Microstructure The original size and spatial distribution of cementitious particles have a large influence on *hydration kinetics*, *micro structural development*, and *ultimate properties of concrete*. Achieving higher particle-packing density of the cementitious blend is believed to enhance the *rheological properties* and *mechanical strength* of concrete. By simply adjusting the *particle size distribution* of cement, a 40 per cent increase in 28-day *compressive strength* can be obtained.

Durability Performance It is reported that fly ash (FA) has a synergistic action in ternary systems with respect to durability. Combinations of high-calcium fly ash and silica fume (SF) are synergistic. The ternary blend also achieves higher resistance to *sulphate attack* and *chloride penetration*.

Thus ternary and quarternary cements involving blends of *silica fume*, *rice husk ash*, metakaolin, diatomaceous earth, *slag*, *fly ash*, and limestone filler could significantly contribute to achieving the needed balance between the industry's quest for *high-performance concrete* and the increasingly restrictive environmental regulations. Synergistic effects could allow individual component ingredients in such blends to compensate for their mutual shortcomings, furthering the extent of the use of environmentally efficient cements and promoting their field implementation.

Though such composite or blended cements are not a cure for all concrete problems, nor shall they replace all binary cements, composite cements provide a unique opportunity to produce environment-friendly concrete with tailor-made properties and may indeed constitute the next generation of cement products. However, lack of infrastructure for the production, storage, and handling of blended cements, is a big hurdle in popularizing the blended cements. There is a growing need for reliable predictive model for the behaviour of high-volume

windows being tightly shut. Once the cement has been properly stored it should not be disturbed until it is to be used. The practice of moving and restacking the bags to reduce warehouse pack only exposes fresh cement to air.

2.6 REJECTION

Due to defective storage for long periods, cement is adversely affected. The cement remaining in bulk storage with the manufactures for more than six months or cement in jute bags in local storage in the hands of dealers for more than three months after completion of tests may be retested before use and rejected if they fail to conform to any of the requirements of IS: 269–1989.

CHAPTER

3

Concrete Making Materials–II: Aggregate

3.1 INTRODUCTION

As explained in Chapter 2, concrete can be considered to be an artificial stone obtained by binding together the particles of relatively inert fine and coarse materials with cement paste. Aggregates are generally cheaper than cement and impart greater volume stability and durability to concrete. The aggregate is used primarily for the purpose of providing bulk to the concrete. To increase the density of the resulting mix, the aggregate is frequently used in two or more sizes. The most important function of the fine aggregate is to assist in producing workability and uniformity in mixture. The *fine aggregate* also assists the cement paste to hold the *coarse aggregate* particles in suspension. This action promotes plasticity in the mixture and prevents the possible *segregation* of paste and coarse aggregate, particularly when it is necessary to transport the concrete some distance from the mixing plant to point of placement.

The aggregates provide about 75 per cent of the body of the concrete and hence its influence is extremely important. They should therefore meet certain requirements if the concrete is to be workable, strong, durable, and economical. The aggregate must be of proper shape (either rounded or approximately cubical), clean, hard, strong and well graded. It should possess chemical stability and, in many cases, exhibit abrasion resistance and resistance to freezing and thawing.

3.2 CLASSIFICATION OF AGGREGATES

The classification of the aggregates is generally based on their geological origin, size, shape, unit weight, etc.

3.2.1 Classification According to Geological Origin

The aggregates are usually derived from natural sources and may have been naturally reduced to size (e.g. gravel or shingle) or may have to be reduced by crushing. The suitability of the locally available aggregate depends upon the geological history of the region. Such an aggregate may further be divided into two categories, namely the natural aggregate and artificial aggregates.

Natural Aggregate

These aggregates are generally obtained from natural deposits of sand and gravel, or from quarries by cutting rocks. Cheapest among them are the natural sand and gravel which have been reduced to their present size by natural agents, such as water, wind and snow, etc. The river deposits are the most common and are of good quality. The second most commonly used source of aggregates is the quarried rock which is reduced to size by crushing. Crushed aggregates are made by breaking rocks into requisite graded particles by blasting, crushing and screening, etc. From the petrological standpoint, 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 geological mode of formation, i.e. igneous, sedimentary or metamorphic origin, and each group may be further divided into categories having certain petrological characteristics in common. Such a classification has been adopted in IS: 383–1970.

Within each group, the quality of aggregate may vary to a great extent due to the change in structure and texture of the parent rock from place to place.

Aggregates from igneous rocks are highly satisfactory because they are normally *hard*, *tough* and *dense*. They have massive structure with crystalline/ glassy texture. The bulk of concrete aggregates are of igneous origin. The aggregate may be acidic or alkaline depending upon silica content and of light or dark colour.

The quality of aggregates derived from sedimentary rocks vary depending upon the formation history of the rock. Limestones and some siliceous sand stones have proved to be source of good concrete aggregates. Sometimes stratifications in the parent rock show up in the individual aggregates and thereby impair the strength of aggregate. Sedimentary rocks may vary from soft to hard, porous to dense and light to heavy. They may also yield flaky aggregates.

The metamorphic rocks show foliated structure. In some cases individual aggregate may exhibit foliations which is not a desirable characteristic in aggregate. However, many metamorphic rocks particularly quartizite and gneiss have provided good concrete aggregates.

Artificial Aggregate

The most widely used artificial aggregates are clean broken bricks and air-cooled fresh blast-furnace-slag. The broken bricks of good quality provide a satisfactory aggregate for the mass concrete and are not suitable for reinforced concrete work if the crushing strength of brick is less than 30 to 35 MPa. The bricks should be free from lime mortar and lime sulphate plaster. The brick

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depends on the bond between the aggregate and cement paste. The bond is partly due to the interlocking of the aggregate and paste. A rough surface results in a better bond. The bond is also affected by the physical and chemical propeties, mineralogical and chemical composition, and the electrostatic condition of the particle surface, e.g. a chemical bond may exist in the case of a limestone aggregate.

3.2.4 Classification Based on Unit Weight

The aggregates can also be classified according to their unit weights as normal-weight, heavyweight, and lightweight aggregates.

Normal-weight Aggregate

The commonly used aggregates, i.e. sands and gravels; crushed rocks such as granite, basalt, quartz, sandstone and limestone; and brick ballast, etc. which have specific gravities between 2.5 and 2.7 produce concrete with unit weight ranging from 23 to 26 kN/m^3 and crushing strength at 28 days between 15 to 40 MPa are termed normal-weight concrete. The properties and the requirements of normal-weight aggregate will be discussed in details in the succeeding sections.

Heavyweight Aggregate

Some heavyweight aggregates having specific gravities ranging from 2.8 to 2.9 and unit weights from 28 to 29 kN/m^3 such as magnetite (Fe_3O_4), barytes ($BaSO_4$) and scrap iron are used in the manufacture of heavyweight concrete which is more effective as a radiation shield. Concretes having unit weight of about 30 kN/m^3, 36 kN/m^3 and 57 kN/m^3 can be produced by using magnetite, baryte and scrap iron, respectively. The compressive strength of these concretes is of the order of 20 to 21 MPa. The cement–aggregate ratio varies from 1:5 to 1:9 with a *water–cement ratio* between 0.5 to 0.65. They produce dense and crack-free concrete. The main drawback with these aggregates is that they are not suitably graded and hence it is difficult to have adequate *workability* without *segregation.*

Lightweight Aggregate

The lightweight aggregates having unit weight up to 12 kN/m^3 are used to manufacture the structural concrete and masonry blocks for reduction of the self-weight of the structure. These aggregates can be either natural, such as diotomite, pumice, volcanic cinder, etc. or manufactured, such as bloated clay, sintered fly ash or foamed blast-furnace-slag. In addition to reduction in the weight, the concrete produced by using light-weight aggregate provides better thermal insulation and improved fire resistance.

The main requirement of the lightweight aggregate is its low density; some specifications limit the unit weight to 12 kN/m^3 for fine aggregate and approximately 10 kN/m^3 for coarse aggregates for the use in concrete. Because

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The *flakiness index* of coarse aggregate is generally limited to 25 per cent. The *surface texture* is a measure of the smoothness or roughness of the aggregate. Based on the visual examination of the specimen, the *surface texture* may be classified as *glassy, smooth, granular, rough, crystalline, porous* and *honeycombed.* The strength of the *bond* between aggregate and cement paste depends upon the surface texture. The bond is the development of mechanical anchorage and depends upon the surface roughness and surface porosity of the aggregate. An aggregate with a rough, porous texture is preferred to one with a smooth surface as the former can increase the *aggregate–cement bond* by 75 per cent, which may increase the compressive and flexural strength of concrete up to 20 per cent. The surface pores help in the development of good bond on account of suction of paste into these pores. This explains the fact that some aggregates which appear smooth still bond strongly than the one with rough surface texture.

The shape and surface texture of fine aggregate govern its void content and thus affect the water requirement of mix significantly. The use of crushed or manufactured sand with proper shape, surface texture and grading has enabled production of highly workable mix with minimum void content.

Specific Gravity

The *specific gravity* of an aggregate is defined as the ratio of the mass of solid in a given volume of sample to the mass of an equal volume of water at the same temperature. Since the aggregate generally contains voids, there are different types of specific gravities.

The *absolute specific gravity* refers to the volume of solid material excluding the voids, and therefore, is defined as the ratio of the mass of solid to the weight of an equal void-free volume of water at a stated temperature. If the volume of aggregate includes the voids, the resulting specific gravity is called the *apparent specific gravity.* As the aggregate generally contains both impermeable and capillary voids (voids between the particles), the apparent specific gravity refers to volume including impermeable voids only. It is therefore the ratio of the mass of the aggregate dried in an oven at 100 to 110 °C for 24 hours to the mass of the water occupying a volume equal to that of solids including impermeable voids or pores. The specific gravity most frequently and easily determined is based on the saturated surface dry condition of the aggregate because the water absorbed in the pores of the aggregate does not take part in the chemical reaction of the cement and can therefore be considered as a part of the aggregate. This specific gravity is required for the calculations of the yield of concrete or of the quantity of aggregate required for a given volume of concrete. The specific gravity of an aggregate gives valuable information on its quality and properties. It is seen that the higher the specific gravity of an aggregate, the harder and stronger it will be. If the specific gravity is above or below that which is normally assigned to a particular type of aggregate, it may indicate that the shape and grading of the aggregate has changed.

amount intended for the *nominal mix* used (based on dry sand). If no allowance is made for the bulking of sand a nominal concrete mix 1:2:4, for example, will correspond to 1:1.74:4 for a bulking of 15 per cent. An increase in bulking from 15 to 30 per cent will result in an increase in the *concrete strength* by as much as 14 per cent. If no allowance is made for bulking the concrete strength may vary by as much as 25 per cent.

3.4 DELETERIOUS SUBSTANCES IN AGGREGATES

The *materials* whose presence may adversely affect the *strength, workability* and long-term *performance of concrete* are termed *deleterious materials.* These are considered undesirable as constituent because of their intrinsic weakness, softness, fineness or other physical or chemical characteristics harmful to the concrete behaviour.

Depending upon their action, the deleterious substances found in the aggregates can be divided into three broad categories:

(i) impurities interfering with the process of hydration of cements,
(ii) coatings preventing the development of good bond between aggregate and the cement paste, and
(iii) unsound particles which are weak or bring about chemical reaction between the aggregate and cement paste.

The impurities in the form of organic matter interfere with the chemical reactions of *hydration.* These impurities generally consisting of decayed vegetable matter and appearing in the form of humus or organic loam are more likely to be present in *fine aggregate* than in *coarse aggregate* which is easily washed. The effect of impurities is tested as per IS: 2386 (Part-II)–1963.

The clay and other fine materials, such as silt and crusher dust may be present in the form of surface coatings which interfere with the *bond* between the aggregate and the cement paste. Since a good bond is essential for ensuring satisfactory *strength* and *durability* of concrete, the problem of *coating of impurities* is an important one. The soft or loosely adherent coatings can be removed by washing. The well-bonded chemically stable coatings have no harmful effect except that the shrinkage may be increased. However, an aggregate with chemically reactive coatings can lead to serious trouble. The silt and the fine dust, if present in excessive amounts, increase the *specific surface of the aggregate* and hence the amount of water required to wet all particles in the mix, thereby reducing the strength and durability of concrete.

The total amount of deleterious material should not exceed 5 per cent as per IS: 383–1970. The limits of deleterious materials are given in Table 3.1.

The sand obtained from a seashore or a river estuary contains salt and sometimes its percentage may be as high as 6 per cent of mass of sand. The salt can be removed from the sand by washing it with fresh water before use. If salt

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approximately 5.4×10^{-6} and 12.6×10^{-6} per °C. For hydrated portland cement the coefficient varies between 10.8×10^{-6} and 16.2×10^{-6} per °C.

Whereas, the linear thermal coefficient of expansion of concrete lies in the range of about 5.8×10^{-6} per °C to 14×10^{-6} per °C depending upon the type of aggregate, mix proportions, degree of saturation, etc. It can be determined by Verbeck's dilatometer. It is observed that while there is thermal compatibility at higher range, there exists thermal incompatibility at the lower range. This thermal incompatibility at lower range causes severe stress affecting durability and integrity of concrete structure. When concrete is subjected to a high range of temperature difference the adverse effects become acute. The coefficient of thermal expansion also affects the fire resistance of the concrete.

The *specific heat* of the aggregate is a measure of its heat capacity, whereas the *thermal conductivity* is the ability of the aggregate to conduct the heat. These properties of the aggregate influence the specific heat and thermal conductivity of the concrete, and are important in the case of mass concrete and where insulation is required.

3.8 FINENESS MODULUS

The *fineness modulus* is a numerical index of fineness, giving some idea of the *mean size of the particles* present in the entire body of the aggregate. The determination of the fineness modulus consists in dividing a sample of aggregate into fractions of different sizes by sieving through a set of standard test sieves taken in order. Each fraction contains particles between definite limits. The limits being the opening sizes of standard test sieves. The material retained on each sieve after sieving represents the fraction of aggregate coarser than the sieve in question but finer than the sieve above. The sum of the *cumulative percentages* retained on the sieves divided by 100 give the fineness modulus. The sieves that are to be used for the sieve analysis of the aggregate (coarse, fine or all-in-aggregate) for concrete as per IS: 2386 (Part-I)–1963, are 80 mm, 40 mm, 20 mm, 10 mm, 4.75 mm, 2.36 mm, 1.18 mm, 600 µm, 300 µm and 150 µm.

The fineness modulus can be regarded as a *weighted average size* of a sieve on which material is retained, and the sieves being counted from the finest. For example, a fineness modulus of 6.0 can be interpreted to mean that the sixth sieve, i.e. 4.75 mm is the average size. The value of fineness modulus is higher for coarser aggregate. For the aggregates commonly used, the fineness modulus *of fine aggregate* varies between 2.0 and 3.5, for *coarse aggregate* it varies between 5.5 and 8.0, and from 3.5 to 6.5 for *all-in-aggregate.*

The object of finding fineness modulus is to grade the given aggregate for the most economical mix for the required *strength* and *workability* with minimum quantity of cement. If the test aggregate gives higher fineness modulus, the mix will be harsh and if, on the other hand, gives a lower fineness modulus it will produce an uneconomical mix. For *workability,* a coarser aggregate requires less

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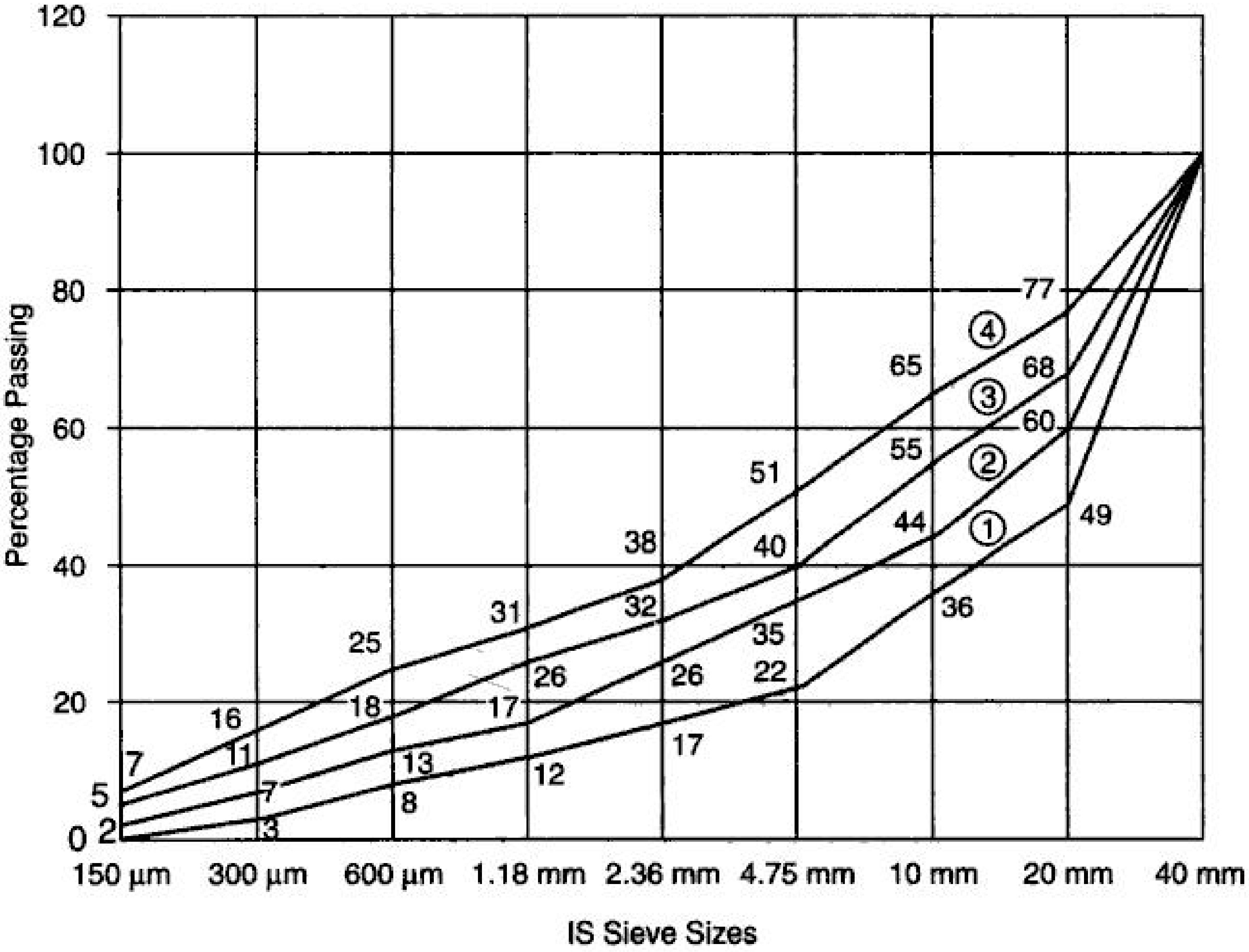

Fig. 3.2 Recommended grading curves for 40 mm nominal maximum size aggregate

No.4 is suitable for *lean mixes* where a high workability is required. The change from one extreme to the other is progressive. The outer curves 1 and 4 represent the limits for the normal *continuous gradings.* The saving in cement affected by using a coarse grading can be considerable. If the locally available aggregate does not conform to the desired grading, the finer and the coarser fractions of aggregates can be suitably combined to obtain the desired grading. This can be achieved either by analytical calculations or graphically as explained in Section 10.3.4 of Chapter 10.

3.10.1 Gap-graded Aggregate

Gap-grading is defined as a grading in which one or more intermediate-size fractions are absent. The term 'continuously graded' is used to distinguish the conventional grading from gap-grading. On a grading curve, gap-grading is represented by a horizontal line over the range of the size omitted. Some of the important features of gap-graded aggregate are as follows.

(i) For the given *aggregate–cement* and *water–cement ratios* the highest workability is obtained with lower sand content in the case of gap-graded aggregate rather than when continuously graded aggregated is used.

(ii) In the more workable range of mixes, gap-graded aggregates show a greater tendency to segregation. Hence gap-grading is recommended

Table 3.4 *Grading Limits for Fine Aggregates (IS: 383–1970)*

IS sieve designation	*Percentage passing by weight* *Grading* *Zone I*	 *Zone II*	 *Zone III*	 *Zone IV*
10 mm	100	100	100	100
4.75 mm	90–100	90–100	90–100	95–100
2.36 mm	60–95	75–100	85–100	95–100
1.18 mm	30–70	55–90	75–100	90–100
600 μm	15–34	35–59	60–79	80–100
300 μm	5–20	8–30	12–40	15–50
150 μm	0–10	0–10	0–10	0–15
Fineness modulus	4.0–2.71	3.37–2.10	2.78–1.71	2.25–1.35

Table 3.5 *Grading Limits for All-in-Aggregate (IS: 383–1970)*

IS sieve designation	*Percentage by weight passing for all-in-aggregate of* *40 mm nominal size*	 *20 mm nominal size*
80 mm	100	–
40 mm	95–100	100
20 mm	45–75	95–100
4.75 mm	25–45	30–50
600 μm	8–30	10–35
150 μm	0–5	0–6

results in a harsh concrete mix requiring the use of superplasticizers to improve the workability. With the advent of modern crushers specially designed for producing cubical, comparatively smooth textured and well graded sand, the crushed sand is fast replacing natural sand. New technologies are available for producing coarse and fine aggregates of the desired quality in terms of shape, texture and grading. The dust, i.e. the portion of aggregates consisting of particles of size finer than 75 micron, is limited to 15 per cent in fine aggregate and 3 per cent in coarse aggregate. Generally, the manufactured sands conform to the grading Zones I and II of fine aggregates as given in the Table 3.7.

Table 3.6 *Suggested Proportion of Fine to Coarse Aggregate for Different Size of Aggregates*

Nominal size of graded coarse aggregate, mm	*Fine aggregate: Coarse aggregate, for the sand of zone* *Zone I*	 *Zone II*	 *Zone III*	 *Zone IV*
10	1:1	1:1.5	1:2	1:3
20	1:1.5	1:2	1:3	1:3.5
40	1:2	1:4	1:3.5	–

the distress of concrete, have been found to be a result of, among others, the mixing and curing water being of in appropriate quality. Potable water is generally considered satisfactory for mixing concrete. In the case of doubt about the suitability of water, particularly in remote areas or where water is derived from sources not normally utilized for domestic purposes, water should be tested.

4.2.1 Effect of Impurities in Water on Properties of Concrete

The strength and durability of concrete is reduced due to the presence of impurities in the *mixing water*. The effects are expressed mainly in terms of difference in the setting times of Portland cement mixes containing proposed mixing water as compared to distilled water, and concrete strengths compared with those of control specimens prepared with distilled water. A difference in 28-days compressive strength up to 10 per cent of control test is generally considered to be a satisfactory measure of the quality of mixing water. IS: 456–2000 prescribes a difference in initial setting time of ± 30 minutes with initial setting time not less than 30 minutes. The effluents from sewerage works, gas works, and from paint, textile, sugar and fertilizer industry are harmful for concrete. The tests show that water containing excessive amounts of dissolved salts reduces compressive strength by 10 to 30 per cent of that obtained using potable water. In addition, water containing large quantities of chlorides tends to cause persistent *dampness*, surface *efflorescence* and increases the *corrosion* of the reinforcing steel. The problem is more in tropical regions, particularly with lean mixes.

The adverse effects on compressive strength of concrete due to various dissolved salts are given in Table 4.1

Table 4.1 *Effects of Dissolved Salts in Water on Compressive Strength*

Percentage of salt in water	*Percentage reduction in compressive strength*
0.5 SO_4	4
1.0 SO_4	10
5.0 NaCl	30
CO_2	20

The effect of various impurities on the properties of concrete are summarized below.

Suspended Particles

The presence of suspended particles of clay and silt in the mixing water up to 0.02 per cent by weight of water does not affect the properties of concrete. Even higher percentage can be tolerated so far as strength is concerned, but other properties of concrete are affected. IS: 456–2000 allows 2000 mg/litre of

to IS: 456–2000, the presence of tannic acid or iron compounds in curing water is objectionable.

It is generally recommended that the seawater should not be used as mixing water for hydraulic-cement concrete works containing corrodible embedded ferrous metals, particularly in the tropics. However, under unavoidable circumstances it may be used for mixing and curing in plain concrete after due evaluation of possible disadvantages and consideration of the use of appropriate cement system.

The general action of accelerators is to cause a more rapid dissolution of compounds of cement, particularly tricalcium silicate, in water and hence facilitate more rapid hydration of these compounds. *The mechanism of action is catalytic in nature.* The most widely used accelerator is *calcium chloride* ($CaCl_2$). It is available as flakes (77 per cent $CaCl_2$) or in the fused from (92 per cent $CaCl_2$). It is always dissolved in a part of mixing water before use. It is the solid mass, which is reckoned in the admixture. Calcium chloride can generally be used in amounts up to 2 per cent by mass of cement, but IS: 7861 (Part-II)–1981 recommends a maximum of 1.5 per cent of $CaCl_2$ for plain and reinforced concrete works in cold weather conditions. However, $CaCl_2$ or admixtures containing soluble chlorides are not permitted to be used in prestressed concrete due to the possibility of stress corrosion. The benefits of the use of calcium chloride are usually more pronounced when it is employed for concreting at temperatures below 25 °C. Calcium chloride should not be used in concrete which will be subjected to *alkali-aggregate reaction* or exposed to soils or to water containing sulphates, in order to avoid lowering of the resistance of concrete to *sulphate attack.*

The use of 2 per cent *calcium chloride* by mass of cement can reduce the setting time by one-third and raise the one to seven day *compressive strength* by 3 to 8 MPa. An increase in *flexural strength* of 40 to 80 per cent of one day and up to 12 per cent at 28 days is obtained. The selection of the optimum amount should be based on the type of cement, temperature of concrete and the ambient temperature. Large doses of $CaCl_2$ result in *flash set* of concrete and also in increased shrinkage. The effect of $CaCl_2$ on the compressive strength of concrete is shown in Fig. 5.1. *Calcium formate* (a fine powder), which is somewhat less soluble than calcium chloride and is less effective does not have the same adverse effect on corrosion of embedded steel as $CaCl_2$. It is added in the same dosages.

Some of the accelerators containing fluro-silicates and trietholamine are capable of reducing the period during which concrete remains plastic to less than 10 minutes. An accelerator produced under the trade name '*Quickset*' which when added to neat cement results in the setting in a matter of seconds. This makes it valuable for making cement plugs to stop pressure leaks. They are added only by a small percentage usually not exceeding 0.2 per cent by mass of cement.

The other less commonly used accelerators consist of $NaCl$, Na_2SO_4, $NaOH$, Na_2CO_3, K_2SO_4 and KOH. In contrast to $CaSO_4$, the effect of Na_2SO_4 and K_2SO_4 is the acceleration of hydration of cement. Rapid hydration can be achieved in the first two hours by the addition of $NaOH$ or KOH. It has been noticed that two per cent of calcium chloride has the same effect on the acceleration of hydration as a rise in temperature of about 11 °C.

Most accelerating admixtures do not significantly affect *rheology* (*flowability*) and hence the *consistency* of cement paste at early ages. However, at later ages

Table 5.1 *Optimum Air Content of Concrete*

Maximum size of aggregate, mm	*Air-content, per cent by volume*	
	Naturally entrained	*Optimal total*
Sand–cement mortar	4.0	14 ± 2
10	3.0	8 ± 1.5
12.5	2.5	7.5 ± 1.5
20	2.0	7.0 ± 1.5
25	1.5	6 ± 1.5
40	1.0	4.5 ± 1.5
50	0.5	4.0 ± 1.0
70	0.3	3.5 ± 1.0
150	0.2	3.0 ± 0.5

water-cement ratio although initially, a large quantity of air entrainment is achieved, however, a large proportion of the bubbles is lost progressively with time. The *grading of aggregate* has significant influence on the quantity of air entrainment.

The entrained air content increases with the mixing time up to a certain limit and thereafter with prolonged mixing the air content gets reduced. The temperature of concrete at the time of mixing has significant effect on the amount of air entrainment. The entrained air content decreases with the increase in temperature of concrete.

Air content is also reduced by the process of compaction, on account of the movement of air bubbles to the surface and their subsequent destruction. It is estimated that as much as 50 per cent of the entrained air may be lost after vibration for 2–2 ½ minutes and as much as 80 per cent may be lost by vibration for about 10 minutes. Similarly, the use of calcium chloride has the tendency to limit air entrainment.

Thus air-entrained concrete is considerably more plastic and workable than non-air-entrained concrete. It is observed that the *placeability* of *air entrained concrete* having 75 mm slump is superior to that of *non-air entrained concrete* having 125 mm slump. This easier placeability of a lower slump concrete should be recognized with respect to concrete construction in difficult situations. The durability of hardened concrete is improved by *increased uniformity, decreased absorption* and *permeability*, and by *elimination of planes of weaknesses* at the top of lifts. Thus there is considerable increase in the *resistance to freezing and thawing* and to the disruptive action of de-icing salts. During freeze cycles, the pressure exerted by the expanded volume of ice is taken up by the air bubbles acting like tiny springs and during thaw cycles these bubbles revert back to their original size. As suggested by Blanks the resistance of concrete to *freezing and thawing* can be measured by means of *durability factor* which is defined as the number of cycles of freezing and thawing to produce failure divided by 100. The *air-entraining agents* also find very useful applications in making *cellular concrete* and *lightweight aggregate concrete.*

Water-reducing admixtures enable a given fresh concrete mix to have higher flowability (workability) without increasing the water content which results in faster rate of concrete placement, easy placement in relatively poorly accessible locations without vibration, true shutter finish for highly reinforced concrete members, and reduction in cement content. Benefits of water reduction in hardened state of concrete are *increased strength*, *density*, *durability*, *volume stability*, *abrasion resistance*, *reduced permeability* and *cracking*. The specific effect of water-reducing and set-controlling admixtures vary with the *type of cement*, *water–cement ratio*, *mixing temperature*, *ambient temperature* and other job conditions, and therefore, it is generally recommended that the admixture used be adjusted to meet the job conditions.

A good plasticizer fluidises the mortar or concrete in a manner different from that of the air-entraining agents. Many of the plasticizers, while improving the workability, also entrain some air. Since the entrainment of air reduces the mechanical strength, a good plasticizer is the one, which does not entrain air more than 1 to 2 per cent.

Action of Plasticizers

The action of plasticizers is mainly to fluidify the mix and improve the workability of concrete, mortar or grout. The mechanisms involved are:

(i) dispersion,
(ii) retarding effect,

Dispersion Portland cement, being in fine state, will have a tendency to flocculate in wet concrete. This flocculation entraps certain amount of water used in the mix and thereby all the water is not freely available to fluidify the mix. When plasticizers are added, they get adsorbed on the cement particles. The adsorption of charged polymer on the cement particles creates repulsive forces between particles, which overcome the attractive forces. This *repulsive force is called Zeta Potential*, which depends on the base, solid content, quantity of plasticizer used. The overall result is that the cement particles are deflocculated and dispersed. When cement particles are deflocculated, the water trapped inside the flocs gets released and becomes available to fluidify the mix. Moreover, in the flocculated state there is interparticle friction between particle and particle, and floc and floc. But in the dispersed state due to presence of water in between the cement particles the interparticle friction is reduced.

Retarding Effect As mentioned earlier the plasticizer gets adsorbed on the surface of cement particles and forms a thin sheath, which inhibits the surface hydration of cement as long as sufficient plasticizer molecules are available at the particle-solution interface. The quantity of available plasticizers progressively decreases as the polymers get entrapped in hydration products. One or more of the following mechanisms may take place simultaneously.

(i) Reduction in the *surface tension* of water.

These condensates are usually employed in the form of sodium salts, which are easily soluble in water. This category of superplasticizers is the nearest to ideal one as they do not interfere with hydration of cement, i.e. they have very little effect on set, even at high dosage and do not have any tendency to entrain air. The dosage can be up to 3 per cent by mass of cement, beyond which beneficial effect is minimized.

In water-reduced concrete, this category results in a fairly *rapid loss of workability* and the *set time* may be accelerated by about 30 to 40 minutes over that of normal mix of equal workability. Twenty-four hour strength is typically in excess of 150 per cent of that of the normal concrete. The *workability* of a *flowing mix* may fall to a 70 mm slump in less than 15 minutes at 40 °C. It is therefore, preferable to add this category of superplasticizer directly into the *ready mix truck* at the job site and then place the concrete as quickly as possible. If workability is lost before placing, a second dose of this admixture may be added to restore workability without significant loss of mechanical properties of the hardened concrete. This procedure is not normally recommended with other categories of superplasticizer.

Category-A superplasticizers, which tend to reduce *air entrainment*, result in a mix which may be more prone to bleeding, and *segregation*, so a higher than normal *sand content* is desirable. Vibration should, therefore, be kept to a minimum. This type of superplasticizers should preferably be used for: low temperature concreting, and where high early strength is required.

Category B: Sulphonated Naphthalene–Formaldehyde Condensates (SNF) The most widely accepted compounds of this group are the poly-B-naphthalene sulphonates, having molecular weight of about 2000. These materials have a significant effect on *surface tension* and are to be used with *defoaming materials*. These condensates are employed in the form of sodium salts for their easy solubility in water.

This category of superplasticizers gives not only slightly greater levels of *set retardation* and *air entrainment* than those of category-A, but also gives significantly larger *periods of workability retention*. This makes it possible for the *admixture* to be dosed at a *ready mix plant* prior to trucking to the site. The increase in the level of air entrainment is too low to affect the cohesiveness of the mix so a *high sand content* is desirable with the high workability mixes to prevent *bleeding* and *segregation*.

The longer period of *workability retention* coupled with a set retardation of 20 to 40 minutes in high strength low water content mixes, makes this category of superplasticizers very effective for *pre-cast concrete*.

Category C: Modified-Lignosulphonates (MLS) The lignosulphonates are naturally occurring *macromolecular organic compounds*. The crude lignosulphonate is the waste liquid product obtained during the process of production of paper making pulp from wood. These liquids contain a complex mixture of

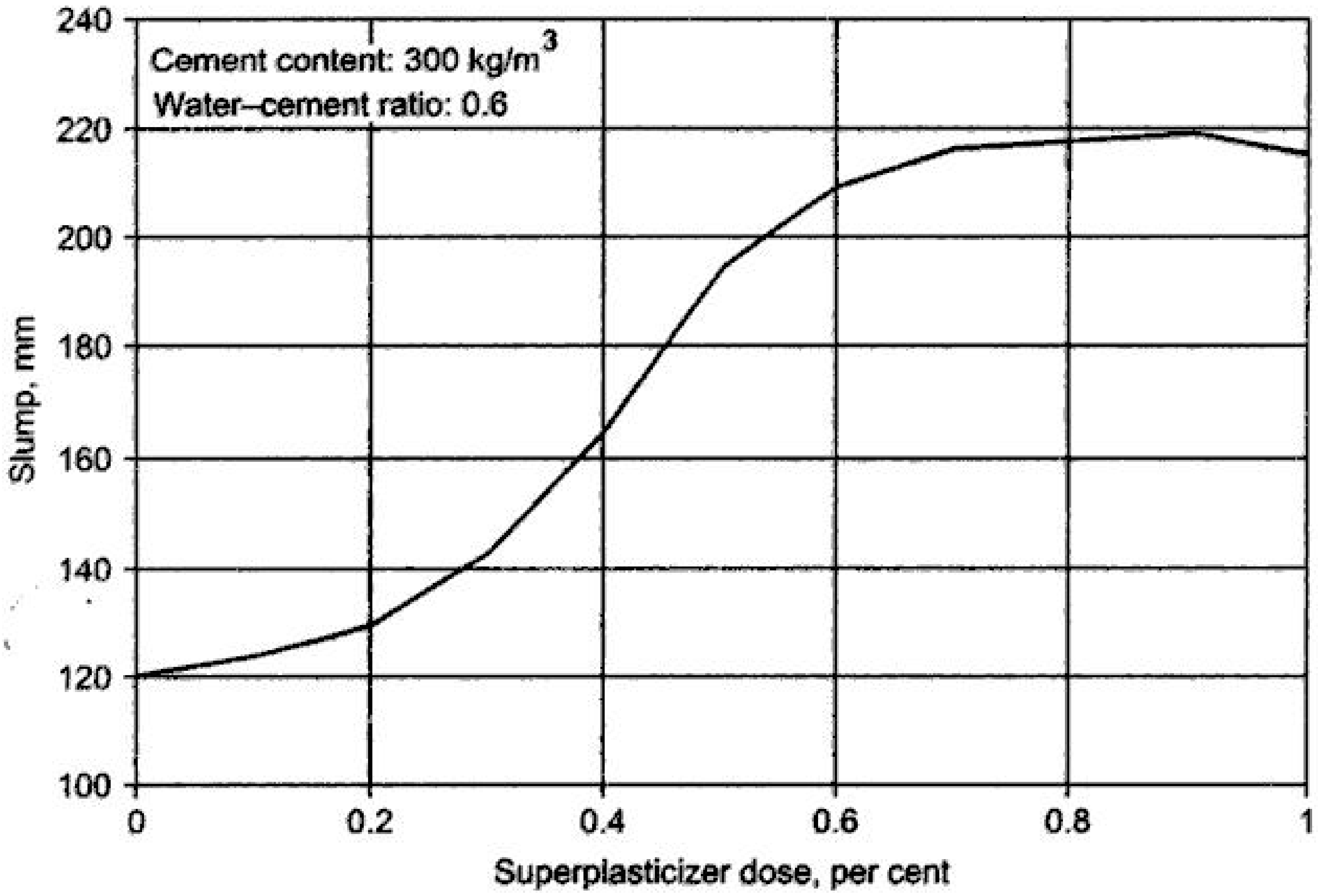

Fig. 5.6 Superplasticizer dose Vs workability in terms of slump

The dosage of plasticizers normally do not exceed 0.25 per cent by weight of cement in case of lignosulphonates, or 0.1 per cent in case of carboxylic acids, the plasticizers of types SMF or SNF require considerably high dosages (0.5 to 3.00 per cent), since they do not entrain air. The modified lignosulphonate based admixtures, which have effective fluidising action, at the relatively high dosages, can produce undesirable effects, such as unduly large accelerations or delays in the setting times. Moreover, they increase the air-entrainment in concrete.

Plasticizers or superplastizers at nominal dosage can only fluidise a mix with an initial slump of about 20 to 30 mm. A high dosage is required to fluidify no slump concrete. An improvement in slump can be obtained to the extent of 250 mm or more depending upon the initial slump of the mix, the dosage and cement content.

The dosage of superplasticizer influences the viscosity of cement matrix and hence the workability of concrete. The optimum dosage can be ascertained from *Marsh cone test* if the brand of cement, plasticizer and water–cement ratio is predetermined. Simple March test cone can give realistic dosage when instructions given by manufacturers are general in nature.

Influence of Cement Composition The finer the cement, the higher the superplasticizer dosage required to achieve a given *workability*, i.e. higher the *cement fineness*, the lower is the *fluidizing effect*. Among the cement constituents, which exert major influence on the properties of superplasticized mixes are: the C_3A content and its morphology, the alkali content, and the form

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and admixture factors that influence the *rheological behaviour in HPC* are: *tricalcium aluminate* (C_3A) *content, Blaine fineness of cement* and the *solubility of the gypsum* present in the cement; the *monomer content* and the *molecular mass fractions.*

5.3.5 Grouting Admixtures

The grouting has become one of the most important operations in civil engineering construction. Grouting below the base plate or machine foundations, grouting of foundation bolt holes in industrial structures, grouting of prestressed concrete ducts, grouting in anchoring and rock bolting systems, grouting of curtain walls, grouting of fissured rocks below dam foundation, grouting the body of the newly constructed dam itself, grouting of deteriorated concrete or fire affected structures for strengthening and rehabilitation, grouting of oil wells are some of the few situations where grouting is extensively used.

The grout material should have high early and ultimate strength, should be free flowing even at low water content, should develop good bond with previously set or hardened concrete, essentially it should be *non-shrink* in nature. The grouting materials can be broadly classified into two categories. One is free flow grout for use in machine foundations, foundation bolts and fixing crane rails, etc. The second category of grout is meant for injection grouting to fill up small cracks, and is normally accomplished under pressure.

Some retarders are especially useful in cement grout slurries, particularly where the grouting is prolonged, or in the cases where the grout must be pumped for a considerable distance, or where hot water or high temperature is encountered underground. Cement grouts containing *pozzolanic materials* are often used in cementing oil wells. Admixtures are also used to prevent the rapid loss of water from cement paste to the surrounding formation. Some of the grouting admixtures are gels, clays, pregelatinized starch and methylcellulose.

5.3.6 Air-detraining Admixtures

These materials are used to:

(i) dissipate excess air or other gases, and
(ii) remove a part of the entrained air from a concrete mixture.

A number of compounds, such as tributyl-phosphate, dibutylphathalate, water-insoluble alcohols and silicones have been proposed for this purpose. However, tributyl- phosphate is the most widely used air-detraining agent.

5.3.7 Gas-forming Admixture

These admixtures when added to mortar or concrete mixture react chemically with hydroxides present in the cement and form *minute bubbles of hydrogen gas* of size ranging from 0.1 to 1 mm throughout the cement–water matrix. This action, when properly controlled, causes a slight expansion in plastic concrete

can be used to join two surfaces. These admixtures increase the bond strength between the old and new concrete. The major applications include: overlay on an existing pavement, provision of screed over roof for waterproofing, repair work, etc.

There are two types of *bonding admixtures* in common use. In the first type the boding is accomplished by a metallic aggregate and in the other synthetic latex emulsions are used. The metallic aggregate type of admixture consists of fine cast-iron particles to which is added a chemical that causes them to oxidize rapidly when mixed with Portland cement and water. The rapid oxidation of the iron particles in the cement slurry applied over the old concrete surface results in the expansion of iron particles. The tiny fingers that thrust out into both the old and the new concrete bind them together. This admixture can also be used as waterproofer by applying additional coats. Successive coats build up a thin but *dense watertight film* over the surface.

There are a number of types of *synthetic latex bonding admixtures*, which essentially consist of highly polymerised synthetic liquid resins dispersed in water. The commonly used *polymer bonding admixtures* are made from natural rubber, synthetic rubber or any of a large number of organic polymers or copolymers. The polymers include polyvinyl chloride, polyvinyl acetate, acrylics and butadiene styrene copolymers. These admixtures are *water-emulsions,* which are generally added to the mixtures in proportions equivalent to 5 to 20 per cent by mass of cement depending upon the actual bonding requirements. Since these admixtures are emulsions, the bonding agent must lose water for its adhesive ingredients to set. When a bonding agent is sprayed on a concrete surface, the pores in the concrete absorb the water and allow the resin particles to coalesce and bond. When a bonding agent is mixed with cement paste or a mortar, the water is used in the hydration of cement and the resin is left to bind both the surfaces.

Many kinds of commercial concrete surface repair materials, mostly modified polymers, are available in the market for effective repairs. They adhere very firmly to the old concrete surface on account of greatly improved *bond* characteristics. They can be successfully used for repairing the chipped off columns, peeled off ceilings and pitted floors.

5.3.12 Concrete Surface-hardening Admixture

The plain concrete surfaces subjected to heavy traffic or the industrial buildings continuously subjected to wear and tear deteriorate after a period of time. The factory floor, on account of movement of materials, iron tyred trollies, vibrations caused by running machines is likely to suffer damages. Wear resistant and chemical resistant floor must be provided in the beginning itself. Replacing and repairing of old floors will interfere with the productivity and prove to be costly.

The hardener commonly used to prevent the destruction of the surface can be divided into two groups, namely, the chemical hardeners and fine metallic

Table 5.3 *Preferred Point of Addition*

Admixture type	*Point of addition*	*Remarks*
(a) All water reducing admixtures except superplasticizers used for flowing or self-leveling concrete.	1. Admixture to be dissolved in a part of mixing water. 2. All materials including the remaining water to be mixed for at least 30 sec (preferably for one minute) 3. Aggregates, if moist, can be mixed with cement for 30 sec to 1 min before the mixing water containing the admixture is added.	The purpose is to allow some partial hydration of cement particles before water reducing or high water-reducing agents come into contact with cement. The dispersive action is better and lasts longer.
(b) Air-entraining agents.	Dissolved in mixing water.	Not sensitive to the point of addition, but important to achieve uniform dispersion in the mix.
(c) Accelerators (except those in powder form).	Added to aggregates and cement in the usual manner.	
(d) Emulsified water-proofing agents.		
(e) Powdered water-proofing agents.	Pre-mixed with dry aggregates and cement before mixing water is added.	To ensure uniform dispersion the powders may be sprinkled into the mixer as the aggregates and dry cement are being mixed.
(f) Powdered accelerators.		
(g) Superplasticizer used for flowing or self-leveling concrete.	After mixing and transporting and just before placing.	Otherwise, the effect will be absent. It is to be noted that for flowing concrete the dosage will normally be high.

can be used individually or in combination with *Portland* or *blended cement* or *as a partial replacement of Portland cement.*

The pozzolanic materials can be divided into two groups namely, natural pozzolanas and artificial pozzolanas. The typical examples of natural pozzolana are: clay, shales, opaline cherts, diatomaceous earth, and volcanic tuffs and pumicites. The commonly used artificial pozzolanas are: fly ash, blast-furnace-slag, silica fume, rice husk ash, metakaoline, and surkhi.

Other mineral additives, like finely ground marble, quartz, granite powder are also used. They neither exhibit the pozzolanic property nor the cementitious properties. They just act as *inert fillers.*

The pozzolanic materials when used as replacement are generally substituted for 10 to 50 per cent of cement. This substitution produces concrete that is more permeable but much more resistant to the action of salt, sulphate or acidic water. Strength gain is usually slower than for the normal concrete.

Table 5.5 Approximate Physio-chemical Characteristics of Various Cementitious Materials

	Oxide composition by weight, per cent									*Physical properties*				
Material	*CaO*	*SiO_2*	*Al_2O_3*	*Fe_2O_3*	*MgO*	*SO_3*	*Alkalis $Na_2O + K_2O$*	*Carbon*	*Glass*	*Particle size, μm*	*Specific Gravity*	*Specific surface (Blaine's), m^2/kg*	*Loss on ignition, per cent*	*Bulk density, kg/m^3*
OPC Clinker	60–67	17–25	3.8	0.5–6.0	0.1–4.0	1.3–3.0	0.4–1.3	–	–	1–80 average:15	3.1–3.2	220–400	<5.0	1100–1300
Slag (GBFS)	30–45	25–38	15–32	0.5–2.0	4–17	–	–	–	85–98	–	2.90	325–600	–	–
Fly ash (FA)	1–7	30–60	10–30	4–10	0.2–5.0	1.5–2.5	0.4–2.6	1.0–1.5	60–90	1–100 average: 10–20	2.15–2.45 2.3–2.6 (bottom ash)	350–700	1.0–2.0	600–900
Silica fume (SF)	0.1–0.5	90–96	0.5–3.0	0.2–0.8	0.5–1.5	0.1–0.4	0.6–1.7	0.5–1.5	85–98	0.02–1.0 average: 0.1–0.3	2.2 1.3–1.4 (slurry)	15000–20000	0.7–2.5	200–300 (as produced) 1400
HRM*	–		97	–	–	–	–	–	–	1.5	2.5	–	–	–
Rice husk ash (RHA)	–	90	–	–	–	–	2.0–2.5	5.0	85–90	6–10	–	50000–100000	–	–

Note *High-reactivity metakaoline

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(iii) *As a simultaneous replacement of cement and fine aggregate* This replacement enables the strength at a specified age to be equaled depending on the water content.

The practice of adjusting the water content and the total amount of cement and fly ash contents to produce the concrete of same *workability* and *28-day compressive strength* but increased strength at later ages is very useful. The factors suggested for this adjustment are given in Table 5.8.

Table 5.8 *Multiplying Factor to Equal the 28-day Compressive Strength*

Cement replacement by fly ash (15 + 5n), per cent*	*Multiplying factor for water content (0.980–0.022n), kg/m^3*	*Multiplying factor for total amount of cement and fly ash, (1.035 + 0.015 n), kg/m^3*
15	0.980	1.035
20	0.958	1.050
25	0.936	1.065
30	0.914	1.080
35	0.892	1.095
40	0.870	1.110
45	0.848	1.125

Note: *n is an integer varying from 1 to 6.

Due to different densities of cement and fly ash 3100–3200 kg/m^3 and 2200–2400 kg/m^3, respectively, a part replacement by equal mass increases the volume of cementitious material, whereas replacement by equal volume reduces the mass. In practice the replacement is usually on a mass basis. The use of fly ash influences the *volume yield of concrete*. It has little effect on the *drying shrinkage* of concrete.

As discussed earlier in Chapter 2, *high volume fly ash* (HVFA) *concretes* have excellent mechanical and durability properties required for structural applications and pavement constructions. Zero slump HVFA concrete without superplasticizer has been extensively used for *roller-compacted concrete* applications. Due to high volume of fly ash and low water content, bleeding is often negligible. *Heat of hydration* is very low and *setting time* is little longer than that of conventional concrete demanding caution in *cold weather concreting* and *stripping of formwork*. HVFA concrete need be cured for longer duration for developing desirable mechanical properties. It has *low permeability, high resistance to freezing and thawing; high resistance to chloride ions penetration* and *sulphate attack, low alkali-aggregate* expansion, *low carbonation* and *high durability in marine environment*. Typical *mix proportions* used at Canada Centre for Mineral and Energy Technology (CANMET) are given in Table 5.9.

The compressive strengths at 28 and 365 days have been reported to be 35 ± 5 MPa and 55 ± 5 MPa, respectively. The flexural strengths at 14 and 91 days of 4.5 ± 0.5 MPa and 6.0 ± 0.5 MPa, respectively, and split tensile strength

does not have any binding property, but it reacts with Ca $(OH)_2$ liberated on hydration of cement. When water is added to cement, hydration occurs forming two primary products. The first product is *calcium–silicate–hydrate* (C–S–H) gel, that is cementitious and binds the aggregate together in concrete and the other product is calcium hydroxide $Ca(OH)_2$ which comprises up to 25 per cent of volume of hydration products. Silica fume reacts with calcium hydroxide to produce more aggregate binding C–S–H gel, simultaneously reducing calcium hydroxide. The net result is an increase in strength and durability. The second mechanism is through the *micro filler effect*. The extreme fineness of silica fume allows it to fill or pack the microscopic voids between cement particles and especially in the voids at the surface of the aggregate particles where the cement particles cannot fully cover the surface of the aggregate and fill all the available space. This so called *interface* zone influences the properties of the concrete. The effect is credited with greatly reduced *permeability* and improved paste to aggregate *bond*, and ultimate strength of concrete.

The attributes of silica fume have found their use in *shotcrete* applications, *pumped concrete*, mining and chemical industries. The *high strength concrete* made with silica fume provides high abrasion/erosion resistance. Silica fume influences the *rheological properties* of the fresh concrete, the *strength*, *porosity* and *durability* of hardened mass. Silica fume concrete with low water content is highly resistant to penetration by chloride ions.

Pozzolana is not useful in concrete made with *high-alumina cement* because it does not produce calcium hydroxide that could react with pozzolana. High-alumina cement, although a hydraulic cement, is not a Portland cement and is not used in structural concrete in most countries.

Silica fume content should be included in the total cementitious material whose mass is counted for the purpose of establishing the *cement content* of the concrete and its water–cement ratio. If the specification contains a clause laying down a minimum or maximum content of cementitious material per cubic meter of concrete, it is to be noted that the *specific gravity* of silica fume is about 2.20, compared with 3.15 for ordinary Portland cement.

5.6.4 Rice Husk Ash

Controlled combustion of rice husk in electricity-generation plants produces amorphous or non-crystalline silica with about 85–90 per cent cellular particles, i.e. the particles that are highly *micro-porous* and possess a very high surface area of 50 000–100 000 m^2/kg. This material in the size of 6–10 micron is more reactive than silica fume and easier to handle. The average composition of rice husk ash is 90 per cent amorphous silica, 5 per cent carbon, and 2 per cent K_2O. Even with small dosages, for instance 10 per cent by weight of cement *rice husk ash* can produce a very strong *transition zone* and very *low permeability* rating in concrete mixtures.

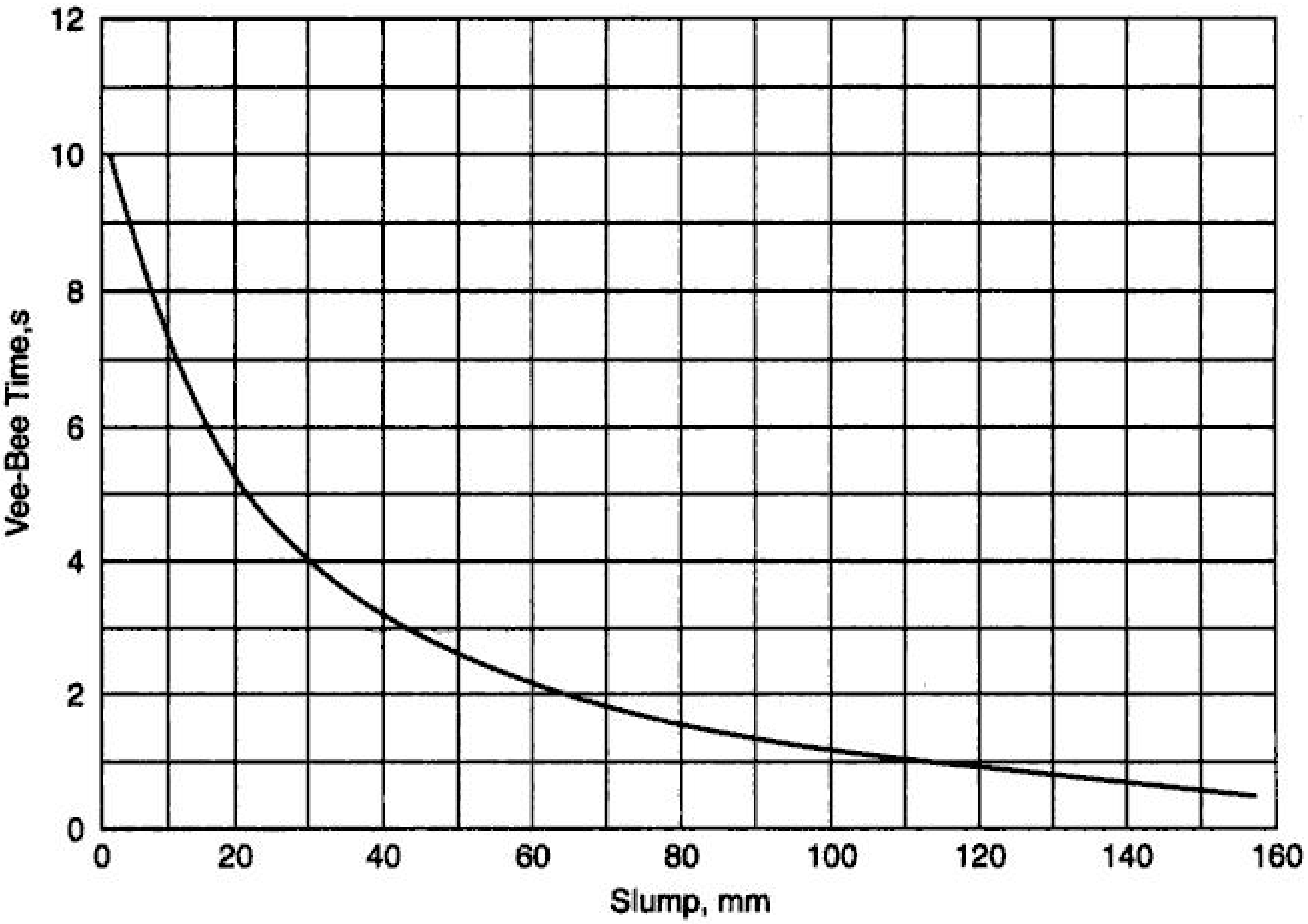

Fig. 6.3 Relationship between slump and Vee-Bee time

recommended that, for a given concrete, the appropriate test method be decided beforehand and workability be expressed in terms of such a test only, rather than be interpreted from the results of other tests. Table 6.1 gives the range of the expected values of workability measured by different test methods for comparable concretes.

In addition to the specific faults inherent to each test the major drawbacks are summarized below.

(i) The tests are quite arbitrary and empirical as far as the measurement of workability is concerned because each of these tests is a single-point test measuring a single quantity which at times may classify two such concretes 'identical', which may behave quite differently on the job.

(ii) The results from these tests are influenced by minor variations in techniques of carrying out the test, i.e. they are operator sensitive.

(iii) None of the tests is capable of dealing with concrete of whole range of workabilities, e.g. the slump test is quite incapable of differentiating between two concretes of very low workability (zero slump) or two concretes of very high workability (collapse slump). Moreover, the test results could be used as a simple statement of qualitative behaviour of concrete under particular circumstances.

However, with all their faults, the empirical tests have facilitated progress in concrete mix design. There is a strong need for the development of a new

workability as compared to ordinary Portland cement because of its higher specific surface and the fact that it hydrates more rapidly, and also the fineness of cement has an influence on bleeding.

6.4.3 Influence of Admixtures

The presence and nature of admixtures, and mineral additives affect the workability considerably. As described in Chapter 5, the plasticizers and superplasticizers improve the workability many folds. It is to be noted that *initial slump* of concrete mix, also called the slump of *reference mix* should be about 20--30 mm to enhance the slump many fold at a minimum dosage.

Use of air-entraining agents which are normally surface-active, reduces the internal friction between the particles. The air bubbles may be considered as artificial fine aggregates of very smooth surface. They also act as a sort of ball bearing between the particles to slide past each other and give easy mobility to the particles. Similarly, the fine glassy pozzolanic materials, in spite of increasing the surface area, offer better lubricating effects for giving better workability.

6.4.4 Effect of Environmental Conditions

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. The amount of mixing water required to bring about a certain change in workability also increases with temperature.

6.4.5 Effect of Time

The fresh concrete loses workability with time mainly because of the loss of moisture due to evaporation. A part of mixing water is absorbed by aggregate or lost by evaporation in the presence of sun and wind, and part of it is utilized in the chemical reaction of hydration of cement. The loss of workability varies with the type of cement, the concrete mix proportions, the initial workability and the temperature of the concrete. On an average a 125 mm slump concrete may lose about 50 mm slump in the first one hour. The workability in terms of *compacting factor* decreases by about 0.10 during the period of one hour from the time of mixing. The decrease in workability with time after mixing may be more pronounced in concrete with admixtures like *plasticizers.* For some particular total time after mixing, the loss in workability is small and initial level could be regained without loss in the strength of hardened concrete simply by adding extra water. The effect of placing time on the workability is illustrated in the Fig. 6.5.

6.5 REQUIREMENTS OF WORKABILITY

The *workability* of fresh concrete should be such that it can be placed in the formwork and compacted with minimum effort, without causing

CHAPTER

7

Rheology of Concrete

7.1 INTRODUCTION

Rheology may be defined as the science of the deformation and flow of materials, and is concerned with relationships between stress, strain, rate of strain, and time. The term rheology deals with the materials whose flow properties are more complicated than those of simple fluids (liquids or gases). The rheological principles and techniques as applied to concrete include the deformation of hardened concrete, handling and placing of freshly mixed concrete, and the behaviour of its constituent parts, namely, cement slurries and pastes. However, in this chapter, only the rheological properties of fresh concrete are considered. The *rheology* of fresh concrete like workability includes the parameters of *stability, mobility* and *compactability,* which are necessary to determine the suitability of any concrete mix as shown in Fig. 7.1. For the purpose of discussion of rheological properties of fresh concrete these parameters are redefined in terms of forces involved in the transmission of mechanical stresses on the concrete. The fresh concrete is subjected to normal and shearing forces during its handling and placing.

Stability is defined as a condition in which the aggregate particles are held in homogeneous dispersion by matrix, and random sampling shows the same particle size distribution during transportation, placing and compaction. The stability of concrete is measured by its *segregation* and *bleeding* characteristics. Segregation is defined as the mixture's instability caused by weak matrix that cannot hold individual aggregate particles in homogeneous dispersion. The resistance to segregation depends upon the *cohesion* between the particles of the mix. Segregation can occur in concretes of both wet and dry consistencies. Segregation in wet mix can occur when the water content is such that the paste cannot hold aggregate particles in the distributed position while the concrete is transported, placed and compacted. Dry segregation takes place when a concrete with low *water–cement ratio* results in a crumbly mix during handling. However, the crumbly mixes are often satisfactory when the concrete is vibrated, as during vibration the matrix becomes fluid momentarily and develops

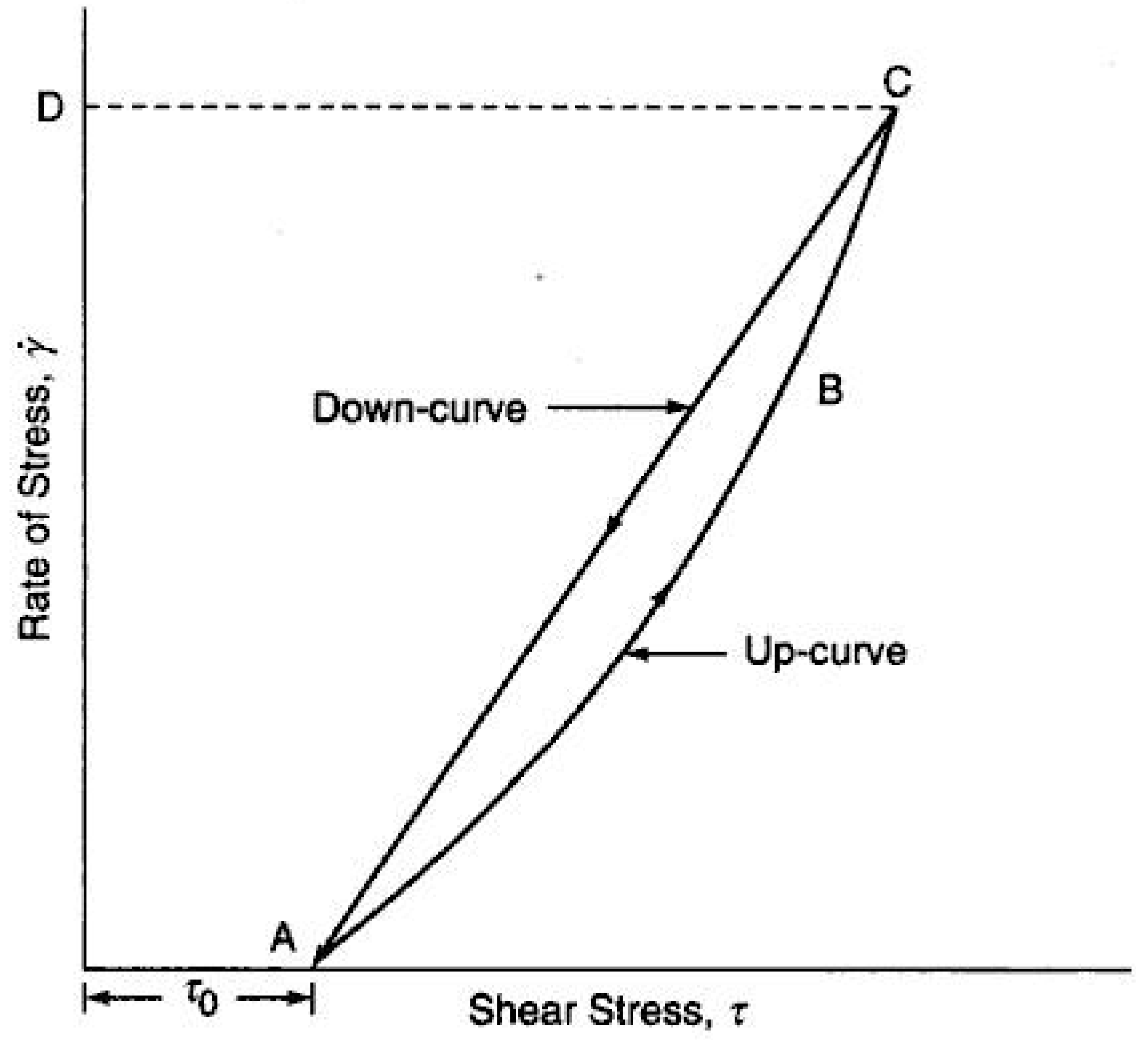

Fig. 7.3 Deformation curve for fresh concrete

is obtained as the rate of shear is gradually increased up to *C*, if the rate of shear is now decreased steadily to zero, the down curve may not coincide with the up curve. If the point *C* represents the breakdown in the structure under shear and the structural breakdown process is immediately and instantaneously reversed, the decrease in shear rate from *D* will result in a progressive built up of structure to the same state as it had on the up curve. On the other hand, time is required for the structure to rebuild, thus as the rate of shear is decreased the shear stress at any particular rate of shear on the down curve will be less than the shear at the same rate on the up curve and the two curves will not be coincident and hysteresis loop will form. Provided its limits are recognized, the Bingham model can be applied to fresh concrete under practical circumstances.

7.3 FACTORS AFFECTING RHEOLOGICAL PROPERTIES

Like workability, the rheological properties of fresh concrete are affected by the mix proportions, i.e. the amount of each constituent, properties of the ingredients, the presence of admixtures, the amount of mixing, and the time elapsed after mixing. Though these factors have already been explained in Chapter 6, in this section the factors are discussed in relation to their effects on the flow properties of concrete.

7.3.1 Mix Proportions

The concrete mixture is proportioned to provide the workability needed during construction and to assure that the hardened concrete will have the stipulated performance characteristics.

The *accelerators* or *retarders* will reduce or extend the workability time for a given mixture. The time of retention of improved workability is critical for the uses where concrete is to be transported or manipulated before placement. The reduction in workability is similar to the stiffening of concrete due to slow chemical reaction taking place during the induction period. Since the admixture changes the nature of membrane around the cement particles and the composition of aqueous solution, they change the rate of stiffening and this has been especially noticed with *superplasticizers.* A retarder plasticizer reduces workability loss and lengthens the retention time due to the slowing down of the process of setting. The superplasticizers give large improvements in the workability without retarders that would be needed if conventional plasticizers were used at such high dosages. It is also seen that the original flowing consistency could be retained by adding a second dose after up to 60 minute for melamine resin and up to 150 minute for naphthalene resin. Sometimes superplasticizers and retarders are used in combination to obtain slower stiffening than that obtained when they are used alone.

7.4 MIXTURE ADJUSTMENTS

Proper attention to the rheological properties of a mixture can effectively reduce construction and material costs. The changes in rheological properties of concrete are often detected visually. It is essential to make adjustments as the properties of material and field conditions change.

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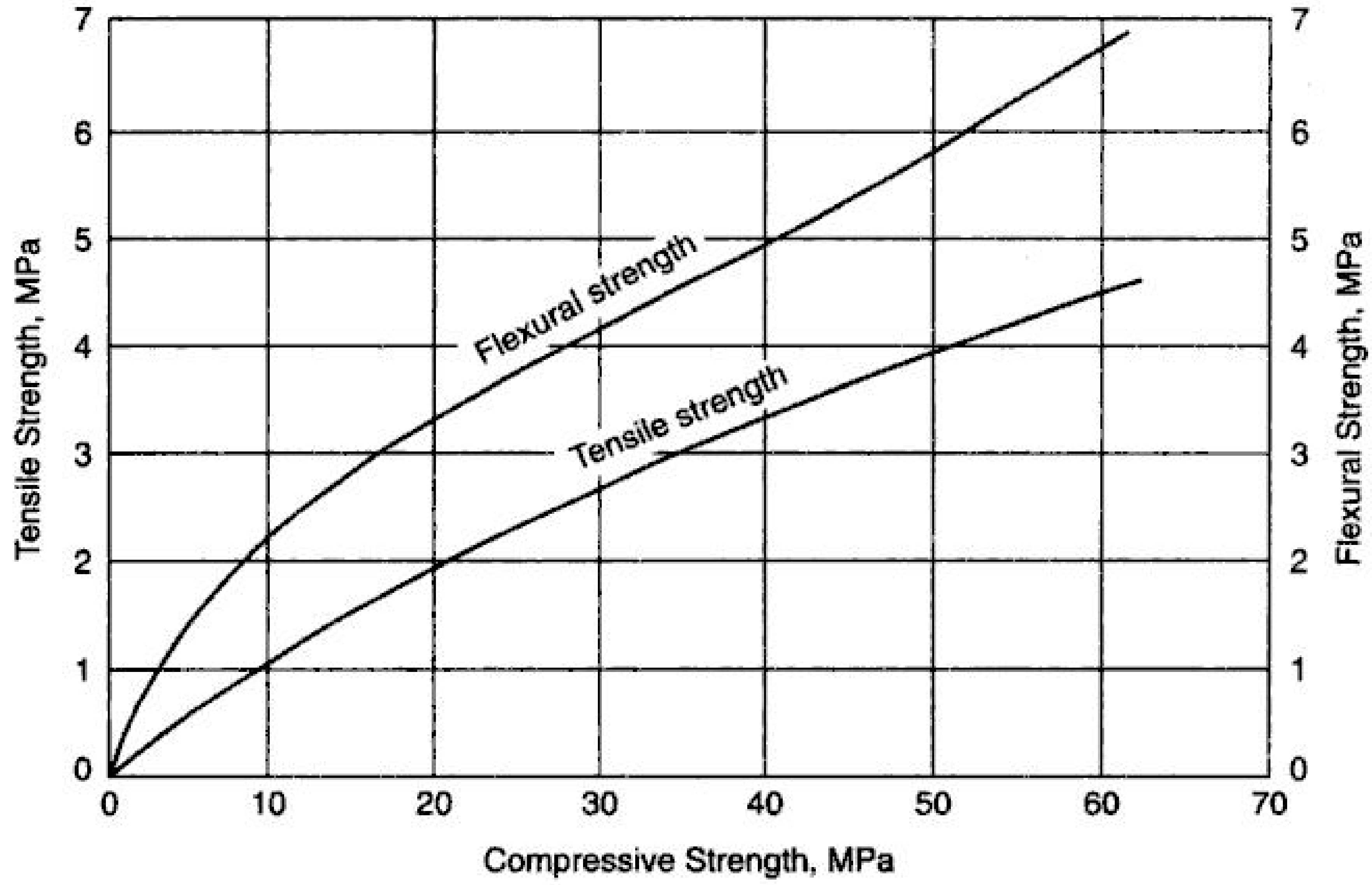

Fig. 8.3 Relationship between compressive strength, tensile strength and flexural strength

in the specimen. The tensile stress at which failure occure is the *tensile strength* of concrete.

The *splitting test* are well-known indirect tests used for determining the tensile strength of concrete, sometimes referred to as the *splitting tensile strength* of concrete. The test consists of applying compressive line loads along the opposite generators of a concrete cylinder placed with its axis horizontal between the plattens as shown in Fig. 8.4. Due to the applied line loading a fairly uniform tensile stress is induced over nearly two-third of the loaded diameter as obtained from an elastic analysis. The magnitude of this tensile stress (acting in a direction perpendicular to the line of action of applied compression) is given by $2P/\pi DL = 0.637P/DL$, where P is the applied load, and D and L are the diameter and length of the cylinder, respectively. Due to this tensile stress, the specimen fails finally by splitting along the loaded diameter and knowing P at failure, the tensile strength can be determined. The test can also be performed on cubes by splitting either: (i) along its middle parallel to the edges by applying two opposite compressive forces through 15 mm square bars of sufficient length, or (ii) along one of the diagonal planes by applying compressive forces along two opposite edges as shown in Fig. 8.5. In the case of side-splitting of the cubes, the tensile strength is determined from $0.642\ P/S^2$ and in diagonal splitting it is determined from $0.5187\ P/S^2$, where P is the load at failure and S is the side of the cube.

The relationships between compressive strength and split tensile strength; and flexural strength and split tensile strength are given in the Figs 8.6 and 8.7, respectively.

(ii) at $f = f_0$, $\epsilon = \epsilon_0$ and $\frac{df}{d\epsilon} = 0$

(iii) at $f = f_f$, $\epsilon = \epsilon_u$

The equation satisfying all these conditions is used in the limit state design method. In another case, some simplifying assumptions are made. One of the major assumptions is made in approximating the stress–strain curve to a straight line, i.e. treating the concrete as linearly elastic material. This approximation is used in *working stress method* of design of structural concrete without much loss of accuracy up to about 50 per cent of f_0. Concrete is not strictly elastic in the sense that if it is unloaded after being stressed to 0.5 f_0 or less, a *permanent set* is noticed (Fig. 8.14). However, the magnitude of the permanent set gradually decreases with more cycles of loading and unloading (within 0.5 f_0) and the stress–strain curve tend to become a straight line. The *creep deformation* of concrete also varies linearly with the sustained stress up to a value of 0.5 f_0. Hence, for all practical purposes, the concrete could be considered as a linear elastic material when stress does not exceed 0.5 f_0.

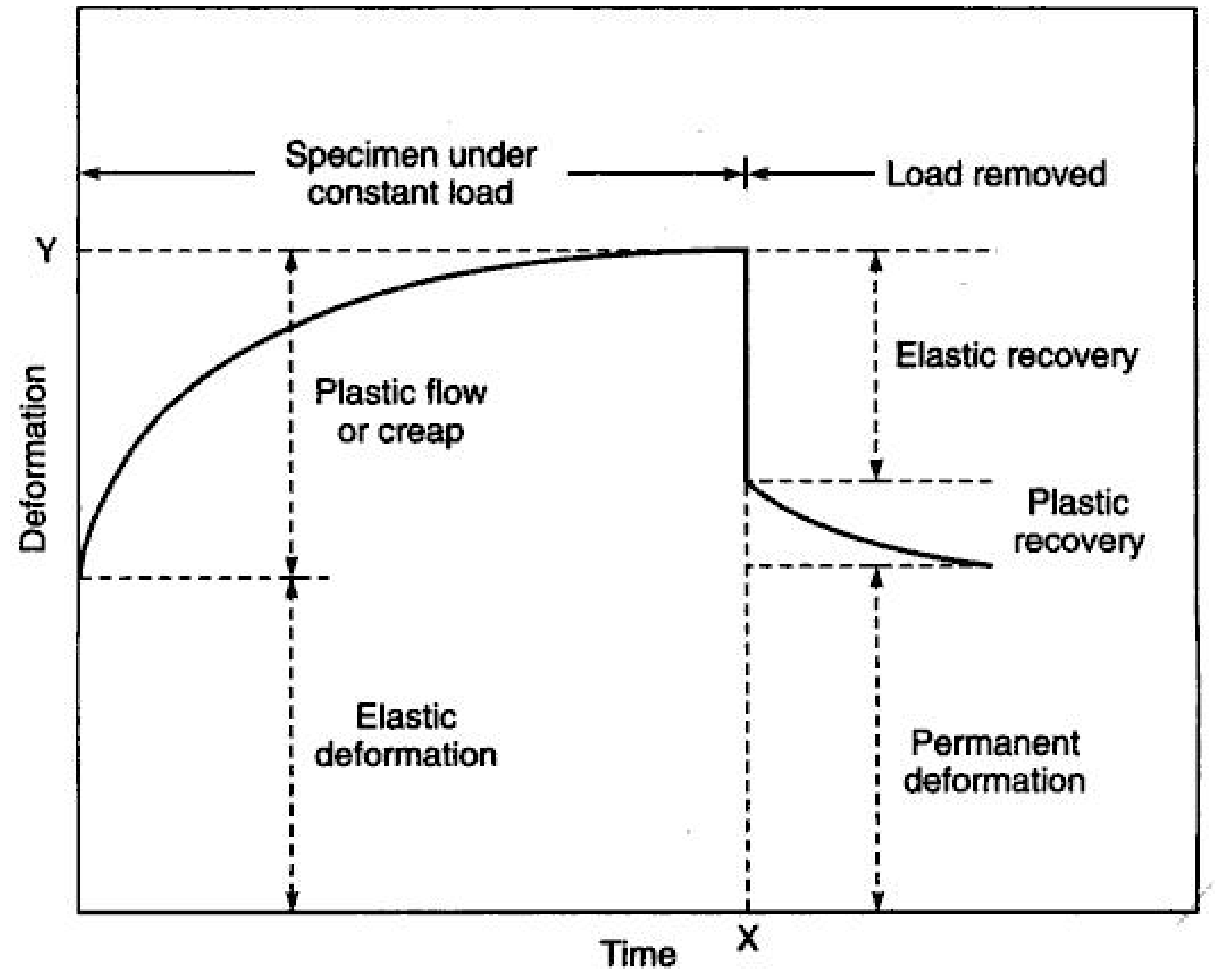

Fig. 8.14 Deformation of hardened concrete under load

8.3.1 Modulus of Elasticity

The *modulus of elasticity* of concrete would be a property for the case when the material is treated as elastic. If we consider the stress–strain curve of the first cycle, the modulus could be defined as *initial tangent modulus, secant modulus, tangent modulus* or *chord modulus,* as shown in Fig. 8.15. In the laboratory

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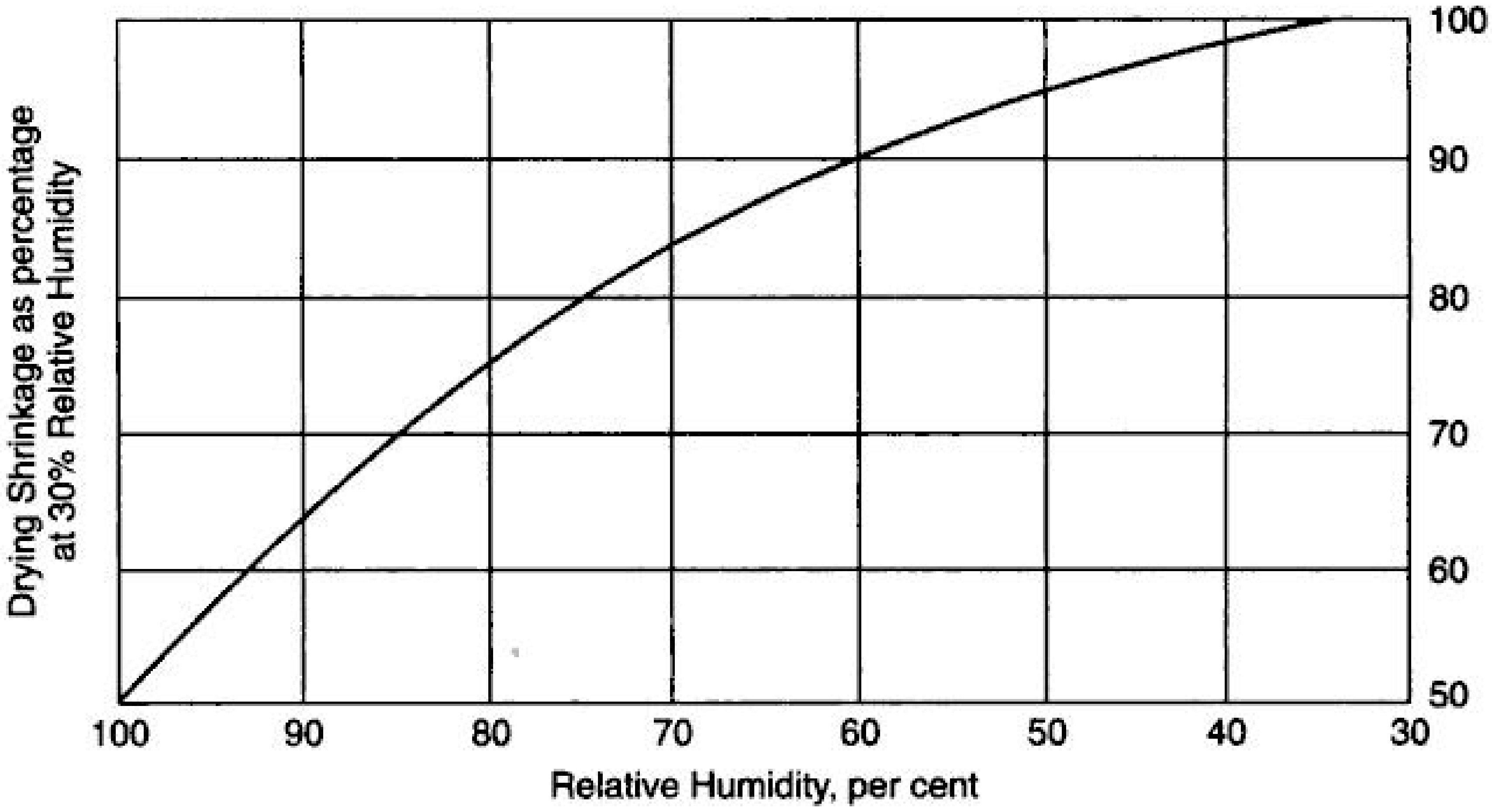

Fig. 8.18 Effect of relative humidity on drying shrinkage of concrete

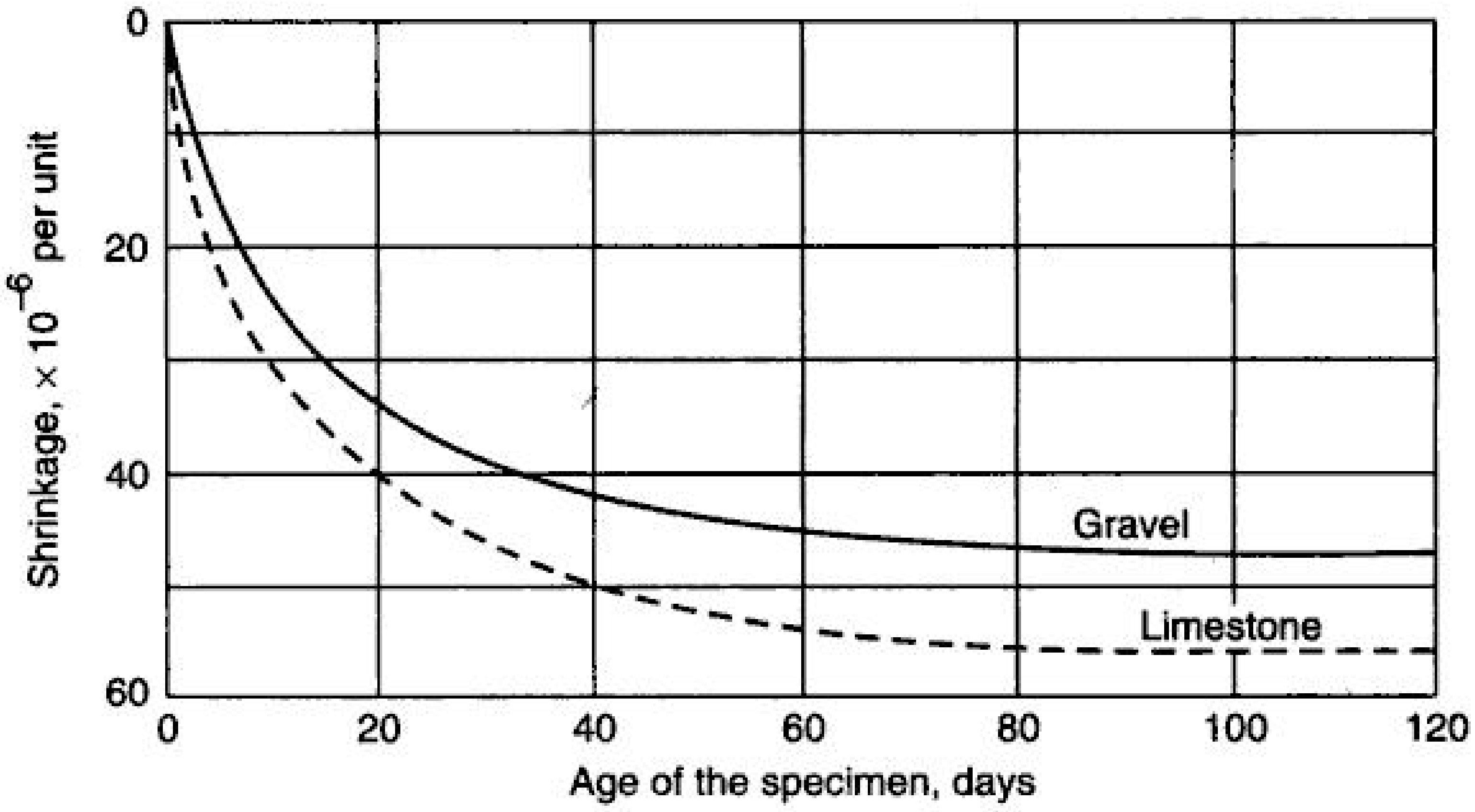

Fig. 8.19 Effect of the type of aggregate on the drying shrinkage of concrete

The *carbonation* penetrates beyond the exposed surface of concrete only slowly. Carbonation is accompanied by increase in weight and shrinkage. The shrinkage due to carbonation occurs mainly at intermediate humidities. Carbonation also results in increased strength and reduced *permeability*.

When the shrinkage is restrained partly or fully due to internal (i.e. aggregate or reinforcement) or external restraints, tensile stresses leading to *cracking* develop in concrete. When the restraints are eccentric, warping could occur leading to shrinkage deflection of member. Suitable joints may be provided to accommodate contraction or expansion movements. The only advantage of shrinkage is that it causes the concrete to grip the steel tightly, thus increasing the bond.

mechanical properties of the concrete with temperature is shown in Fig. 8.20. The values of coefficient of thermal expansion of concrete prepared with different types of aggregates are given in the Table 8.5.

Table 8.5 *Coefficient of Thermal Expansion of Concrete*

Type of aggregate	α_c/°C	*Type of aggregate*	α_c/°C
Quartz	11.9×10^{-6}	Granite	9.5×10^{-6}
Sandstone	11.7×10^{-6}	Basalt	8.6×10^{-6}
Gravel	10.8×10^{-6}	Limestone	6.8×10^{-6}

8.12 THERMAL PROPERTIES OF CONCRETE

The important thermal properties required for the design of structures are thermal conductivity, thermal diffusivity, specific heat, and coefficient of thermal expansion.

Thermal conductivity is a measure of the ability of the concrete to conduct heat and is measured in British Thermal Units per hour per square foot area of the body when the temperature difference is 1°F per foot thickness of the body. Thermal conductivity depends upon the composition of concrete. The structural concrete containing normal aggregate conducts heat more readily than light-weight concrete. Lower the *water–content* of the mix, the higher the conductivity of the hardened concrete. The density of the concrete does not appreciably affect the conductivity of ordinary concrete. The variation of thermal conductivity of concrete with temperature is shown in Fig. 8.21.

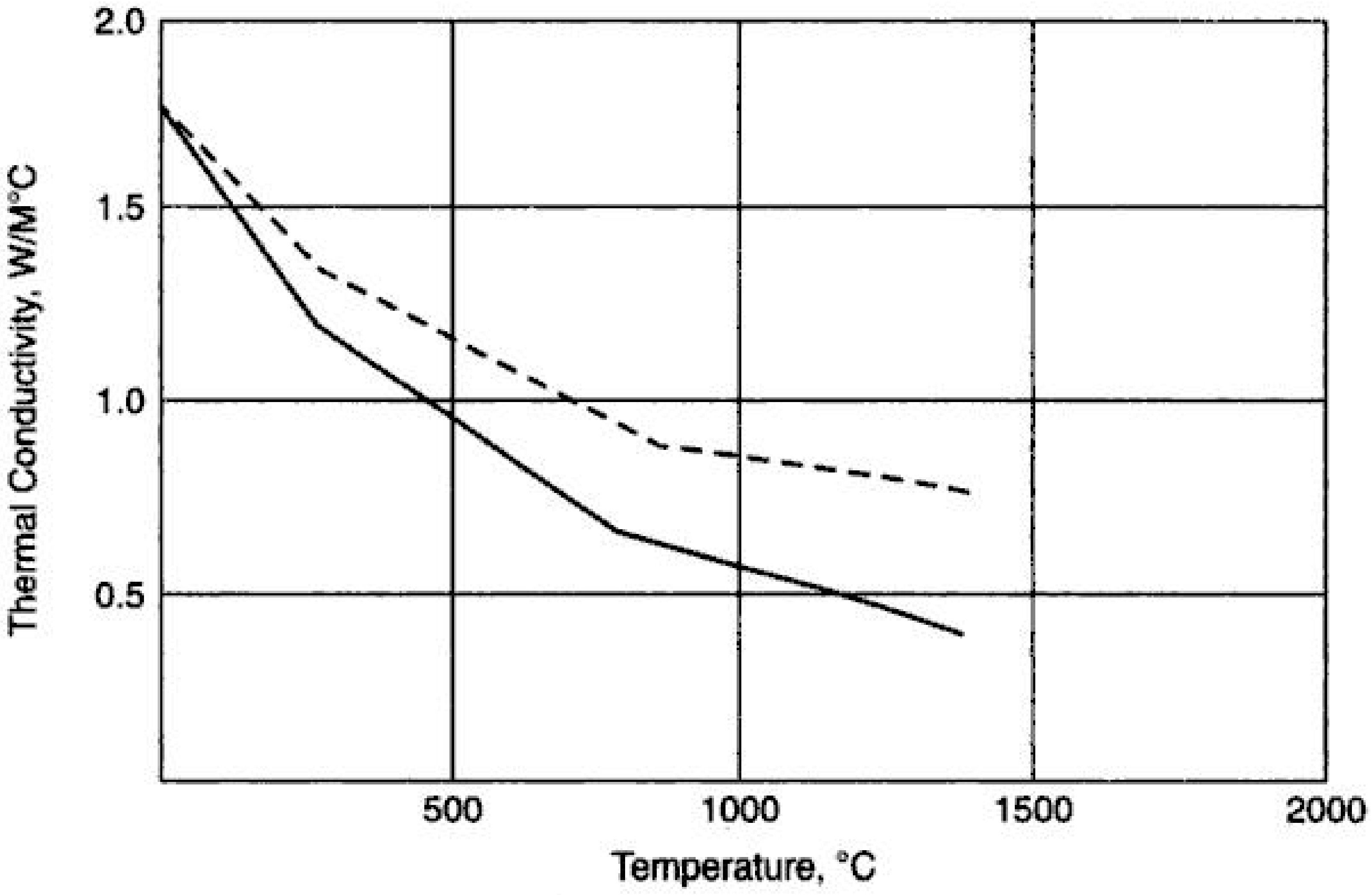

Fig. 8.21 Thermal conductivity of concrete

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specifications and *control requirements.* The specifications have to state clearly and explicitly the steps and requirements, adherence to which would result in a construction of acceptable quality. Except for compressive strength and appearance there is no early measure of construction performance. Each step in construction procedure is therefore to be specified. The probability based specifications containing allowable tolerances on its attributes is more rational and is preferred. *Quality control is thus conformity to the specifications, no more no less.* The most practical method of effective quality control is to check what is done in totality to conform to the specifications. An owner will have no right to expect anything more than what is in the specifications. The builder, on the other hand, knows that anything less than what is in the specifications will not be acceptable to the owner.

In view of the different processes involved in the manufacture of concrete, the problems of quality control are diversified and their solution elaborated. The factors involved are the personnel, the materials and equipment, the workmanship in all stages of concreting, i.e. batching of materials, mixing, transportation, placing, compaction, curing, and finally testing and inspection. It is therefore necessary to analyse the different factors causing variations in the quality and the manner in which they can be controlled.

9.2 FACTORS CAUSING VARIATIONS IN THE QUALITY OF CONCRETE

The main factors causing variation in concrete quality are as follows.

Personnel

The basic requirement for the success of any quality control plan is the availability of experienced, knowledgeable and trained personnel at all levels. The designer and the specification-writer should have the knowledge of construction operations as well. The site engineer should be able to comprehend the specification stipulations. Everything in quality control cannot be codified or specified and much depends upon the attitude and orientation of people involved. In fact, quality must be a discipline imbibed in the mind and there should be strong motivation to do every thing right the first time.

Material, Equipment and Workmanship

For uniform quality of concrete, the ingredients (particularly the cement) should preferably be used from a single source. When ingredients from different sources are used, the strength and other characteristics of the materials are likely to change and, therefore, they should only be used after proper evaluation and testing. The same type of cement from different sources and at different times from the same source exhibit variations in properties, especially in compressive strength. This variation in the strength of cement is related to the composition of raw materials as well as variations in the manufacturing process. The cement

are said to follow a normal distribution as shown in Fig. 9.1, if they are equally spaced about the mean value. However, some divergence from the smooth curve is only to be expected, particularly if the number of results available is relatively small. The normal distribution curve can be used to ascertain the variation of strength from the mean. The area beneath the curve represents the total number of test results. The proportion of results less than the specified value is represented by the area beneath the curve to the left-hand side of the vertical line drawn through the specified value.

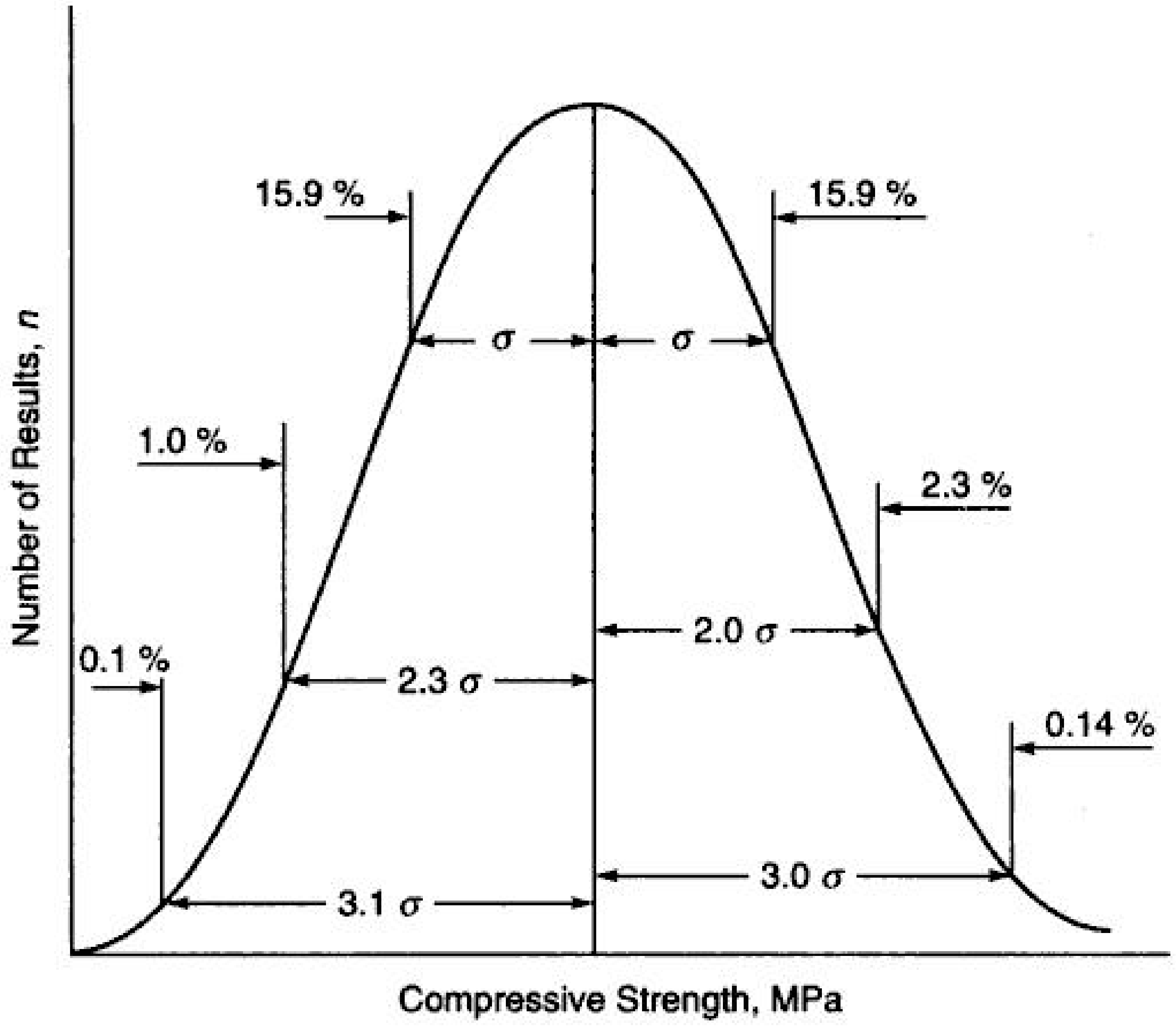

Fig. 9.1 Normal distribution of compressive strength results

A *normal distribution* curve can be defined by two parameters, namely, the *mean strength* and the *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 computing the standard deviation is given in IS: 456–2000 as explained section 9.63.

Figure 9.2 shows the frequency density versus compressive strength distribution curves of data population of the concrete mixes A and B. The distribution curves follow the normal distribution pattern. The curves are symmetrical about the mean value. Mix B indicates better quality control than that obtained for the mix A although both the mixes have the same average strength. Thus by exercising a better quality control, the standard deviation of the mix can be reduced by giving a lower probability of failure or a higher degree of reliability.

S = standard deviation.

The value of k where not more than 5 per cent (1 in 20) of test results are expected to fall below characteristic strength is 1.65 as obtained from Fig. 9.3 or Table 9.1 and the above relation reduces to

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

However, it should be noted that for a given degree of control, the standard deviation method yields higher target mean strengths than the coefficient of variation method for low-strength and medium-strength concretes. For high-strength concrete, the coefficient of variation method yields higher values of target mean strength. The cost of production being dependent on the target mean strength of concrete, the method of evaluation should be consistent with the observed trend of results for different ranges of strength. However, the use of the coefficient of variation is not envisaged in IS: 546–2000.

To keep a control on the quality of concrete produced, it is required to cast a number of specimens from random samples and test them at suitable intervals to obtain results as quickly as possible to enable the level of control to be established with reasonable accuracy in a short time. IS: 450–2000 stipulates that *random samples* from fresh concrete shall be taken as specified in IS: 1199–1959 and the cubes shall be made, cured and tested at 28 days as described in IS: 516–1959. The test result of a sample be the average of the strength of three specimens (constituting the sample). The individual variation should not be more than ±15 per cent of the average. If it is more, the test result of the sample is invalid. The *random sampling* procedure is adopted to ensure that each concrete batch shall have a reasonable chance of being tested, i.e. the sampling should be spread over the entire period of concreting and cover all mixing units. The code prescribes minimum *frequency of sampling* of 1, 2, 3 and 4 number of samples, respectively, for 1–5, 6–15, 16–30 and 31–50 m^3 of concrete being used in the job. For concrete quantity of 51 m^3 and above, the number of samples shall be 4 plus one additional sample for every 50 m^3 of concrete or part thereof. At least one sample should be taken from each shift. In case of continuous production unit, e.g. *ready mixed concrete plant,* the frequency of sampling may be as per agreement. Additional samples may be required for various purposes, e.g. for determination of 7 days strength, accelerated strength, time of striking the formwork, etc. As far as the requirements of specifications with regard to the acceptance criteria for concrete is concerned, IS: 456–2000 stipulates that the concrete shall be deemed to satisfy the strength requirements provided the mean strength f_m of any group of four non-overlapping consecutive test samples satisfies the following.

(i) For M15 grade concrete

$f_m \geq f_{ck}$ + 0.825 times the standard deviation or

f_{ck} + 3 MPa, whichever is greater

(ii) for group test results

(b) $f_{ck} + 0.825\ S = 32.48$ MPa

(c) $f_{ck} + 4.00 = 34.00$ MPa

In the light of above strength requirements, it may be noted that:

(i) The individual test values of all samples except the 16th sample are greater than 26.0. The 4th group containing this sample is straightaway unacceptable.

(ii) The concretes pertaining to the Groups 1, 2 and 3 do not comply with the strength requirements stipulated in (b) and (c) above, hence are unacceptable.

(iii) The concrete produced during sampling of Group 5 is straightaway accepted as it satisfies both the strength requirements specified in (b) and (c).

(iv) The concrete represented by the Group 6 does not satisfy the strength requirement specified in (c) above but has strength greater than the requirement specified in (b) and hence may be accepted as being structurally adequate without further testing at the discretion of the designer.

9.8 QUALITY MANAGEMENT IN CONCRETE CONSTRUCTION

As explained earlier the quality, meant to measure the degree of excellence, does in fact measure the degree of fulfilment. *The quality is thus a philosophy rather than a mere attribute.* It is from this philosophy the distinctive culture emanates, guiding the society to attain targets set by it. The presence or absence of this culture makes all the difference which determines the level of acceptability. The constant awareness of this culture amongst other endowments have led many nations where they exist today.

In the industrial climate particularly in manufacturing and process industry, the concept of quality management is age old and is extensively used, whereas it is recent in concrete construction industry. Every piece of equipment or product is subjected to quality management in the industrial production as a matter of routine. The quality management ensures that every piece of product keeps on performing over a period of time without heavy maintenance and upkeep. Fortunately in concrete construction even if rigid quality management measures are not followed, it performs, at least for reasonable period of time. On account of this co-operative property of the material, the *concrete construction industry* has been operating under the misconception that rigid quality management measures which are essential for an industrial product are not that essential for concrete. Thus in concrete industry of most of the developing countries, in spite of best efforts a great deal is yet to be achieved to derive maximum benefit out of this culture. Measures have been devised to enhance serviceable, maintenance and rehabilitation free life of the material and minimize,

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QC is the responsibility of the contracting organization. The contracting organization is also responsible for QC activities related to its sub-contractors. Quality control starts with the construction. The constructing organization prepares the QA programme manual describing and establishing the QA and control system to be used by it in performing design, purchasing, fabrication, production of concrete and other construction activities for the contractual responsibilities assigned to it. Application area, indentification of agencies and personnel responsible for implementing, managing and documenting the QC programmes, their responsibilities and authorities must be well established in the document. The detailed steps in these procedures depend upon the scope and type of work and owner's policy decision.

Quality Audit (QA)

This is a system of tracking and documentation of Quality Assurance and Quality Control programmes. Quality Audit is the responsibility of the owner, and has to be performed at regular intervals through the tenure of the project. Quality Audit covers both the design as well as the construction phases. Thus the concept of Quality Management encompasses a total project and each element of that project. The systems on methodology of implementing concept of Quality Management depend on the available materials and construction technology. As the concrete technology changes, these systems also change. As such the systems of implementing concepts of Quality Management are not universal but regional and not static but dynamic, and ever changing.

An integrated systematic implementation of QMS is extremely beneficial, but any attempt to make its piecemeal use will defeat the very purpose for which it is intended. In other words in order to produce a safe, reliable and durable structure, Quality Culture must begin at the beginning and be carried through all the stages of design, procurement, construction and be continued further into the in-service regime. It is only a matter of systematic cultivation and a desire towards increased perfection that can make a complete metamorphosis of a developing construction industry.

9.8.3 Cost Effectiveness of Quality Management

It has been the general experience that whether it is the owner who has to cover the cost of *Quality Assurance, Quality Audit* and *Peer Review* or the contractor who has to cover the cost of *Quality Control*, the expenditure is met out of savings which accrue from the project due to implementation of *Quality Management Systems* (QMS). On the part of owner the Quality Management ensures a product of assured quality, strength, reliability and maintenance free durable life cycle. This is achieved by eliminating chances of mistakes in planning, overdesign or underdesign and ensures proper detailing and constructability. Any of these items if overlooked can later cost heavily to the owner. It is universally accepted that every project has a *Quality Cost Component.* Every contractor has

of plastic concretes of medium strengths can be based on the following assumptions.

(i) The *compressive strength* of concrete is governed by its *water–cement ratio.*
(ii) For the given *aggregate* characteristics, the *workability* of concrete is governed by its *water content.*

For *high-strength* or *high performance concrete* mixes of *low workability,* considerable interaction occurs between the above two criteria and the validity of such assumptions may become limited. Moreover, there are various factors which affect the properties of concrete, e.g. the quality and quantity of cement, water and aggregates; techniques used for batching, mixing, placing, compaction and curing, etc. Therefore, the specific relationships used in the proportioning of a concrete mix should be considered only as a basis for making an *initial guess at the optimum combination of the ingredients* and the *final mix proportion* is obtained only on the basis of further *trial mixes.*

10.3 FACTORS INFLUENCING THE CHOICE OF MIX PROPORTIONS

According to IS: 456–2000 and IS: 1343–1980, the design of concrete mix should be based on the following factors.

(i) Grade designation
(ii) Type and grade of cement
(iii) Maximum nominal size of aggregates
(iv) Grading of combined aggregates
(v) Water–cement ratio
(vi) Workability
(vii) Durability
(viii) Quality control

10.3.1 Grade Designation

The *grade designation* gives *characteristic compressive strength* requirements of the concrete. As per IS: 456–2000, the *characteristic compressive strength* is defined as that value below which not more than 5 per cent of the test results are expected to fall. It is the major factor influencing the mix design. Depending upon the *degree of control* available at the site, the concrete mix has to be designed for *a target mean compressive strength* which is somewhat higher than the characteristic strength.

10.3.2 Type and Grade of Cement

The type of cement is important mainly through its influence on the rate of development of compressive strength of concrete. The choice of type of cement

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Table 10.2 *Combining Fine and Coarse Aggregates to a Stipulated Grading*

IS sieve	*Percentage passing*			
	Fine aggregate	*Coarse aggregate*	*Specified grading*	*Combined grading*
(a)	(b)	(c)	(d)	(e)
40 mm	100	100	100	100
20 mm	100	98	100	99
10 mm	100	43	65	67
4.75 mm	96	03	42	42
2.36 mm	89	0	35	37
1.18 mm	73	0	28	31
600 µm	48	0	20	20
300 µm	20	0	07	08
150 µm	02	0	0	01

column d, it can be noted that the percentage passing IS: 4.75 mm sieve is same and combined grading is close to the desired grading.

In the above problem there is only one point on the grading curve to which the aggregate is required to approximate. Comparing the grading of resulting combined curve with the selected standard grading curve, the percentage passing the *criterion sieve* necessarily agree but the other values may not. In some cases variation is very small which may be ignored. If, however, the discrepancies are large, the proportions may be changed by adopting another criterion point. It should be realized that mix proportioning is approximate, and it is extremely doubtful that the result would be better if the grading is further made closer.

The method can also be applied if three or more aggregates are to be combined. Example 10.2 will illustrate the procedure.

Example 10.2 The gradings of fine and two coarse aggregates available at a project site are listed in Columns b, c and d, respectively, of Table 10.3. These aggregates are to be combined so as to approximate the grading listed in column e of Table 10.3.

Solution: It is required to determine fractions x and y of the two coarse aggregates to be combined with unit weight of fine aggregate so as to obtain the specified grading. Two unknowns need two equations for solution. Let the criterion sieve sizes be 10 mm and 2.36 mm.

According to the specified grading, the combined aggregate passing the IS: 10 mm sieve is 45 per cent, hence using Table 10.3.

$$100(1) + 94x + 18y = 45(1 + x + y)$$

or

$$49x - 27y = 55$$

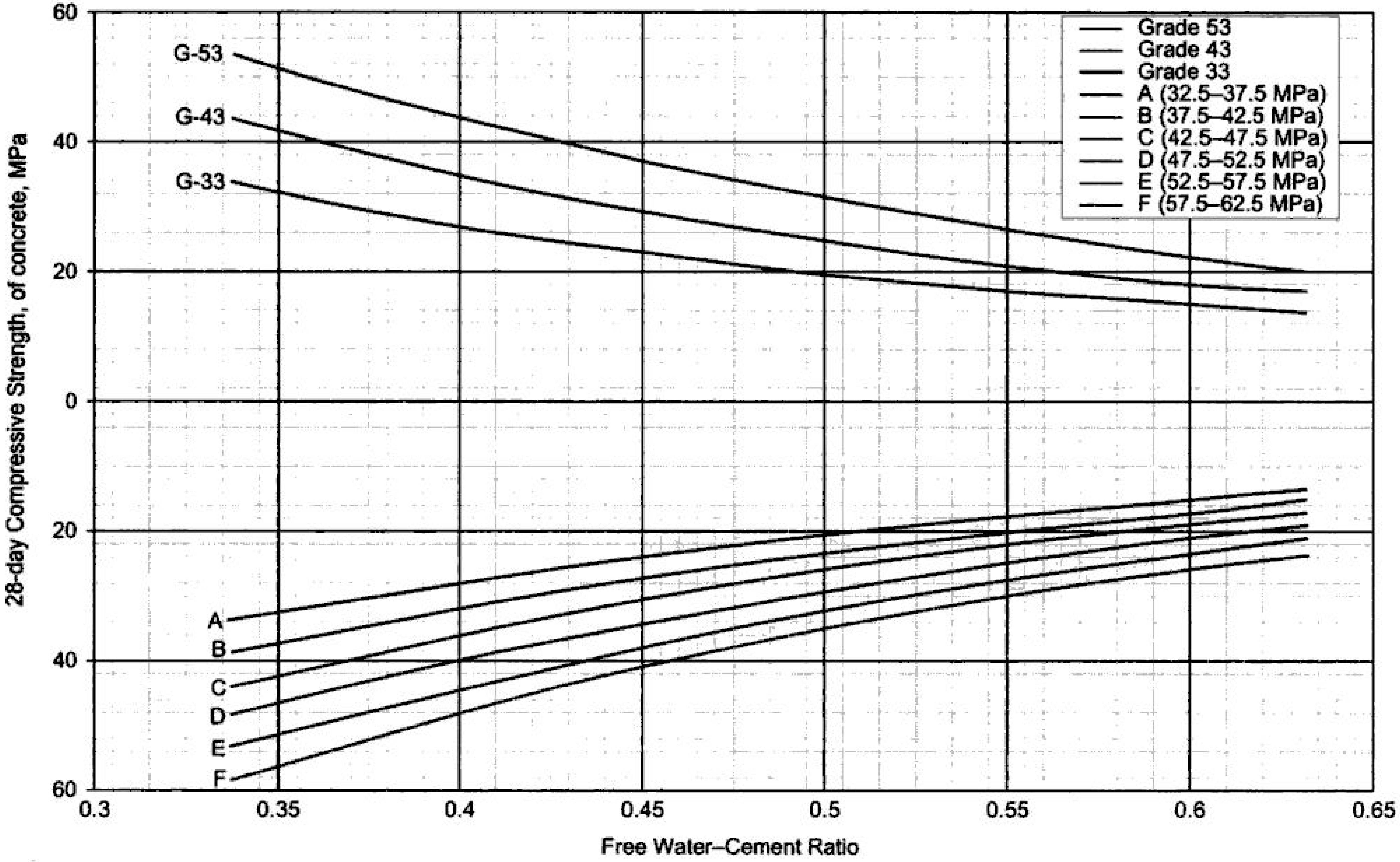

Fig. 10.3 Relation between free water–cement ratio and concrete strength at 28 days for different cement strengths (IS: 10262–1982)

Table 10.6 Minimum Cement Content (kg/m³), Maximum Water–cement Ratio and Grade of Concrete to Ensure Durability under Specified Exposure with Aggregate of 20 mm Nominal Maximum Size

General environment	*Exposure conditions*	*Plain concrete*			*Reinforced concrete*		
		Minimum cement content, kg/m³	*Maximum free water–cement ratio*	*Minimum grade of concrete*	*Minimum cement content, kg/m³*	*Maximum free water–cement ratio*	*Minimum grade of concrete*
Mild	Concrete surfaces protected against weather or aggressive conditions, except those situated in coastal area.	220	0.60	–	300	0.55	M20
Moderate	Concrete surfaces sheltered from severe rain or freezing whilst wet. Concrete exposed to condensation and rain. Concrete continuously under water. Concrete in contact or buried under non-aggressive soil/ground water. Concrete surfaces sheltered from saturated salt air in coastal area.	240	0.60	M15	300	0.50	M25
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation. Concrete completely immersed in sea water Concrete exposed to coastal environment.	250	0.50	M20	320	0.45	M30

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(ix) The final proportions are expressed either on mass or volume basis.

Most of the available mix design methods are essentially based on the above procedure and due consideration should be given for the *moisture content of aggregate* and the *entrained air.*

10.4.1 Trial and Adjustment Method of Mix Design

The trial and adjustment method is based on experimental approach and aims at producing a concrete mix which has *minimum voids* and hence, *maximum density.* The fine aggregate is mixed in sufficient quantity to fill the voids in the coarse aggregate; and cement paste is used in sufficient quantity to fill the voids in the mixed aggregate. The proportion of fine to coarse aggregate which gives maximum mass of combined aggregate can be obtained by trials. The process consists of filling a container of known volume with the two materials in thin layers, the fine being placed over the coarse aggregate and lightly rammed after each layer. If the container is shaken too much, the coarse aggregate will try to come on the top and the fine aggregate will deposit at the bottom without filling the voids of the coarse aggregate. Since the density of the particles of fine and coarse aggregates is nearly the same, the mixture giving maximum weight will have maximum solid matter and hence *least voids.* Such a combination will need the least amount of cement per cubic metre of concrete and will be most economical for a given *water–cement ratio* and *slump.*

In an alternate trial mix method, sand is combined with the coarse aggregate in several proportions, such as 20:80, 30:70, 40:60, 50:50 and 60:40, and for each such mixture, the quantity of cement paste of a certain *water–cement ratio* per unit volume of concrete is determined to give the required *workability* (expressed in terms of slump). The percentage of sand corresponding to the ratio requiring minimum cement, is termed *optimum percentage.* If the quantity of sand used is more than the optimum, more cement will be needed to have the same consistency. On the other hand, a smaller quantity of sand will make the mix harsh unless more cement is used for proper consistency. The optimum percentage of sand is lower for a low water–cement ratio. The step-by-step procedure of mix proportioning is as follows.

(i) The target mean compressive strength is determined from the characteristic strength.

(ii) The water–cement ratio is chosen for the target mean strength computed in Step (i). The water–cement ratio so chosen is checked against limiting water– cement ratio for the requirements of durability and the lower of the two values is adopted.

(iii) The workability is determined in terms of the slump required for a particular job.

(iv) The maximum nominal size of the coarse aggregate that is available or desired to be used, is determined.

(iv) Determination of Aggregate–Cement Ratio

The total *aggregate content* (in saturated surface dry condition) is obtained by subtracting the cement and water contents from the wet density of concrete obtained from Fig. 10.7 which depends on the *specific gravity of combined aggregates* (in the saturated surface dry condition). Alternatively, it can also be calculated easily from Eq. (10.6).

$$\text{Absolute volume occupied by the aggregate} = 1 - \frac{\text{cement content (kg)}}{1000\,S_c} - \frac{\text{water content (kg)}}{1000} \quad (10.6)$$

where S_c is the specific gravity of cement particles.

Therefore,

Total aggregate content (kg/m^3) = 1000 S_a × (absolute volume occupied by the aggregates)

where S_a is the specific gravity of aggregate particles. If no information is available S_a should be taken as 2.6 for uncrushed aggregate and 2.7 for crushed aggregate i.e curves A and B and can be used.

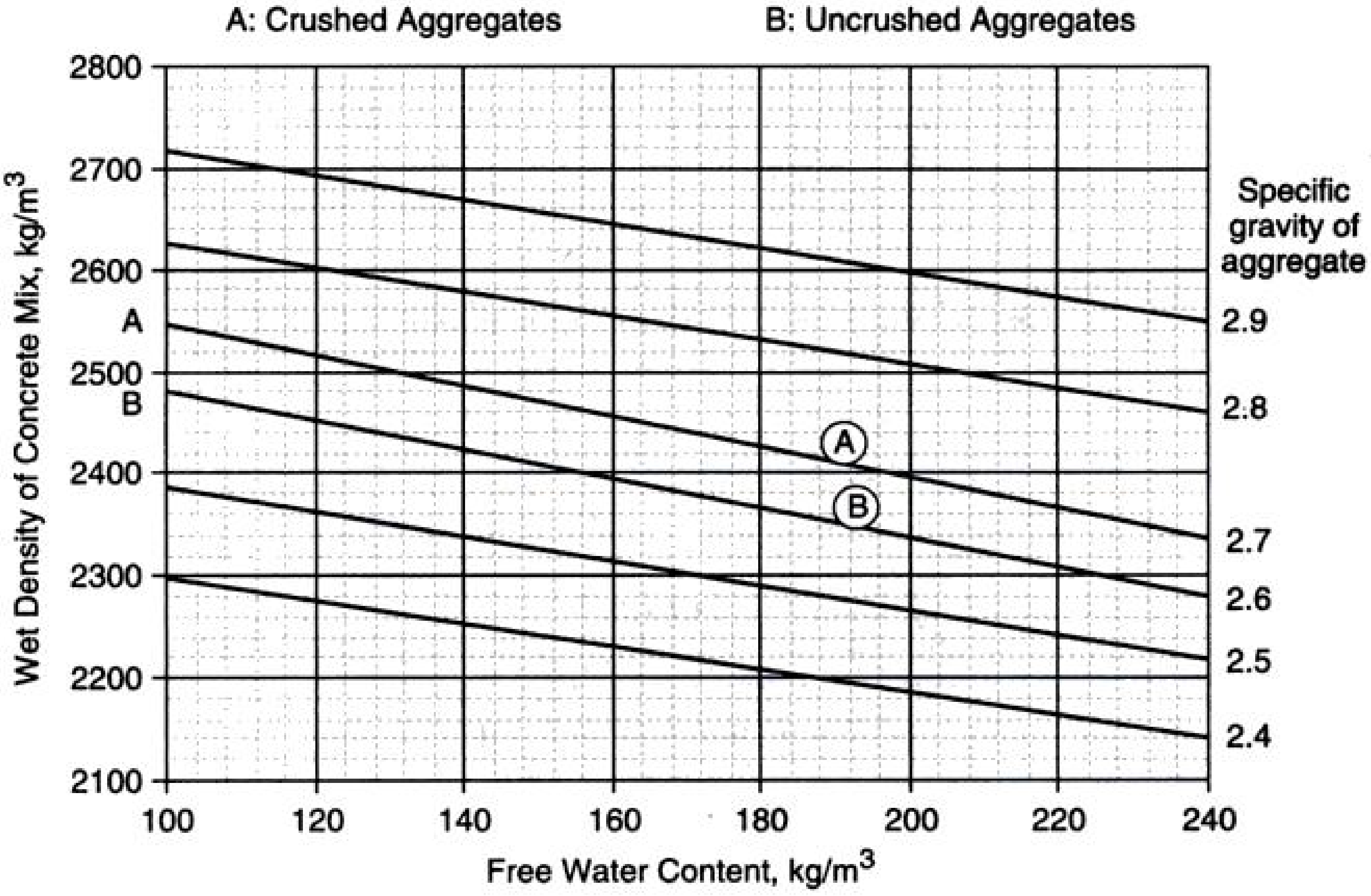

Fig. 10.7 Estimated wet density of fully compacted concrete (DoE)

(v) Determination of the Fine and Coarse Aggregate Contents

Depending on the water–cement ratio, the nominal maximum size of coarse aggregate, the workability and the grading zone of fine aggregate, the proportion of fine aggregate is determined from Fig. 10.8. Having calculated the proportion

3. For the uncrushed aggregate of maximum nominal size of 10 mm, the water content to give a slump of 10–30 mm as obtained from Table 10.8 is 180 kg/m^3.
4. For the free water–cement ratio of 0.48,

   Cement content = 180/0.48 = 375 kg/m^3

   This is satisfactory as it is greater than specified minimum of 300 kg/m^3 and less than the permitted maximum value of 450 kg/m^3.
5. From Fig. 10.7, wet density of concrete is 2400 kg/m^3.
6. Total aggregate content $= 2400 \times 1 \left[1 - \frac{375}{3150} - \frac{180}{1000}\right] = 1682$ kg/m^3.
7. From Fig. 10.8 (a.2), for a slump of 10–30 mm, a water–cement ratio of 0.48, and percentage of fine aggregate passing 600 micron seive being 70, i.e. fine aggregate belonging to grade zone III, the proportion of fine aggregate is 35 per cent. Therefore, proportions of saturated surface dry aggregate are:

   Fine aggregate content = 0.35 × 1682 = 589 kg/m^3

   Coarse aggregate content = (1 − 0.35) × 1682 = 1093 kg/m^3

   The required mix proportions are:

cement content	375 kg/m^3
water content	180 kg/m^3
fine aggregate content	589 kg/m^3
coarse aggregate content	1093 kg/m^3

   The mix ratio by mass may be expressed as:

Water		*Cement*		*Fine Aggregate*		*Coarse Aggregate*
180	:	375	:	589	:	1093 (kg/m^3)
0.48	:	1.0	:	1.57	:	2.91

   The final proportions shall be established by trial batches and site adjustments.
8. The weights of ingredients for predetermined quantity of a trial mix to be batched in an oven-dry condition can be obtained by multiplying the weights of saturated-surface-dry aggregates by 100/(100 + w) where w is the percentage of water (by mass) required to bring the dry aggregates to a saturated surface dry condition. The quantities of ingredients for a trial mix of (say) 0.05 m^3 can be computed as follows. If the absorption of fine and coarse aggregates are 2 and 1 per cent, respectively, then:

   mass of oven-dry fine aggregate = (0.05 × 589) × 100/102 = 28.87 kg

   mass of oven-dry coarse aggregate = (0.05 × 1093) × 100/101 = 54.11 kg

   amount of water absorbed = (29.45 − 28.87) + (54.65 − 54.11) = 1.12 kg

   Normally 10 per cent additional quantities are taken to account for any underestimation and wastage. Thus, the quantities for the trial batch are: cement: 20.63 kg, water: 11.13 kg; fine aggregate: 31.76 kg (oven-dry) and coarse aggregate: 59.52 (oven-dry).

Table 10.11 *Approximate Water Requirements for Different Slumps and Maximum Size Coarse Aggregate (ACI Manual of Concrete Practice, Part-I)*

Slump, mm	*Compacting factor*	*Relative water content, per cent*	*Mixing water (kg/m³ of concrete)*					
			Maximum sizes of aggregate, mm					
			10	12.5	20	25	37.5	50
Non-air entrained concrete								
Stiff–plastic: 25–50	0.85	92	207	199	190	179	166	154
Plastic: 75–100	0.91	100	228	216	205	193	181	169
Flowing: 150–175	0.95	106	243	228	216	202	190	178
Approximate mean entrained air, per cent			3.0	2.5	2.0	1.5	1.0	0.5
Air-entrained concrete								
Stiff-plastic: 25–50	0.85	92	182	177	162	152	143	139
Plastic: 75–100	0.91	100	203	192	177	168	158	145
Flowing: 150–175	0.95	106	212	203	188	177	168	155
Recommended average total air content, per cent	Mild exposure		4.5	4.0	3.5	3.0	2.5	2.0
	Moderate exposure		6.0	5.5	5.0	4.5	4.5	4.0
	Extreme exposure		7.5	7.0	6.0	6.0	5.5	5.0

Table 10.12 *Relationship between Water–cement Ratio and Average Compressive Strength (ACI Manual of Concrete Practice, Part I)*

Compressive Strength at 28 days, MPa	*Water–Cement Ratio by Mass*	
	Non-air-entrained concrete	*Air-entrained concrete*
45	0.38	—
40	0.42	—
35	0.47	0.39
30	0.54	0.45
25	0.61	0.52
20	0.69	0.60
15	0.79	0.71

available fine aggregate has a fineness modulus of 2.60 and a specific gravity of 2.68. The bulk density of coarse aggregate is 1600 kg/m³. The type of cement

Table 10.15 *Adjustment of Values in Water Content and Sand Percentage for other than the Reference Conditions*

Change in conditions stipulated for Table 10.14	*Adjustment required in*	
	Water content	*Per cent sand in total aggregate*
For the sand conforming to grade zones, I, III and IV of Table 4 of IS: 383–1973	0	+1.5 for Zone I –1.5 for Zone III –3.0 for Zone IV
Increase or decrease in the value of compacting factor by 0.1	±3 per cent	0
Each 0.05 increase or decrease in water–cement ratio	0	±1 per cent
For rounded aggregate	–15 kg/m³	–7 per cent

(vi) The cement content is calculated using the selected water–cement ratio and the final water content of the mix obtained after adjustments. The cement content so calculated is compared with minimum cement content from the requirements of durability, and the greater of the two values is used.

(vii) The total aggregate content (saturated surface dry condition) per unit volume of concrete is determined by subtracting the air, cement and water quantities per unit volume of concrete. With the percentage of sand in total aggregate already determined, the coarse and fine aggregate contents per unit volume of concrete are calculated.

If W be the mass (kg) of water, C be the mass (kg) of cement and v be the air content (m^3) per cubic metre of concrete, the absolute volume of total aggregate (V_a) per unit volume of concrete is given by Eq. (10.10).

$$V_a = 1.0 - \left(W + \frac{C}{S_c}\right)\frac{1}{1000} - v \qquad (10.10)$$

where S_c is the specific gravity of cement. For the ratio of fine aggregate to total aggregate by absolute volume of p, the absolute volumes of coarse and fine aggregates per unit volume of concrete may be calculated from Eq. (10.11).

$$V_{fa} = pV_a$$

and

$$V_{ca} = (1 - p)V_a \qquad (10.11)$$

where V_{fa} and V_{ca} are absolute volumes of fine and coarse aggregates (m^3), respectively. The mix proportions by mass (kg) are given by Eq. (10.12):

$$\begin{array}{ccccccc} \textit{Water} & & \textit{Cement} & & \textit{Fine Aggregate} & & \textit{Coarse Aggregate} \\ W & : & C & : & V_{fa}S_{fa}(1000) & : & V_{ca}S_{ca}(1000) \end{array} \qquad (10.12)$$

where S_{fa} and S_{ca} are specific gravities of saturated surface dry fine and coarse aggregates, respectively, in kilograms per litre.

mix proportion of reference concrete is 1:0.81:2.07 with a water–cement ratio of 0.35.

Using the above proportions, 150 mm cube specimens of reference concrete are made and the *accelerated strength* is determined by using accelerated *curing* by the *boiling-water method.* Corresponding to the accelerated strength of the reference concrete, the water–cement ratio for the required target mean strength of normal concrete is determined from Fig. 10.9.

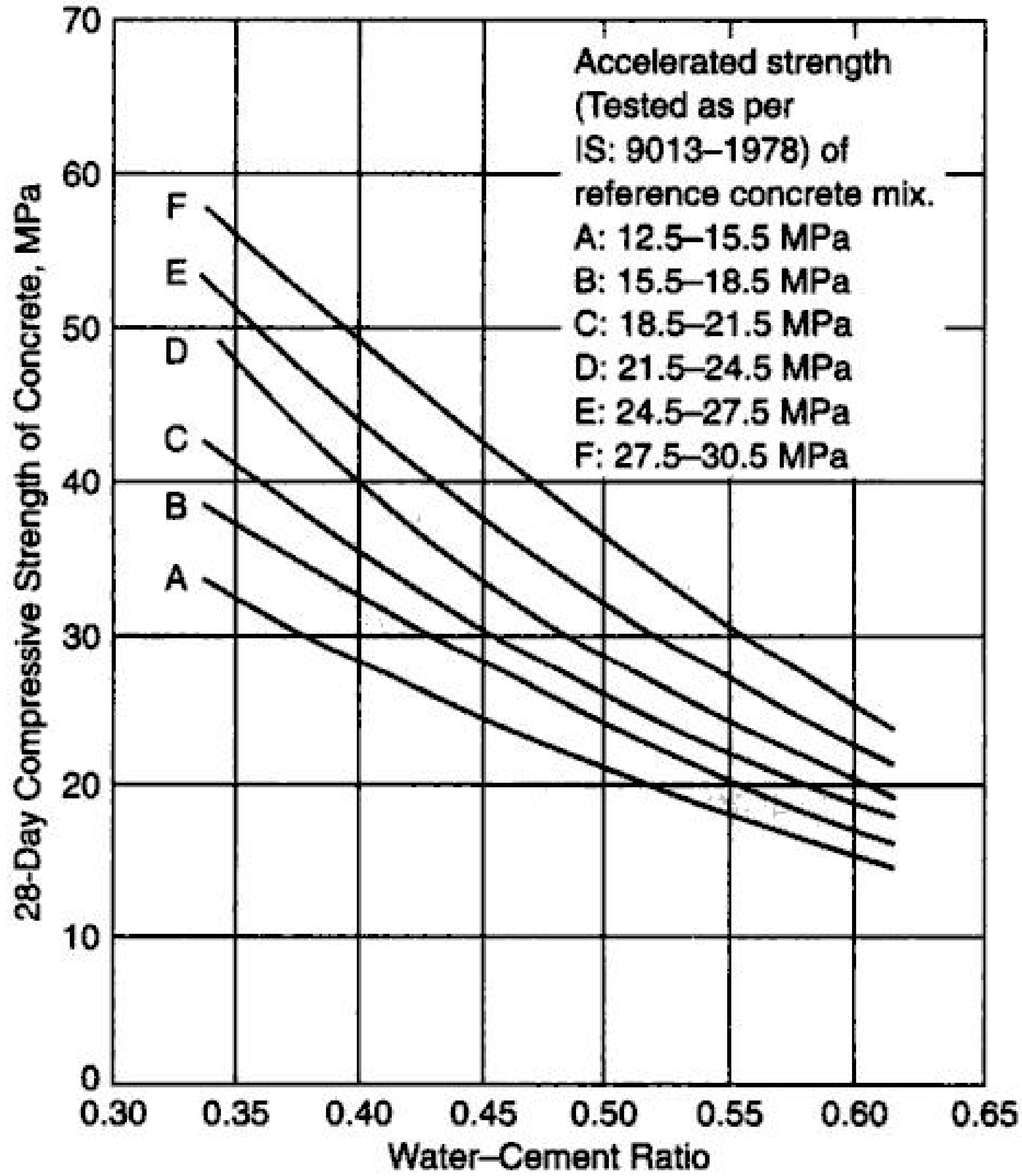

Fig. 10.9 Water–cement ratio versus compressive strength of concrete for different reference strengths (boiling-water method)

The *accelerated strength* of the *trial mix* using this water–cement ratio is checked against the characteristic target strength using the correlation of accelerated and normal 28-day strengths of concrete given in Fig. 10.10. The step-by-step procedure of mix design is as follows:

(i) The accelerated strength of reference or standard concrete using the cement at hand is determined by testing 150 mm cubes cured by the boiling-water method in accordance with IS: 9013–1978.

(ii) The water–cement ratio for the required target mean strength of normal concrete is determined by using the corresponding accelerated strength of standard concrete obtained in Step (i).

Type of aggregate	*Percentage passing the IS sieve*							
	20 mm	*10 mm*	*4.75 mm*	*2.36 mm*	*1.18 mm*	*600 μm*	*300 μm*	*150 μm*
Coarse	100	49	1	0	0	0	0	0
Fine	100	100	100	98	82	63	30	6.5

3. Target mean strength

   Target mean compressive strength = (20.0 + 1.65 × 5.0) ≈ 28.0 MPa

4. Maximum water–cement ratio

Water–cement ratio for durability under moderate exposure conditions	0.55

5. Water content

Maximum size of aggregate	20 mm
Workability	medium (slump, 30–60 mm)
Water content per cubic metre of concrete	(as per the method used).

6. Mix proportions

Mix Design

Proportioning of Aggregates to Obtain Standard Grading The gradings of available fine and coarse aggregates are to be combined in a suitable proportion so as to obtain the desired standard or specified grading. Adopt the percentage passing IS: 4.75 mm sieve as criterion. Let 1 kg of fine aggregate is combined with *x* kg of coarse aggregate to obtain desired standard grading and assume that 38 per cent of combined aggregate passes the criterion sieve. The percentage passing IS: 4.75 mm sieve individually must be equal to the total aggregate passing the same sieve, i.e.

$$100 \times (1) + 1 \times (x) = 38(1 + x) \text{ or } x = 1.63$$

Hence the fine and coarse aggregates must be combined in the proportion 1:1.63. The combined grading obtained is compared with the specified grading in Table 10.16.

Table 10.16 *Comparison of Combined Grading with Standard Grading*

Grading	*Percentage passing*							
	20 mm	*10 mm*	*4.75 mm*	*2.36 mm*	*1.18 mm*	*600 μm*	*300 μm*	*150 μm*
Combined aggregate	100	68	38	37	31	24	11	2
Specified grading	100	65	42	35	28	21	5	1

Table 10.17 *Summary of Mix Proportions Obtained by various Methods*

Method of mix design	Mix proportions by mass: Cement	Fine aggregate	Coarse aggregate	Aggregate–cement ratio
1. DoE method	1	1.38	2.69	4.07
2. ACI method	1	1.66	2.92	4.58
3. Indian Standard Recommended Guidelines	1	1.43	3.11	4.54

7. Actual quantities of material required per bag of cement

The weights of materials must be adjusted for moisture content. As an illustration, adopt the mix proportions obtained by using the Indian Standard Recommended Guidelines for mix design. Thus for one bag of cement:

Cement content = 50 kg
Fine aggregate (dry) = 71.5 kg
Coarse aggregate (dry) = 155 kg
Weight of wet fine aggregate = 71.5 × 1.02 = 72.93 kg

The free moisture present in the aggregate must be deducted from the water to be added and extra water is needed to provide for absorption. Surface moisture contributed by:

Fine aggregate = 71.5 × 0.02 = 1.43 kg
Coarse aggregate = Nil

Extra water to be added to provide for absorption in:

Fine aggregate = Nil
Coarse aggregate = 155 × 0.005 = 0.78 kg

Therefore,

Estimated requirement of water = 25.00 – 1.43 + 0.78 = 24.35 kg (or litres)

Batch mass per bag of cement

Water content = 24.35 kg
Cement content = 50.00 kg
Fine aggregate (wet) = 72.93 kg
Coarse aggregate = 155.00 kg
Total = 302.28 kg

Therefore, the net mix proportion by mass is 1.00:1.46:3.10 with free water cement ratio as 0.487.

8. Trial Mixes

Trial mixes should be prepared using these proportions as explained in Sec. 10.9 and tested to check if the mix meets the design stipulations. Otherwise, suitable adjustments should be made till it satisfies the design stipulations.

distributions where necessary. This principle of systems engineering can be applied to select a most efficient or optimal proposal from a large number of feasible alternatives which may be imperfect to some degree. Sometimes the model as modified and improved in light of preliminary design, tested for its feasibility and optimized for main system parameters may help to reach a reasonably firm decision concerning acceptance or rejection of the proposal.

In the mix design problems the mix proportions, water–cement ratio or water content may be selected as decision variables, since effectiveness of the concrete (system) can be evaluated directly or indirectly in terms of these variables. The object of the analysis is to determine the best possible set of values with respect to system effectiveness. This is called *optimal proposal.* The *objective function* which is the measure of effectiveness of a particular proposal is expressed as a function of these decision variables.

The conditions, which a mathematical model must satisfy before the decision variable values can represent a *feasible solution* are termed *constraints.* The process of mix design may be summarized by the following five sequential activities:

(i) Selection of decision variables.
(ii) Definition of objectives and identification of design criteria.
(iii) Generation of design alternatives.
(iv) Testing of feasibility of proposals.
(v) Optimization and refinement of design to maximise the effectiveness.

The *objectives* should be stated in the most basic and general terms possible. The information provided in the preceding sections may help in building up a picture of the problem environment. In concrete mix design problems, the economy of end product, i.e. the concrete may be the objective.

Once the objectives have been determined, design criteria must be identified. In mix design problems the workability, the 28-day compressive strength, and durability are generally taken as the design criteria.

Testing of Feasibility

For each alternative proposal, the first test must be that of feasibility. Technical constraints are normally carried out routinely in the course of preliminary design. The other constraints of economic and inter-disciplinary nature also exist, they must be identified and quantified at this stage. As the design proceeds any constraint which is violated will result in the proposal being modified or rejected.

Measure of Effectiveness

The most important factor influencing the nature of the final solution is the definition of objective and selection of appropriate measures of effectiveness. In its simplest form the effectiveness of a mix design may be measured in terms of cost of final product, i.e. the concrete.

To illustrate the formulation of objective function and constraints consider the following example.

Compatibility between cement and superplasticizer is one of the major factors affecting the choice of the cement and, consequently of additives for HPC. The chemical composition and fineness of the cement affect the behaviour of superplasticizers with cement.

There is an optimum cement content that produces the highest strength. Optimum strength is obtained (for the same water–cement ratio) with approximately 450 kg/m^3 of cement. The increase of strength due to larger cement content is slight, usually much less than 10 per cent. However, if cement content is increased there will be a remarkable influence on the consistency of concrete for the same water–cement ratio. Increase of cement content allows addition of more water to the mix for the same water–cement ratio, and thus enhancing the workability. Therefore, on some occasions it may be necessary to increase the cement content in the mix for better workabilities rather than for higher strengths.

Coarse Aggregate Usually, high strength concrete is produced with normal weight aggregate which is clean, free from fissures or weak planes, and free from surface coatings. Smaller size aggregates have been found to provide higher strength potential for a given $w/(c + p)$ ratio. A 20–12.5 mm nominal maximum size aggregate is commonly used for producing concretes up to 60 MPa and 16.5–10 mm is used for producing concretes above 60 MPa. Aggregates also influence the consistency of concrete. For good consistency a selected coarse aggregate should have: a low coefficient of absorption ($\leq$ 1 per cent); a shape coefficient $\leq$0.25; and a maximum size of 16.5 to 10 mm. To improve compressive strength, aggregate shall have: a Los Angeles coefficient $\leq$15; and a crushing index $\leq$15. The influence of aggregate grading on workability and strength for the same water–cement ratio is not much. However, the grading has to be maintained within the limits to avoid segregation.

Limestone Aggregates are especially suitable to produce HPC due to the development of *epitaxic adherence* which increases strength remarkably. In this case, even aggregates not complying with the preceding limits of the Los Angeles coefficient or crushing index can produce very high strength concretes.

Fine Aggregate The *grading* and *particle shape* of fine aggregate significantly influence the mixing water content and compressive strength. The quantity of cement paste required per unit volume of a concrete mixture decreases as the relative volume of coarse aggregate *versus* fine material increases. A difference of one per cent in void content in the sand may result in approximately 4.55 kg/m^3 difference in water demand. *Fine aggregates* with a *fineness modulus* in the range of 2.5 to 3.2 are preferable for high performance concretes. Concrete mixtures made with a fine aggregate that has a fineness modulus of less than 2.5 may be *sticky* and result in *poor workability* and higher water requirements.

Fly ash The use of fly ash in concrete results in a lowered cost of materials in the finished concrete with improved performance characteristics. It is

The relationships between water–cement ratio and mean compressive strength of concrete f_{cm} (MPa) using silica fume can be expressed by Eq. (10.16):

$$f_{cm} = e^{D-R\,(w/c)} \tag{10.16}$$

The above expression contains a constant part e^D and a variable part $e^{-R\,(w/c)}$. The constant part depends upon the type of aggregate. For a coarse aggregate of average properties, $D = 4.95$ and the relation is expressed by Eq. (10.17):

$$f_{cm} = 140e^{-R\,(w/c)} \tag{10.17}$$

The parameter R, given in Table 10.20, depends on the strength of the cement and on the proportion of silica fume in the mix. If the concept of *efficiency factor* (expressing the equivalent amount of silica fume which produces the same strength as a fixed content of cement) is introduced, Eq. (10.17) can be written as Eq. (10.18):

$$f_{cm} = 140e^{-R'(w/c)/[1\,+\,(k-1)\,a]} \tag{10.18}$$

where 'a' represents the fraction of silica fume by weight of cement and the parameter R represents the *cementing efficiency factor.* The maximum efficiency factor for the silica fume normally recommended is 4.00. The expressions are valid for the concrete mixes containing silica fume and having water–cement ratios less than 0.40.

10.9.4 Mix Proportioning Procedure

The procedure consists of a series of steps, which when completed provide a mixture meeting strength and *workability* requirements based on the combined properties of the individually selected and proportioned ingredients. Following are the necessary steps.

1. To obtain the desired *workability* an *initial starting slump* of 25–50 mm prior to addition of HRWR is recommended. This will ensure an adequate amount of water for mixing and allow *the superplasticizer* to be effective. For high strength concretes made without HRWR, a recommended slump range of 50–100 mm may be chosen depending on the type of work to be done.
2. Depending on the level of quality control contemplated on the site, the target mean strength necessary to reach characteristic strength is determined. Based on this target strength the maximum size of the coarse aggregate is selected.
3. The optimum content of the coarse aggregate is selected depending on its strength, potential characteristics and maximum size. The recommended optimum *coarse aggregate contents*, expressed as a fraction of the dry-rodded unit weight are given in Table 10.21 as a function of *nominal maximum size*. Using the optimum coarse aggregate fraction values obtained from Table 10.21, the oven-dry weight of the coarse aggregate per cubic metre of concrete is computed.

11. Once the trial mix proportions have been adjusted to produce desired workability, strength specimens shall be cast from the trial batches, cured and tested as per codal provisions.

It is recommended that the cement content should be less than 500 kg/m^3. If it is necessary to exceed this limit to get a good workability, it is desirable to reconsider the compatibility of cement–mineral additive to reduce water and cement content.

The mix proportioning procedure is illustrated by Example 10.8.

Example 10.8 Proportion an HSC mix of grade M 60 with flowable consistency using fly ash or silica fume, a high range water reducer and retarding admixture for a large-scale construction with closely spaced reinforcement and prestressing cables. Special attention should be paid to split tensile strength of concrete. The desirable characteristic split tensile strength is 4.00 MPa. The concrete is to be pumped to a height of 52 metres.

Solution

(i) Selection of Materials

Cement Considering compressive strength at various ages, fineness, heat of hydration, alkali content, and compatibility with admixtures, Grade 43 Portland cement is selected.

Coarse Aggregate As the tensile strength of concrete is of prime importance, 20 mm or 12.5 mm maximum size crushed granite stones aggregates are used in the concrete mix.

Fine Aggregate A properly graded river sand which is washed at the site to remove deleterious materials and chloride contamination is used as a fine aggregate.

Admixture A high range water-reducing admixture is used for this concrete of low water–cementitious material ratio. The optimum dose of admixture is evaluated by consistency test. Retarder is also added to increase the setting time of the concrete and improve the slump retention properties to avoid cold joints during large construction.

Water The potable water available at site is used as mixing water. The thermal stresses can be reduced by restricting the placement temperature of concrete below 23 °C.

(ii) Design Stipulations and Materials Characteristics

(a) *Characteristic strength*	60 MPa
(b) *Cement*	
Type of cement	OPC of Grade 43
Specific gravity	3.15

Weight of wet sand = 524.00 × (1 + 0.06) ≈ 555.00 kg

Weight of wet coarse aggregate = 1095.00 × (1 + 0.005) ≈ 1100.00 kg

Net weight of water =172.00 – 524.00 × (0.06 – 0.01) – 1095.00 × (0.005 – 0.0075) ≈ 148.00 kg.

Batch weights	cement	:	FA	:	sand	:	CA	:	water
	470.00	:	117.00	:	555.00	:	1100.00	:	148.00

Adjust batch weights for each trial mix to obtain required slump, before and after addition of HRWR.

(iv) Mix Proportions with Silica Fume

It is generally proportioned on the basis of experience and test results from the similar projects in the past using similar ingredients. In addition, extensive testing of trial batches will be required to arrive at optimum mix proportions. However, trial batch proportions may be estimated from the concrete mix proportions using fly ash.

IS: 456–2000 recommends silica fume as replacement of cement in the proportion of 5 to 10 per cent of the cementitious materials. In the current example consider a cementing efficiency factor of silica fume as 3.25.

The silica fume content = 117.00/3.25 ≈ 36.00 kg

From the yield point of view,

Sand content

$$= \left[1.0 - \left[\frac{470.00}{3.15 \times 1000}\right] - \left[\frac{36.00}{2.2 \times 1000}\right] - 0.3967 - 0.1820 - 0.0200\right]$$

$$\approx 0.2357 \text{ m}^3$$

Weight of sand = 0.2357 × 2.6 × 1000 ≈ 613 kg

HRWR at the rate of 1.75 per cent = 0.0175 × (470 + 36) ≈ 8.90 kg (8.9 litre by volume)

Net weight of water excluding that in HRWR = 182.00 – 8.90 ≈ 173 kg

Percentage of silica fume = 36 × 100/(470 + 36) ≈ 7.11, which is within the optimum range of 6.7 to 9.3 per cent.

The mix proportions using silica fume are:

Ingredients	*Weight per m^3 of concrete, kg*
Cement	470
Silica fume	36
Sand (dry)	613
Coarse aggregate	1095
Water (including that in the retarder)	173
HRWR	8.9 litre

Apply corrections for the total moisture present in the coarse aggregate and sand to obtain field batch weights.

Table 10.23 *Frequency of Sampling of Concrete*

Quantity of concrete in the job, m^3	*Number of samples**
1–5	1
6–15	2
16–30	3
31–50	4
51 and above	4 plus one additional sample for each additional 50 m^3 or part thereof

Note: *At least one sample shall be taken from each shift. Where concrete is produced at continuous production unit, such as *ready-mixed concrete plant, frequency of sampling* may be agreed upon mutually.

(i) Every sample has a test strength not less than $(f_{ck} - 3)$ MPa for M15 concrete and $(f_{ck} - 4)$ MPa for M20 or higher grade concretes.

(ii) For M15 grade concrete, the mean strength of the group of 4 non-overlapping consecutive test samples is not less than $f_{ck} + 0.825\ S$ or $(f_{ck} + 3)$ MPa whichever is greater.

For M20 or higher grade concrete the mean strength of the group of 4 non-overlapping consecutive test results is greater than $f_{ck} + 0.825\ S$ or $(f_{ck} + 4)$ MPa whichever is greater, where f_{ck} is characteristic strength, and S is established standard deviation (rounded to 0.5 MPa).

There are many factors that influence the variability of strength measurements in the field. It is generally noticed that *test strength under ideal field conditions* attains only 90 per cent of the strength measured by tests performed *under laboratory conditions*. To take into account the variation of individual test sample, the laboratory design strength can be obtained by increasing the average field strength or the target mean strength by 10 per cent or by dividing it by the factor, 0.90.

10.14 FIELD ADJUSTMENTS

In a concrete mix if W, C, F_a and C_a are the required quantities of water, cement, fine aggregate, and coarse aggregate, respectively, to produce one cubic metre of fully compacted concrete, then based on concept that volume of compacted concrete is equal to the sum of the absolute volumes of all ingredients, the relation given by Eq. 10.23 is obtained

$$\frac{W}{1000} + \frac{C}{1000S_c} + \frac{F_a}{1000S_{fa}} + \frac{C_a}{1000S_{ca}} = 1.00 \qquad (10.23)$$

When entrained air is also present and its content is v per cent of the volume of concrete, the right-hand side of Eq. 10.23 would read as, $(1.00 - 0.01v)$.

If specific gravities of fine and coarse aggregates are assumed to be same say S_a, then for a given type of cement the Eq. 10.23 can be written as:

Table 10.24 Contd.

C. *Mineral Additives*		
C.1	Type of additive	fly ash/silica fume
C.2	Specific gravity	______________
C.3	Bulk density	______________kg/m^3
C.4	Efficiency factor	______________
C.5	Volume (if predetermined)	______________

PART-II: MIX DESIGN

	Stage *Stage*	*Item*	*Reference of calculation*	***Value***
1	1.1	Characteristic compressive strength (f_{ck})	Specified (Part-I)	__at __days Proportion of defective specimens______per cent
	1.2	Standard deviations (S)	Fig./Table	______________MPa
	1.3	Probability factor (k)		______________
	1.4	Target mean strength (f_t)	$f_t = f_{ck} + kS$	__+ __ ×__= __ MPa
	1.5	Free water–cement ratio	Table/Fig.	—use the lower value
	1.6	Maximum free water–cement ratio	Specified	
2	2.1	Compacting factor or Slump or V-B	Specified	C.F.—Slump—mm or V-B—s
	2.2	Maximum aggregate size	Specified (Part-I)	_______mm
	2.3	Free water content	Table	_______kg/m^3
3	3.1	Cement content	_______	_____ / _____= ____kg/m^3
	3.2	Maximum cement content	Specified	_______kg/m^3
	3.3	Minimum cement content	Specified	_______kg/m^3. Use if greater than Item 3.1 and use to calculate Item 3.4
	3.4	Modified free water–cement ratio		______________
4	4.1	Relative density of aggregate (SSD)	Specified (Part-I)	______________
	4.2	Concrete density	_______	_______kg/m^3
	4.3	Total aggregate content		___-___-___=___kg/m^3
5	5.1	Grading of fine aggregate	Specified (Part-I)	______________
	5.2	Proportion of fine aggregate	Fig.	_______per cent
	5.3	Fine aggregate content		___×___=_____kg/m^3
	5.4	Coarse aggregate content		___−___=_____kg/m^3

			Water (kg)	Cement (kg or l)	Fine aggregate (kg)	Coarse aggregate (kg)
6	6.1	Ingredients				
	6.2	Quantity per m^3 (to nearest 5 kg)	_____	_____	_____	_____
	6.3	Quantity per trial mix of ____m^3	_____	_____	_____	_____
	6.4	Ratio	_____	_____	_____	_____

blending of different particle sizes of the ingredients to be uniformly distributed throughout the concrete mass, and (ii) a vigorous rubbing action of cement paste on to the surface of the inert aggregate particles. Concrete mixing is normally done by mechanical means called *mixer,* but sometimes the *mixing* of concrete is done by hand. Machine mixing is more efficient and economical compared to hand mixing.

In the mixing process, the cement paste is formed first with simultaneous absorption of water in the aggregates. In the second stage the cement paste coats the aggregate particles. The mixing process should be continued till a thoroughly and properly mixed concrete is obtained. At the end of this stage the concrete appears to be of uniform colour and grading. The uniformity must be maintained while discharging the concrete from the mixer. As a matter of fact the classification of the mixers is based on the technique of discharging the mixed concrete as follows:

(i) the tilting type mixer,
(ii) the non-tilting type, and
(iii) the pan or stirring mixer.

The size of a mixer is designated by a number representing its nominal mix batch capacity in litres, i.e. the total volume of mixed concrete in litres which can be obtained from the mixer per batch. The capacity of a mixer for a particular job should be such that the required volume of concrete per hour is obtained without speeding up the mixer or reducing the mixing time below the specified period and without overloading the mixer above its rated capacity. The standardized sizes of the mixers given in IS: 1971–1968 are listed in Table 11.2. Most of the mixers can handle a 10 per cent overload satisfactorily. If the quantity mixed is much less than the rated capacity of the mixer the resulting mix may not be uniform, and the mixing operation becomes uneconomical.

Table 11.2 *Standard Sizes of Mixer*

Type of mixer	*Nominal mixed batch capacity, liters*
Tilting (T)	85T, 100T, 140T, 200T
Non-tilting (NT)	200NT, 280NT, 340NT, 400NT, 800NT
Reversing (R)	200R, 280R, 340R, 400R

In the *tilting-type mixer,* the chamber (drum) which is generally bowl-shaped or double-conical-frustum type, is tilted for discharging. The efficiency of the mixing operation depends upon the shape and design of the vanes (blades) fixed inside the drum. These vanes direct the concrete into tracing a circulatory path. In addition, there is vertical free falling action due to gravity. The mixed concrete is discharged from the open top of the drum by tilting it downwards. The discharge action is always good as all concrete can be tipped out rapidly under gravity in an unsegregated mass as soon as the drum is tilted. For this reason

the larger aggregate is thrown to the far side of the container being charged and the mortar is collected at the near side, thus resulting in *segregation*.

The plant required for transporting the concrete varies according to the size of the job and the level at which the concrete is to be placed. The principal methods of transporting concrete from the mixer are the following.

(i) Barrows
  (a) Wheel barrows and handcarts
  (b) Power barrows or powered buggies or dumpers
(ii) Tippers and lorries
(iii) Truck mixers and agitator lorries
(iv) Dump buckets
(v) The monorail system or trolley or rails

The most commonly used method of transporting concrete by the hand pans passing from hand to hand is slow, wasteful and expensive. If concrete is to be placed at or below the mixer level, steel wheel barrows are a better mode of transportation. Concrete can be discharged from the wheel barrow to the required point. When concrete is to be placed much below the general ground level, as in basement slabs, foundations, etc. a wooden or steel chute may be used for chuting the concrete into place. The wheel barrows are suitable for small jobs and where the length of transport is small, and over muddy ground. The average quantity that can be carried in one wheel barrow is about 35 litres (80 kg). Sometimes, for relatively bigger jobs, *power barrows* which are motorized version of wheel barrows are used.

Dumpers and ordinary open-steel body tipping lorries can be used economically for hauls of up to about 5 km. These lorries are suitable only for dry mixes to avoid difficulties caused by *segregation* and *consolidation*. The time of journey should be as short as possible. It is essential that the lorry body be watertight to prevent loss of fines. The concrete has to be covered with tarpaulins to prevent the concrete being exposed to sun, wind and rain. If the haul is long, agitators have to be used to prevent segregation. Steel buckets transported by rail or road may be used to transport the concrete for long distances and for large jobs like dams, bridges, etc. While using this method it is necessary to see that: (i) the entire mixer batch is placed in the bucket, and (ii) segregation is prevented while filling the bucket.

The monorail system is useful when the ground conditions are not suitable for normal wheeled traffic. In the monorail system, the rail can easily be provided at such a level that the concrete be tipped directly into the formwork. Basically, the system consists of a power wagon mounted on a single rail capable of a travelling speed of 90 m/min. The engine may be diesel or petrol powered, without a driver.

Quality of ready mixed concrete is generally specified in terms of *performance parameters,* i.e. purchaser specifies the strength level and intended use of concrete. It is the best way to order ready mixed concrete because the ready mixed concrete (RMC) producer, who is an expert in this field, would design an economical mix with the desired properties. The RMC producer accepts responsibility for the design of the mixture for desired performance. In another system the quality of ready mixed concrete is specified in terms of *prescriptive specifications*, i.e. purchaser specifies aggregate size, slump, air content, cement content or weight of cement per cubic meter of concrete, maximum water content and admixtures required. In this case, the purchaser accepts the responsibility for concrete strength and its performance.

In the first system ready mixed concrete producer independently selects the material proportions based on previously developed guidelines and experience resulting in an economical and practical mix. Thus, to serve the goal of materials conservation, a paradigm shift is needed from *prescriptive* to *performance* based standard specification for materials.

RMC is ordered and supplied by volume (cubic metre) in a freshly mixed and unhardened state. When ordering concrete 5 to 10 per cent more concrete than estimated from a volumetric calculation is ordered. This will account for the wastage or spillage, over-excavation, spreading of forms, some loss of entrained air, settlement of wet mixture, and change in volume, dry concrete volume is 1 to 2 per cent less than that of fresh concrete. It is important not to order too much concrete. The processing and disposal of returned concrete is an expensive proposition for the ready mixed concrete producer, who has to comply with various environmental regulations.

Proportioning of Ready Mixed Concrete

The ready mixed concrete embodies the concept of treating the concrete in its entity as a building material rather than ingredients. The proportioning of an RMC aims at obtaining an economical and practical combination of materials to produce concrete with the properties desired for its intended use, such as workability, strength, durability and appearance. The following basics of a good concrete mix should be considered while proportioning RMC for the desired performance.

(i) Concrete aggregates are required to meet appropriate specifications and in general should be *clean, strong* and *durable.*

(ii) *Fly ash* or other *supplementary cementitious* materials, which enhance concrete properties, are normally added to RMC. *The key to quality concrete is to use the least amount of water that can result in a mixture which can be easily placed, compacted and finished.*

(iii) *Admixtures,* are commonly used in relatively small quantities to improve the properties of fresh and hardened concrete such as the rate of setting and strength development of concrete, especially during hot and cold weathers.

loss of mortar which may result in honeycombing. Before placing the concrete, the inside of the forms are cleaned and treated with a release agent to facilitate their removal when concrete is set. Any coating of the hardened mortar on the forms should be removed. The reinforcement should be checked for tightness and clean surface. It should also be freed of all loose rust or scales by wire brushing or any other method. Coatings like paint, oil, grease, etc. are removed. The reinforcement should be checked for conformity with the detailing plans for size, spacing and location. It should be properly spliced, anchored and embedded to a given minimum distance from the surface. Anchor bolts, pipe sleeves, pipe conduits, wiring and other fixtures should, in general, be firmly fixed in position before the concrete is placed. Rubbish, such as sawdust shavings and wire, must be blown out with compressed air.

The concrete should be placed in its final position rapidly so that it is not too stiff to work. Water should not be added after the concrete has left the mixer. The concrete must be placed as closely as possible to its final position. It should never be moved by vibrating it and allowing it to flow, as this may result in segregation which will show on the surface of the finished work. When placing the concrete, care should be taken to drop the concrete vertically and from not too great height. *Segregation,* if it occurs, should be eliminated by taking remedial measures.

The surfaces against which the fresh concrete is to be placed must be examined as to their possible effect in absorbing mixing water. For example, subgrades should be compacted and thoroughly dampened to prevent loss of moisture from concrete.

Where fresh concrete is required to be placed on a previously placed and hardened concrete, special precautions must be taken to clean the surface of all foreign matter and remove the *laitance* or scum before the fresh concrete is placed. For securing a good bond and watertight joint, the receiving surface should be made rough and a rich mortar placed on it unless it has been poured just before. The mortar layer should be about 15 mm thick, and have the same water–cement ratio as the concrete to be placed. In all cases, the base course should be rough, clean, and moistened. The surface can be cleaned by a stiff or steel broom a few hours after placement when the concrete is still soft enough to allow removal of scum but hardened enough not to permit loosening of aggregate particles.

It is becoming increasingly more economical to place concrete in deep lifts. This technique saves time and reduces number of horizontal joints. For placing in deep lifts to be successful, the mix must be designed to have a low risk of segregation and bleeding. The concrete should be introduced into the forms through trunkling, as this reduces impact damage to the forms and reinforcement, and enables the layer of concrete to be built up evenly.

The actual procedure depends largely upon the type of structure, the quality of concrete and of the receiving surface. In mass concrete construction, as in

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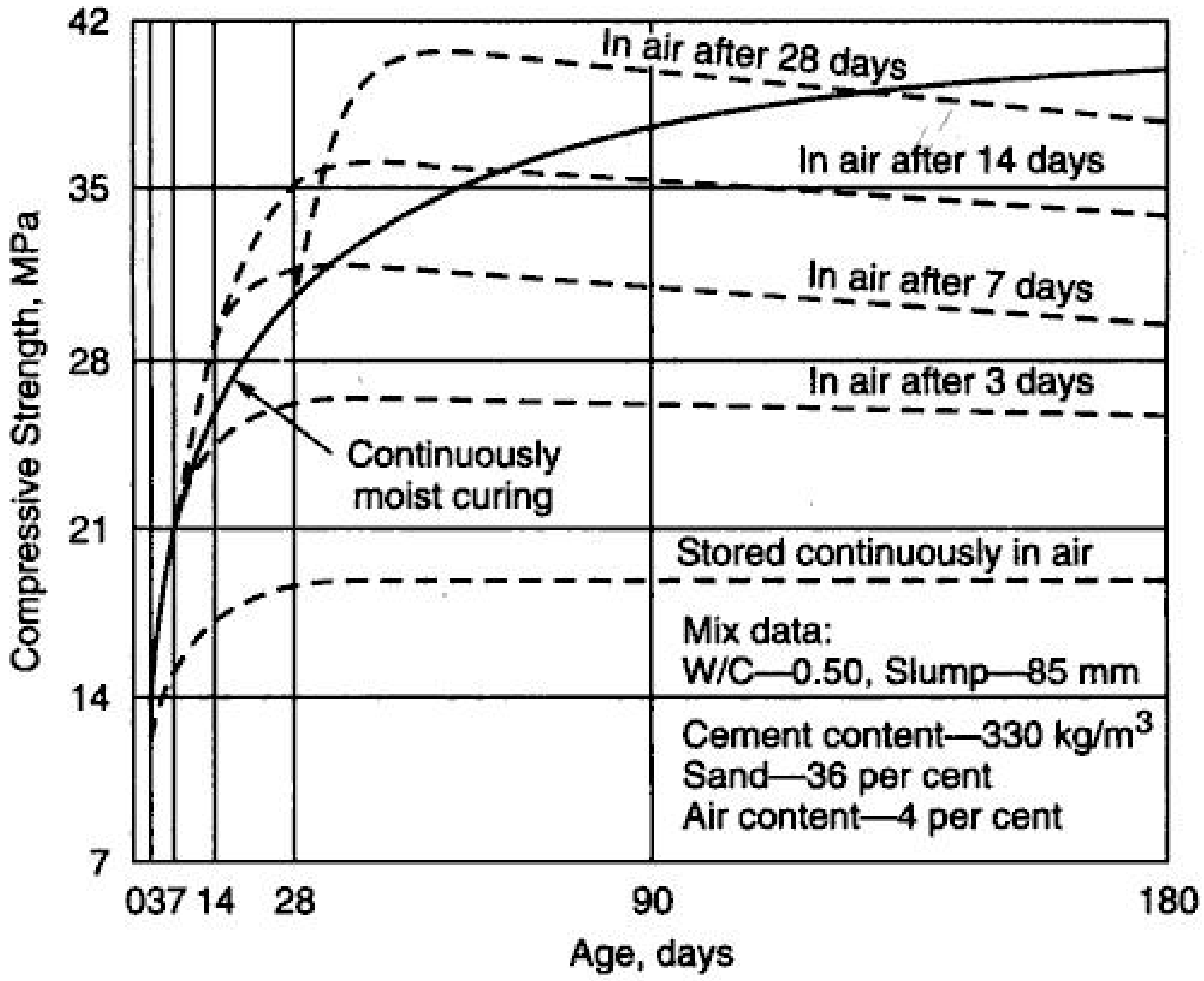

Fig. 11.2 Strength of concrete dried in air after preliminary moist curing

evaporation and self-desiccation. The rate of evaporation depends upon the temperature and the relative humidity of the surrounding air and on the velocity of wind. An air-cured concrete develops considerably less strength compared to the moist-cured concrete as is seen in Fig. 11.2. The rate of development of strength with curing period is given in Fig. 11.3.

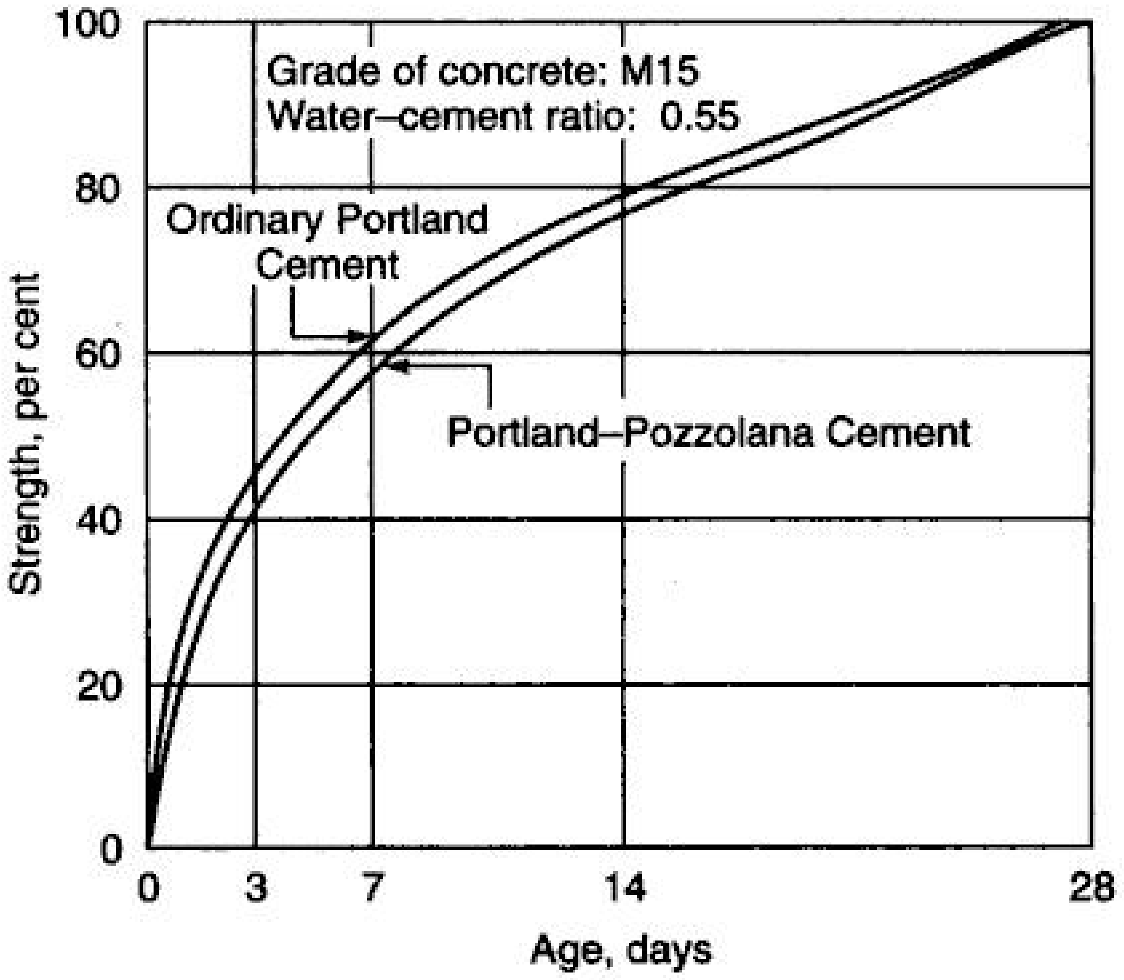

Fig. 11.3 Development of strength with curing period

The rate of development of strength not only depends on the period of curing but also on the temperature during the period of curing. The influence of

be economical. The effectiveness of the method as compared with the ponding is shown in Fig. 11.7.

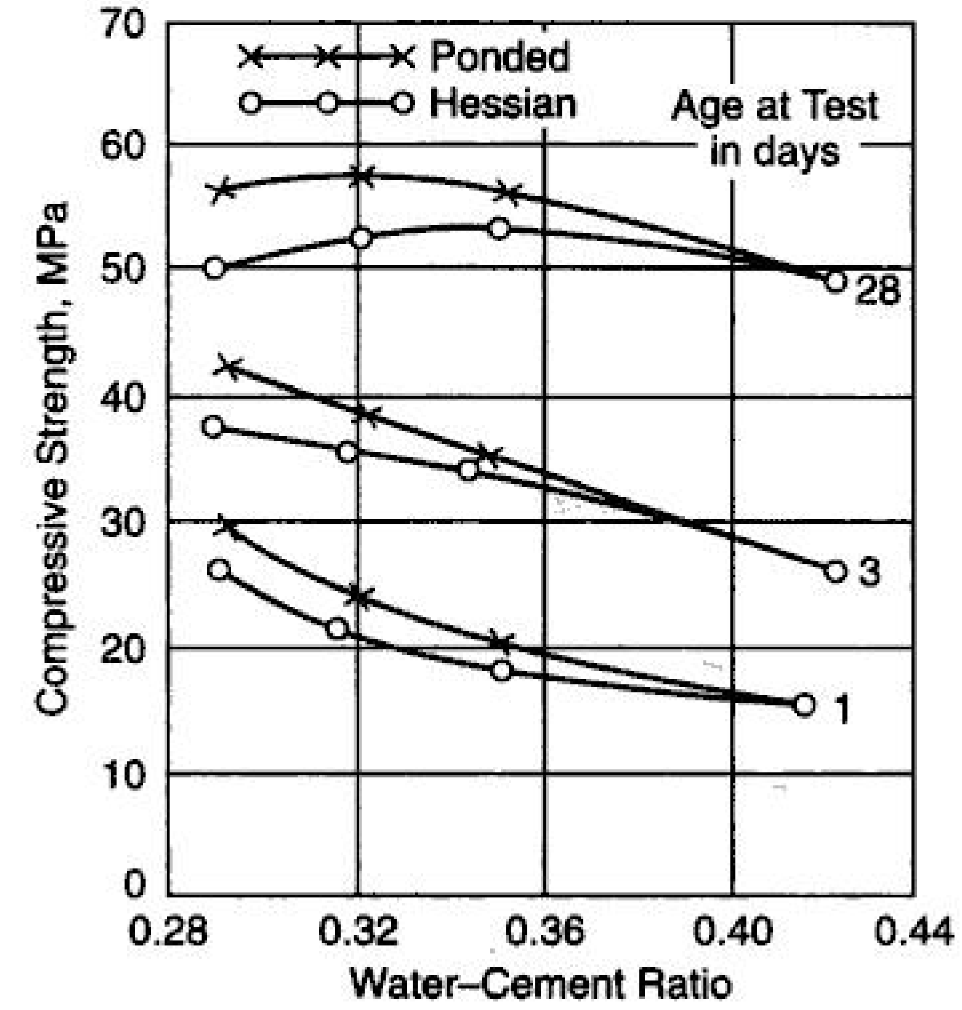

Fig. 11.7 Effect of curing condition on the compressive strength of concrete

Sprinkling of Water

This is a useful method for curing vertical or inclined surfaces of concrete wherein the earlier methods cannot be adopted. The method is not very effective as it is difficult to ensure that all the parts of concrete be moist all the time. The spraying can be done in fine streams through nozzles fixed to a pipe spaced at set intervals. *Flogging* is done in the same way except that the flogging nozzles produce a mist-like effect, whereas spraying nozzles shed out fine sprays.

(ii) The methods preventing or minimizing the loss of water by interposing an impermeable medium between the concrete and the surrounding environment are as follows.

Covering the Surface with Waterproof Paper

Waterproof paper prevents loss of water in concrete and protects the surface from damage. The method is satisfactory for concrete slabs and pavements. A good quality paper can be often reused. The paper is usually made of two sheets struck together by rubber latex composition.

Plastic sheeting is a comparatively recent innovation as a protective cover for curing concrete. Being light and flexible, it can be used for all kinds of jobs, effectively covering even the most complex shapes. Several types of sheets, which are guaranteed to give excellent results consistent with economy and can be used over and over again, are available. Most plastic sheetings used in the concrete industry are milky or white in appearance, and this helps keep the, concrete temperature at a reasonable level. Plastic sheeting can be welded at the

(i) The type of cement, (ii) the aimed stripping and delivery strengths, and (iii) the accelerator.

A typical steam curing cycle is given below:

Presteaming period	3 hours
Temperature rise period	4 hours
Period of maximum temperature	4 hours
Cooling off period	5 hours

High-Pressure Steam Curing

In the case of normal steam curing at atmospheric pressure, the ultimate strength of concrete may be adversely affected if the temperature is raised rapidly. This difficulty can be overcome by employing the steam at a pressure of 8 atmospheres. The process is termed *high-pressure steam curing*. High-pressure steam curing is done in the cylindrical steel chambers called autoclaves. The concrete products, after a suitable presteaming period, are wheeled on racks into the autoclaves. The steam is let in gradually until the prescribed pressure or temperature (generally about 1 MPa or 185 °C) is reached. This heating stage should be completed and the prescribed pressure reached in about three hours. The increase in temperature allowed is up to 50 °C in the first hour, up to 100 °C in second hour and up to 185 °C in the third hour. The period of treatment under full pressure depends upon the strength requirements. This period is 7 to 10 hours for hollow block products and 8 to 10 hours for slab or beam elements, the period increasing with the thickness of concrete. The steam is cut off and the pressure is released after the completion of this stage and the products are left in the autoclaves for two hours for cooling off gradually.

High-pressure steam curing is usually applied to precast products when any of the following characteristics is desired.

(a) *High early strength* With high-pressure steam curing, the compressive strength at 24 hours is at least equal to that of 28-day normally cured products.

(b) *High durability* High-pressure steam curing results in an increased resistance to sulphate action and other forms of chemical attack, and to *freezing* and *thawing*.

During the hydration of cement at higher temperatures, the calcium hydroxide released as the result of primary reaction, reacts with finely divided silica, which is present in the coarse and fine aggregates, forming a strong and fairly unsoluble compound. This results in higher concrete strengths. *Leaching* and *efflorescence* are minimized due to reduction in free calcium hydroxide content. The hydrating dicalcium silicates and tricalcium alluminates react together at high temperatures to form sulphate-resisting compounds. Hence *autoclaved products* show higher resistance to sulphate attack. The *initial drying shrinkage* and *moisture movements* are also considerably reduced. On the debit side, *high-pressure steam*

Bending stress	8.4 MPa
Compressive stress parallel to grains	6.4 MPa
Compressive stress across the grains	2.6 MPa
Shear stress parallel to grains	0.92 MPa

The stresses are based on the assumption that the timber will remain dry. In case it is subjected to alternate wetting and drying, the stresses should be reduced to 0.85 times the value given above and, if continuously wet, to 0.65 times these values.

In practice it would be economical to standardize the size of timber used in formwork so that their repeated use is possible. This would necessarily entail proper planning, and great care is to be exercised by the designer in adjusting the parameters in such a way that the standard scantling can be used. Sometimes it would result in a bit of over-expenditure on concrete but, in the long run, especially in large projects, the saving in formwork will offset this loss. It is better to use clamps and screws, rather than nails, in the formwork to facilitate its stripping and reuse.

Plywood Formwork

Plywood sheets bound with synthetic resin adhesive are being widely used now a days. The thickness of ply varies from 3 to 18 mm. Sizes less than 6 mm thick are used for lining the timber formwork to get neat and smooth surface finish and as a formwork for curved surfaces. The common sizes are 1200 × 1200 mm to 3000 × 3000 mm. The main advantage is that large panel surfaces are available. The fixing of forms is rapid and economical. It does not warp, swell and shrink during the setting of concrete. Moreover, it has high impact resistance.

Steel Formwork

Steel formworks are commonly employed for big projects where the forms are to be repeatedly used. The steel forms can be easily fabricated and do not require many adjustments as the units are standardized. They give smooth surfaces needing very little finishing. These prove to be economical and are best suited for circular columns and flat slab construction. Joists can be used from wall to wall to support the steel beams used for stiffening the plates. Square steel plates of 500 mm size are generally used. Light steel sheet panels of 500 mm size and stiffened with angles are also available.

11.8.4 Design Loads on the Formwork

The formwork used in the construction of roofs and floors has to carry its own weight, the weight of wet concrete, the live load due to labour, and the impact owing to the pouring of concrete, etc. The surfaces of the formwork should be so dressed that after the deflection due to concrete weight, etc. the surface takes the shape desired by the designer. In the design of formwork for columns or

given amount of *air-entraining agent,* hot concrete entrains less air than does concrete at normal temperatures.

12.2.1 Recommended Practices and Precautions

Temperature Control of Concrete Ingredients

The temperature of the concrete can be kept down by controlling the temperature of the ingredients. The *aggregates* may be protected from direct sunrays by erecting temporary sheds or shelters over the aggregate stockpiles. Water can also be sprinkled on to the aggregate before using them in concrete. The mixing water has the greatest effect on lowering the temperature of concrete, because the specific heat of water (1.0) is nearly five times that of common aggregate (0.22). Moreover, the temperature of water is easier to control than that of other ingredients. Under certain circumstances, the temperature of water can most economically be controlled by mechanical refrigeration or mixing with crushed ice. The precooling of aggregates can be achieved at the mixing stage by adding calculated quantities of broken ice pieces as a part of mixing water, provided the ice is completely melted by the time mixing is completed.

Proportioning of Concrete Mix

The mix should be designed to have minimum cement content consistent with other functional requirements. As far as possible, cement with lower *heat of hydration* should be preferred to those having greater *fineness* and heat of hydration. Use of water-reducing or set-retarding admixtures is beneficial. *Accelerators* should not be used under these conditions.

Production and Delivery

The temperature of aggregates, water and cement should 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 batching plant should be measured with a suitable metal clad thermometer. The period between mixing and delivery should be kept to an absolute minimum by coordinating the delivery of concrete with its rate of placement.

Placement and Curing of Concrete

The formwork, reinforcement and subgrade should be sprinkled with cool water just before the placement of concrete. The area around the work should be kept wet to the extent possible to cool the surrounding air and increase its humidity. Speed of placement and finishing helps minimize problems in hot weather concreting. Immediately after compaction, the concrete should be protected to prevent the evaporation of *moisture* by means of wet (not dripping) gunny bags, hessian, etc. After the concrete has attained a degree of hardening sufficient to withstand surface damage, moist-curing should begin. Continuous curing is important because the volume changes due to alternate wetting and drying

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dams are to be constructed to reduce the velocity of flow through the construction zone.

12.4.1 Concreting Methods

The following are the principal techniques which have been used for placing concrete underwater.

(i) Placing in dewatered caissons or coffer dams
(ii) Tremie method
(iii) Bucket placing
(iv) Placing in bags
(v) Prepacked concrete

The placing in dewatered caissons or coffer dams follows the normal *in-the-dry* practice.

Tremie Method

A *tremie* is a watertight pipe, generally 250 mm in diameter, having a funnel-shaped hopper at its upper end and a loose plug at the bottom or discharge end as shown in Fig. 12.1. The valve at the discharge end is used to de-water the tremie and control the distribution of the concrete. The tremie is supported on a working platform above water level, and to facilitate the placing it is built up in 1 to 3.5 m sections.

During the concreting, air and water must be excluded from the tremie by keeping the pipe full of concrete all the time; and for this reason the capacity of the hopper should be at least equal to that of the tremie pipe. In charging the tremie a plug formed of paper is first inserted into the top of the pipe. As the hopper is filled the pressure of fresh concrete forces the plug down the pipe, and the water in the tremie is displaced by concrete.

For concreting, the tremie pipe is lowered into position and the discharge end is kept as deeply submerged beneath the surface of freshly placed concrete as the head of concrete in tremie permits. As concreting proceeds the pipe is raised slightly and the concrete flows outward. Care should be taken to maintain continuity of concreting without breaking the seal provided by the concrete cover over the discharge end. Should this seal be broken, the tremie should be lifted and plugged before concreting is recommended. The tremie should never be moved laterally through freshly placed concrete. It should be lifted vertically above the surface of concrete and shifted to its new position.

When large quantities of concrete are to be placed continuously, it is preferable to place concrete simultaneously and uniformly through a battery of tremies, rather than shift a single tremie from point to point. It has been recommended that the spacing of tremies be between 3.5 and 5 m and that the end tremies should be about 2.5 m from the formwork.

(iii) Pumping the grout into the aggregate mass from bottom at carefully designed positions through a network of pipes. The formwork should be constructed at the top of the coarse aggregate in this method.

The quantity of grout in any of these methods should be estimated from the void contents of the coarse aggregates. The grout pressure employed will be of the order of 0.2 to 0.3 MPa.

This technique is very much suited for underwater construction and repair work of mass concrete structures, such as dams, spillways, etc. The prepacked concrete is known to exhibit lower drying shrinkage and higher durability, especially the freezing and thawing resistance compared to ordinary concrete of the same proportions. The rate of development of strength is comparatively slow for the first two months and the eventual strengths are about the same as for normal concrete. In USA and USSR, the tremie method is most commonly used. In Holland, where large volumes of concrete have been placed under water, the usual method is that of placing by bucket. The bag method is now-a-days seldom used for important works overseas, but has found some applications in the building up of permanent underwater forms.

In the HCl heat of solution method, a sample of concrete is diluted by adding a fixed quantity of water. To the diluted solution is added hydrochloric acid, resulting in an exothermic reaction which decomposes the cement contained in the sample. The heat of reaction reaches a steady temperature quickly. The temperature rise is related to the cement content of sample of concrete. This method has been developed by the Cement Research Institute of India.

A comparison of the four analysis methods has been given in Table 13.2.

Table 13.2 *Comparison of various Analysis Methods*

Method of analysis	*Variation of estimated cement content from the actual values, per cent*	*Approximate time required for sample preparation*	*Approximate time required for conducting the test*
IS: 1199–1959 method	–11.3 to 2.2	Nil	2 h
RAM method	–10.3 to 6.7	Nil	10 min
EDTA method	–6.6 to –3.1	1.5 h	30 min
HCl heat of solution method	–10.4 to +10	Nil	10 min

The *accelerated strength tests* give a reliable idea about the potential 28-day strength of concrete. The details of accelerated strength tests for the purpose of *quality control of concrete* are available in IS: 9013–1978. Either of the following two methods may be adopted as a standard for the accelerated curing of concrete.

(i) *Warm water method*

The specimens are immersed in water $1\frac{1}{2}$ to $3\frac{1}{2}$ hours after moulding.

Curing water temperature	55 ± 1 °C
Curing period	20 h ± 10 min

Demould and cool at 27 ± 2 °C for 1 h before test.

(ii) *Boiling water method*

Standard moist curing	23 h ± 15 min
Water temperature	100 °C
Curing period	$3\frac{1}{2}$ h ± 5 min
Cooling period	2 h

The actual correlation of accelerated test results to 28-day normally cured specimens depends upon the curing method adopted, the chemical composition of cement and the concrete mix proportions. Typical relationships are shown in Fig. 10.10. It is recommended that the actual relationship under given site conditions should be established using local concrete making materials and such relationships be continuously improved upon as more and more data become available progressively. In the absence of past records with local materials, the

Table 13.3 Number of Test Results for various Test Methods

Test Method	*Number of results at a location*
Standard core test	3
Non-standard small core test	9
Schmidt rebound hammer test	12
Ultrasonic Pulse velocity test	1
Pull-out test	6

recommended for beams, columns and walls, and surface zone tests on slabs must be restricted to soffits unless the top layer is first removed. Care must similarly be taken to discard material from the top 20 per cent (or at least 50 mm) of slab when extracting cores. Columns may be expected to be reasonably uniform except for a weaker zone in the top 300 mm or 20 per cent of their depth.

Like dry cubes, the cores generally yield strengths which are approximately 10–15 per cent higher than those tested in saturated condition.

13.3.1 Surface Hardness Method

The concrete surface hardness testing with SCHMIDT rebound hammer (developed by a Swiss engineer Ernst Schmidt) is most frequently used method worldwide for *non-destructive testing* (NDT)) of concrete structural element. In this method a test hammer hits the concrete at a definite energy and its rebound which is dependent on the hardness of concrete, is measured by test equipment. By reference to the *conversion table*, the rebound value furnishes the compressive strength of *in situ* concrete. A test hammer is used for—testing the strength of concrete, to check the uniformity of concrete and structural element; to detect the damaged zones. It provides useful information about the surface layer of concrete up to 30 mm deep, i.e. *covercrete*. The commonly used Schmidt test hammer weighs less than 2 kg and has an impact energy of 2.2 Nm. The spring controlled hammer mass slides on a plunger within a tubular casing as shown in Fig. 13.1. The plunger retracts against a spring when pressed against the concrete surface, and this spring is automatically released when fully tensioned, causing the hammer mass to impact against the concrete through the plunger. When the spring-controlled mass rebounds, it takes with it a rider which slides along a graduated scale and is visible through a small window in the side of the casing. The rider can be held in position on the scale by depressing the locking button. The equipment can be operated horizontally or vertically (up or down). The plunger is pressed strongly and steadily against the concrete surface to be tested at right angles, until the spring loaded mass is triggered from its locked position. After impact the *scale index* is read while the hammer is still in the test position. The measurement taken is an arbitrary quantity referred to as *rebound number*. A *calibration* or *reference curve* relating the compressive

However, the resistance of concrete may differ very widely depending on the local conditions and the environmental influences. An investigation of the measured values of resistivity permits the determination of spots where corrosion may occur. The combination of resistance and potential measurements furthermore improves the information about the corrosion condition of the rebars.

Permeability Tester

This equipment enables non-destructive measurement of the *permeability* of concrete structures. The permeability of concrete at the surface (cover concrete) has been recognized as a major factor determining the *durability* of concrete structures. This important property can be measured reliably at the building site or at the components.

The basic features of the commonly used typical equipment are *a two-chamber vacuum cell* and *a pressure regulator,* which ensure that an air flow at right angles to the surface is directed towards the inner chamber. This permits the calculation of the *permeability coefficient kT* on the basis of a simple theoretical model. The instrument measures the pressure increase as a function of time according to a specific sequence. The associated data is automatically stored by the display unit and the permeability coefficient *kT* and the depth of penetration *L* of the vacuum are calculated. The measurement takes 2 to 15 minutes, depending on the permeability of the concrete. In the case of *dry concrete,* the quality of the concrete cover can be read from the reference table using the *kT* value. In the case of *moist concrete, kT* is combined with the electrical resistivity of concrete ρ and the quality is determined from the reference monogram. The results of these site measurements, are comparable to those obtained by laboratory methods, such as *oxygen permeability, capillary suction, chloride penetration,* etc.

In the case of moist concrete, the measured permeability is lower, i.e. the concrete quality appears to be good. This effect can be corrected by using the electrical resistance ρ of the concrete. The concrete quality level is determined from *kT* and ρ from the reference monogram.

13.3.4 Partially Destructive Strength Tests

The test methods used to assess *in-situ concrete strength* which cause some *localized damage* sufficiently small to cause no loss in structural performance are called *partially destructive strength tests.* These methods which involve penetration, pull-out and pull-off techniques and use correlation charts for the estimation of strength, are not sensitive to as many variables as are rebound hammer or pulse-velocity testing. A major advantage of this type of testing is that strength can be estimated immediately as compared to core testing but the estimate may not be as accurate.

(b) *Tensile strength* The tensile strength may be measured by the *splitting test* on the core as explained in Section 8.2.3.

Factors Influencing Core Compressive Strength

The significant factors are outlined below.

Moisture and Voids The moisture condition of the core influences the measured strength; a saturated specimen has a value of 10 to 15 per cent lower than comparable dry specimen. It is therefore important that the relative moisture conditions of core and *in situ* concrete are taken into account while estimating the actual *in situ* concrete strengths. Voids in core will reduce the measured strength.

Length/Diameter Ratio of Core As *l/d* ratio increases, the measured strength will decrease due to the effect of specimen shape, and stress distribution during test. For establishment of relation between core strength and standard cube strength, a ratio *l/d* = 2.0 is regarded as the datum of computation.

Diameter of Core The diameter of the core may influence the measured strength and variability. Measured concrete strength decreases with the increase in the size of specimen; for sizes above 100 mm this effect will be small, but for smaller sizes this effect may become significant.

Direction of Drilling As a result of layering effect the measured strength of specimen drilled vertically relative to the direction of casting is likely to be greater than that for a horizontally drilled specimen from the same concrete; an average difference of 8 per cent has been reported in the literature.

Reinforcement Due to presence of reinforcement, the measured strength of concrete is underestimated up to 10 per cent. Reinforcement must therefore be avoided wherever possible, but in the case where it is present, the measured core strength may be corrected using Eq. (13.3).

$$\text{Corrected strength} = \text{measured strength} \times \left[1.0 + 1.5\left(\frac{\phi_r h}{\phi_c l}\right)\right] \qquad (13.3)$$

where

ϕ_r = reinforcement bar diameter

ϕ_c = Core diameter

h = distance of bar axis from the nearer end of core

l = core length (uncapped)

For the case of core having multiple bars, the corrected strength may be computed by, Eq. (13.4)

$$\text{Corrected strength} = \text{measured strength} \times \left[1.0 + 1.5\left(\frac{\Sigma\phi_r \cdot h}{\phi_c \cdot l}\right)\right] \qquad (13.4)$$

If the spacing of two bars is less than the diameter of the larger bar, only the bar with the higher value of (ϕ_r h) should be considered.

Calculation of Cement and Aggregate Contents

The cement content should be calculated separately from both the measured calcium oxide and soluble silica contents, unless the calcium oxide content of the aggregate is less than 0.50 per cent or greater than 35 per cent in which case results based on CaO are not recommended. In the latter case, if the soluble silica content of the aggregate is greater than 10 per cent the analysis should be undertaken to determine some other constituent present in a larger quantity in the cement.

The calculation of cement content is based on the assumption that the combined water of hydration is 0.23 times the cement content, and that 100 g of oven dried concrete consists of C g of cement + A g of aggregate + $0.23C$ g of combined water of hydration as given by Eq. (13.10).

$$100 = C + A + 0.23C = 1.23C + A \tag{13.10}$$

Thus if,

a = calcium oxide or soluble silica content of cement, per cent,

b = calcium oxide or soluble silica content of aggregate, per cent, and

c = measured calcium oxide or soluble silica content of the analytical sample, per cent;

then
$$\frac{c(1.23C + A)}{100} = \frac{aC}{100} + \frac{bA}{100}$$

or
$$(c - b)\,A = (a - 1.23c)C \tag{13.11}$$

Then from Eqs (13.10) and (13.11) the cement content

$$C = \frac{c - b}{a - 1.23b} \times 100 \text{ per cent (to nearest 0.10 per cent)}$$

and

Aggregate content,

$$A = \frac{a - 1.23c}{a - 1.23b} \times 100 \text{ per cent (to nearest 0.10 per cent)}$$

Thus the aggregate–cement ratio A/C may be obtained to the nearest 0.10. The cement content by mass is given by

$$\frac{C \times \text{oven dry density of concrete}}{100} \text{ kg/m}^3 \text{ to nearest 0.10 kg/m}^3$$

The above calculations require that an analysis of both the cement and aggregate to be available. If an analysis of cement is not available, OPC or RPC complying with the relevant Indian Standard Code may be assumed. If the two estimated cement contents are within 25 kg/m^3 or 1 per cent by mass, the value is adopted. Thus, method of analysis suffers from the drawback that it cannot be used for concretes which contain aggregates or admixtures or additives such as fly ash or pozzolanas, which liberate soluble silica under the conditions of test.

cellular construction, by entraining large quantities of air, by using *no-fines* concrete and lightweight aggregates which are made lighter by introducing internal voids during the manufacturing process.

The term no-fines indicates that the concrete is composed of cement and coarse aggregate (commonly 10 or 20 mm grading) only, the product has uniformly distributed voids. Suitable aggregates used are natural aggregates, blast-furnace slag, clinker, foamed slag, sintered fly ash, expanded-clay, etc.

Lightweight aggregate is a relatively new material. For the same crushing strength, the density of concrete made with such an aggregate can be as much as 35 per cent lower than the normal weight concrete. In addition to the reduced dead weight, the lower modulus of elasticity and adequate ductility of lightweight concrete may be advantageous in the seismic design of structures. Other inherent advantages of the material are its greater fire resistance, low thermal conductivity, low coefficient of thermal expansion, and lower erection and transport costs for pre-fabricated members. For pre-fabricated structures a smaller crane is required or the same crane can handle larger units due to reduction in dead weight. For cast *in-situ* structures, its smaller dead weight makes foundations less expensive.

Moreover, continued extraction of conventional dense natural aggregate from the ground is bound to be accompanied by severe environmental problems leading to deterioration of the countryside and its ecology. On the other hand, use of manufactured aggregates made of industrial wastes (slags, etc.), preferably those containing sufficient combustible materials (pulverized fuel ash) which provide all or most of the energy for their production, may help in alleviating the problem of disposal of industrial waste.

14.2.1 Light-weight Aggregates

Light-weight aggregates may be grouped in the following categories:

(i) Naturally occurring materials which require further processing, such as expanded-clay, shale and slate, etc.
(ii) Industrial by-products, such as sintered pulverized fuel ash (fly ash), foamed or expanded-blast-furnace slag.
(iii) Naturally occurring materials, such as pumice, foamed lava, volcanic tuff and porous limestone.

Aggregates Manufactured from Natural Raw Material

The artificial lightweight aggregates are mainly made from clay, shale, slate or pulverized-fuel ash, subject to a process of either *expansion (bloating)* or *agglomeration*. During the process of expansion the material is heated to fusion temperature at which point pyroplasticity of material occurs simultaneously with the formation of gas. Agglomeration on the other hand occurs when some of the material fuses (melts) and various particles are bonded together. Thus to achieve

Table 14.1 Physical Properties of Light-weight Aggregate

Aggregate Type	*Particle shape, and surface texture*	*Density (kg/m^3)*	*Bulk density (kg/m^3)*	*24-hour water absorption capacity (per cent)*	*Typical concrete*	
					Compressive strength (MPa)	*Unit weight (kg/m^3)*
A. *Aggregate for structural concrete (f_{ck} > 15 MPa)*						
Expanded-clay	Rounded and slightly rough particles.	*Coarse* 600 to 1600 *Fine* 1300 to 1800	300 to 900	5 to 30	10 to 60	1000 to 1700
Expanded-shale and slate	Often angular and slightly rounded, smooth surface.	*Coarse* 800 to 1400 *Fine* 1600 to 1900	400 to 1200	5 to 15	20 to 50	1300 to 1600
Fly ash	Similar to expanded-clay.	1300 to 2100	600 to 1100	20	30 to 60	1500 to 1600
Foamed-blast-furnace slag	Irregular angular particles with rough and openpored surface.	1000 to 2200	400 to 1100	10 to 15	10 to 45	1800 to 2000
Sintered-colliery waste	Angular with open-pored surface.	1000 to 1900	500 to 1000	15	10 to 40	1400 to 1600
B. *Aggregate for low-medium strength concrete (3.5 to 15 MPa)*						
Pumice	Rounded particles with open-textured but rather smooth surface.	550 to 1650	350 to 650	50	5 to 15	1200 to 1600
C. *Aggregate for low strength concrete (0.5 to 3.5 MPa)*						
Perlite	Rounded and of angular shape and rough surface.	100 to 400	40 to 200	—	1.2 to 3.0	400 to 500
Vermiculite	Cubical	100 to 400	60 to 200	—	1.2 to 3.0	300 to 700

service areas, container yards, material-handling yards, apron and carriageway of airports, for binder course of expressway and heavily trafficked roads.

Construction of Roller Compacted Concrete Pavement (RCCP)

RCC is *weigh-batched* and mixed in a *continuous mixing pug mill* or a normal mixer such as used for soil–cement treated base or asphalt concrete construction. The pavement is initially constructed in lifts of 150–200 mm for a pavement thickness of more than 400 mm with an elapse time of 30 minutes to 2 hours between the lifts. A common paver can place and compact a 150 mm lift to 95 per cent of the specified density in only two or three passes of roller. The lower lift requires more compactive effort with more passes of the roller, to achieve the desired density. If a smooth pavement surface is not obtained, a layer of asphalt may be used to cover the surface and smooth out the roadway.

Vee-Bee consistometer apparatus with some modification can be used to evaluate *RCC consistency. Curing* of RCCP is accomplished by keeping the surface of the pavement wet for 7 days. Water spray or fine mist is most appropriate and is commonly used. Wet sand can also be used for curing RCC.

Due to very low *drying shrinkage* of RCC as compared to PCC, the *contraction joints* are provided at a spacing of 10 to 20 metre whereas the spacing between contraction joints is 4.5 metre in PCCP.

RCCP as a result of the high stability achieved by the mineral skeleton formed by the aggregates after compaction can be opened to traffic almost immediately. Furthermore, the *stability* of the mineral structure allows high volume of *mineral binders* to be used than that used in the vibrated concrete pavements. The total thickness of an RCCP structure is much less than an asphalt/gravel pavement of the same load carrying capacity. Due to its advantages as a comparatively low cost and durable paving material, RCC is also emerging as a common base for conventional highway and street pavements. Thus RCC is a material that has the longevity of concrete at the price of asphalt.

Dam Construction

Applications of RCC technology to dam construction has enabled the concrete to play an increasingly important role in the construction and rehabilitation of dams. Typically, in RCC dam construction, no-slump concrete mix is spread in 300 mm thick lifts. The RCC mixture is usually mixed at a temporary plant erected near the dam site and transported by conveyor belt, front-end loader, or dump truck to the placement site. The newly placed lift of RCC, is compacted with a vibratory roller. Continuous placement of RCC is desirable on dam projects to minimize *cold joints* between the horizontal lifts that could inhibit *bonding* of the concrete lifts to each other. RCC can be used to overlay the downstream slope of the existing embankment dam to protect the dam from erosion if the structure is overtopped by water.

urbanization. The quantities increase tremendously during massive reconstruction after devasting earthquakes. Disposal of such materials is difficult in view of the scarcity of suitable dumping grounds, and meeting the environmental requirements. Hence, the broken concrete is increasingly being recycled. Recycled concrete is simply the old concrete that has been removed from buildings, foundations, pavements and other structures, and crushed to the specified size.

The basic requirement for recycled aggregates for concrete construction is that the original concrete shall be sound, hard, normal weight concrete. As a rule, recycled concrete aggregate of distinctly different qualities shall be used separately. The cement mortar attached to the recycled aggregate primarily determines the performance of recycled concrete. Finishing materials, reinforcing bars and other embedded material, if any, in the original concrete shall be removed in the best possible way. Recycled aggregates shall not contain excessive amounts of dirt, plaster of Paris or gypsum and other injurious foreign matter like wood and asphalt which may adversely affect recycled aggregate concrete and steel used therein.

Production The basic method of the recycling is one of crushing the debris to produce a granular product of given particle size and then reprocessing and screening, the degree of which depends on the level of contamination and the application for which recycled aggregate is produced. Recycled aggregates normally have more angular shape and more coarser surface and exhibit more or less similar particle size distribution as that for natural aggregate. It has been reported that the properties of aggregates from demolished concrete are affected more by the method of crushing than by the properties of the original concrete.

The specific gravity and bulk density of the recycled aggregate are lower than the original aggregate and water absorption is 5 per cent higher, due to the presence of low density mortar. The relative densities of crushed concrete fine and coarse aggregates are 2.1 and 2.3, respectively. Recycled aggregates produced from good quality concrete can be expected to fulfil the requirements for the Los Angeles abrasion loss percentage, crushing and impact values. Before using the crushed concrete as an aggregate in concrete for roads and buildings, investigations should be carried out for establishing its suitability and feasibility especially in helping the solid-waste disposal problem.

Properties of Recycled Aggregate Concrete The concrete produced with recycled aggregate loses its workability more rapidly than the conventional concrete, because recycled aggregate is more porous than natural aggregate. Thus concrete with recycled concrete aggregate may require 5 to 10 per cent more mixing water to achieve the same workability as the gravel concrete. If both fine and coarse aggregates are recycled aggregates, around 15 per cent more free water is required.

An air entraining and water-reducing admixture shall be incorporated into fresh recycled aggregate concrete mix. The air content of recycled aggregate concrete may be slightly higher than that of conventional aggregate concretes,

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14.9.3 Properties of Ferrocement

Though ferrocement is often considered to be just a variation of conventional reinforced concrete which may be true for the ferrocement with small quantities of reinforcement, however, it is not true for the quantity of reinforcement provided in most of the applications. Moreover, a system of construction using layers of closely spaced wire mesh separated by skeleton bars and filled with cement mortar presents all the mechanical characteristics of a *homogeneous material.*

Tensile strength of ferrocement depends mainly on the volume of reinforcement in the direction of force and the tensile strength of the mesh. The tension behaviour may be divided into three regions, namely, pre-cracking stage, post-cracking stage and post-yielding stage. A ferrocement element (member) subjected to increasing tensile stresses behaves like a linear elastic material till the development of *first crack* in the matrix. Once the cracks have developed the material enters the stage of *multiple cracking* and this stage continues up to the point where wire meshes start to yield. In this stage number of cracks keep on increasing with an increase in tensile stress without any significant increase in crack width. With the yield of reinforcement, the composite enters the stage of *crack-widening.* The number of cracks remains essentially constant and the crack widths keep increasing. The behaviour is primarily controlled by the reinforcement bars.

In the elastic pre-cracking stage the modulus of ferrocement composite E_c can be expressed in terms of modulii of mortar and reinforcement E_m and E_r, respectively, and volume fraction of reinforcement in longitudinal direction, V_{r_1}:

$$E_c = (1 - V_{r_1})E_m + V_{r_1}E_r \approx E_m + V_{r_1}E_r = E_m(1 + \eta V_{r_1})$$

where $\eta = E_r/E_m$.

During the multiple cracking stage, the contribution of mortar to the stiffness of composite is negligible. Hence the stiffness of composite is approximately represented by:

$$E_c = V_{r_1}E_r$$

The value of E_r may be substantially different for woven-mesh from that for a welded-mesh. It has been noticed that higher the volume of reinforcement and smaller the diameter of wires, longer is multiple cracking stage with a larger number of cracks developed in the same *gauge length.*

An inverse relationship between the first crack strength and average wire spacing based on linear elastic fracture mechanics has been established. The load carrying capacity of ferrocement is correlated with the *specific surface area of reinforcement*, *S*, which is defined as the total surface area of the wires in contact with cement mortar divided by the volume of composite. However, it should be noted that some investigators have used the surface area of the wires in the load resisting direction S_L only. The specific surface area has been found to influence the *first crack load* in tension, as well as the *width and spacing of cracks*. For

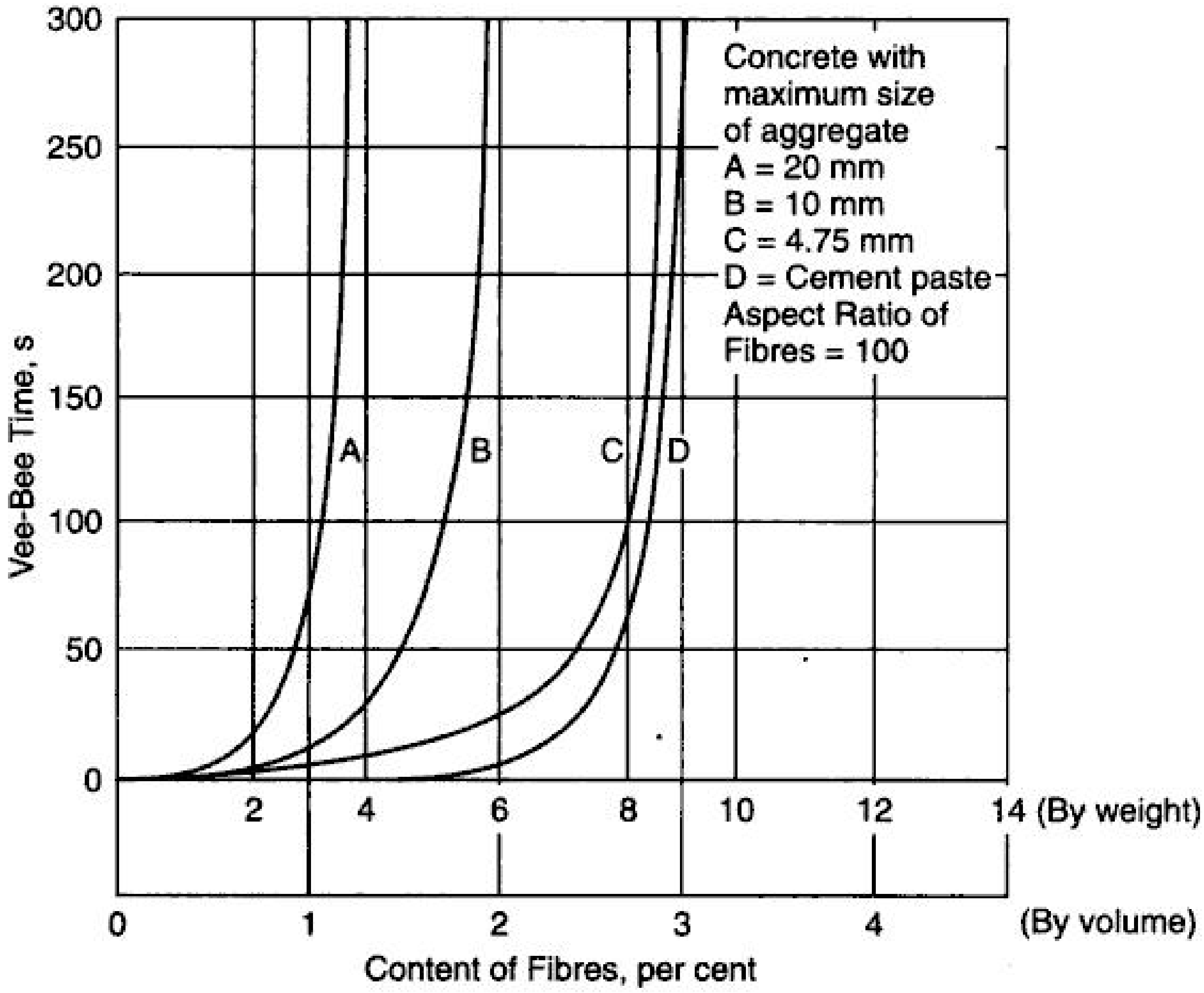


Fig. 14.9 Effect of aggregate size on workability of fibre reinforced concrete

14.10.3 Concrete Matrix

The cement required is OPC or PPC conforming to IS: 269–1989 or IS: 1489–1991, respectively. The aggregates are usually crushed quartz conforming to IS: 383–1970. A fibre-reinforced concrete requires a considerably greater amount of fine material than plain concrete so that it may be conveniently handled and placed. To be fully effective, each fibre needs to be completely embedded in the matrix and this determines the proportion of fine to coarse aggregate. Fibre concrete, therefore, generally requires a greater proportion of cement paste than conventional concrete for handling and placing by using the equipment meant for ordinary concrete. Normal concrete contains 25 to 35 per cent of cement paste of the total volume of concrete, and fibre-reinforced concrete requires paste content of the order of 35 to 45 per cent of the total volume of concrete, depending upon the fibre geometry and fibre volume.

14.10.4 Different Types of Fibres

The most commonly used man-made fibres have been *steel* and *polypropylene*, principally in concrete, and glass, principally in cement mortar for thin section applications. Properties of some of the commonly used fibres are given in Table 14.8.

Steel-fibre Reinforced Concrete

A number of steel-fibre types are available as reinforcement. Round steel fibres, the commonly used type, are produced by cutting round wires into short lengths.

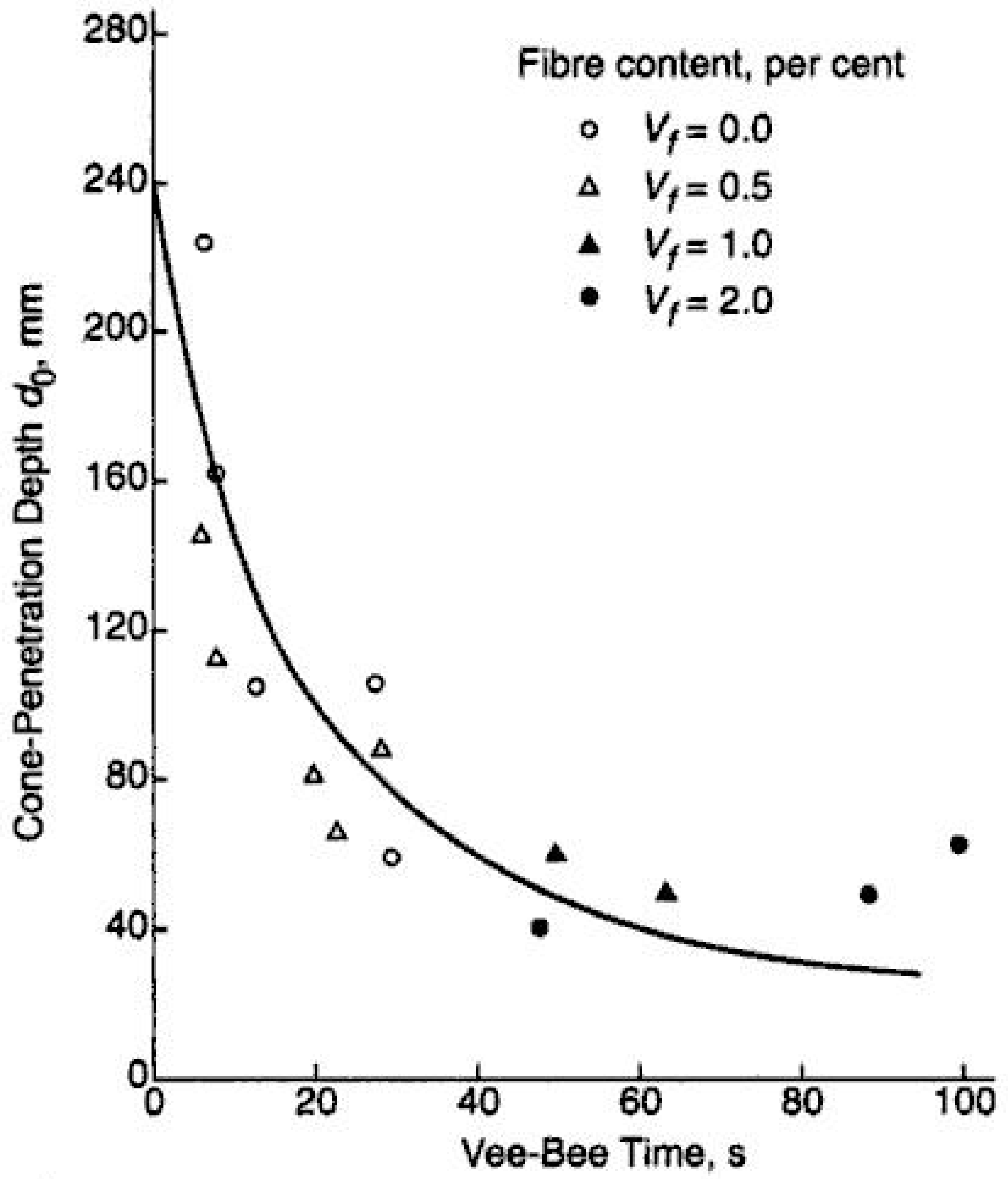


Fig. 14.12 Relation between Vee-Bee time and cone penetration depth

while being suitable even for low workability mixes for which conventional tests fail. The test data have a consistent relationship with the other measures of workability given by slump, Vee-Bee time, compacting factor and ACI inverted cone method. The relationships between workabilities measured by different methods are given in Figs 14.11 to 14.13.

Factors affecting workability The factors having a predominant effect on the workability are: aspect ratio (l/d) and fibre volume concentration. Long thin fibres ($l/d > 100$) tend to mat together while short stubby fibres ($l/d < 50$) cannot interlock and can be dispersed by vibration. A *minimum fibre volume concentration* called *critical concentration* is needed to increase the strength. The critical concentration is generally inversely proportional to the aspect ratio. For $l/d = 100$, a volume concentration of 0.5 per cent for flexural strengthening and 1.7 per cent for tensile strengthening is required. However, for a 1.7 per cent concentration, an adequate workability can be obtained only with cement paste, and cement–sand mortar; whereas a 0.5 per cent concentration can perfectly be provided in the concrete. Thus there is a restricted range of practical fibre reinforced concrete with improved strengths. The performance of hardened concrete depends upon the *specific fibre surface (SFS)* which is defined as the total surface area of all the fibres present within the unit volume of the composite. The specific fibre surface depends upon the fibre volume concentration, fibre size and aspect ratio. For a fibre volume concentration of V_f per cent, the specific fibre surface in a unit volume of composite is given by

$$SFS = n(\pi dl)$$

on this observation, following general equation for predicting the ultimate flexural strength of the fibre composite has been proposed.

$$f_c = C f_m (1 - V_f) + D (V_f L/d_f)$$

where

f_c is the ultimate strength of the fibre composite

f_m is the maximum strength of the plain matrix (mortar or concrete)

C and D are constants which can be determined experimentally.

For plain concrete $C = 1$ and $D = 0$. The constant C accounts for the *bond strength of the fibres* and *randomness of fibre distribution.* The values for the constants C and D have been proposed as 0.95 and 4.95 for the ultimate flexural strength of steel fibre-reinforced concrete and 0.85 and 4.25 for its first cracking strength.

The increasing fibre-reinforcing index ($V_f L/d_f$) has a positive influence on performance because of the improved resistance to pull-out of the fibres from the matrix. The maximum quantity of hooked-end fibres that can be added without causing balling is limited to 1.0 per cent by volume. Compared to plain concrete, the addition of fibres increase the first cracking strength by 15 to 75 per cent and static flexural strength (characterized by modulus of rupture) by 15 to 30 per cent for the values of $V_f L/d$ from 40 to 120 (a practical limit from workability consideration). Compared on equal basis of 1.0 per cent by volume, the hooked-end steel fibres contribute the highest increase, and the straight fibres provide the least increase in the above-mentioned properties.

The *ultimate load carrying capacity* of fibre-reinforced concrete beam depends mainly on the adequacy of bond. In the absence of excellent *interfacial bond*, the fibres are debonded as soon as the load is transferred to them immediately after cracking of the matrix and the ultimate load will not be greater than the ultimate load of beams without fibre reinforcement. If the bond is excellent, the fibres can withstand loads even after the cracking of the matrix, and this results in an increase in the *ultimate strength*. An improvement in bond can be achieved by the introduction of *indented, crimped*, or *bent fibres.*

The polyester and polypropylene fibres significantly increase the flexural toughness and the post-peak resistance of concrete. These improvements continue as fibre volume increases, except in ultimate strength, for which it starts to decrease beyond fibre volume of 0.35 per cent. The addition of silica fume enhances toughness and post-peak strength of plastic fibre concrete.

Slurry infiltrated concrete (SIFCON) when used over the reinforced concrete beams leads to ductility indexes exceeding three times those obtained without it. Crack widths and spacing are more than an order of magnitude smaller than in conventional reinforced concrete. There is no need for stirrups in flexural members with SIFCON matrix.

(iv) ***Shear Strength*** The enhancement of shear strength of fibre reinforced high strength concrete is of the order of 60 per cent with steel and 15 per cent

(d) Fibre reinforcement increases the shear strength of concrete. As a consequence punching shear strength of slabs is increased and sudden punching failure may be transformed into gradual ductile one.

Mix Design for Steel-Fibre Reinforced Concrete

The mix should contain minimum fibre content and maximum aggregate for the specified strength and workability. The *cement paste content* depends upon three factors

(i) Volume fraction of fibres.
(ii) Shape and surface characteristics of fibres, i.e. specific fibre surface.
(iii) Water-cement ratio.

For the commonly encountered SFRC mixes, the following range of parameters is associated:

Cement content	300 to 500 kg/m^3
Water–cement ratio	0.45 to 0.60
Ratio of sand to total aggregate, per cent	50 to 100
Maximum size of aggregate	10 and 20 mm
Fibre content	1.0 to 2.5 per cent
Fibre–aspect ratio	50 to 1000

Mix Design Procedure Following are the steps involved in the mix design of fibre reinforced concrete.

(i) Corresponding to the required 28-day field flexural strength of steel fibre–reinforced concrete, the design strength for laboratory mix is determined.
(ii) For fibres of known geometry and for stipulated volume fraction, the water–cement ratio is selected between 0.45 and 0.60.
(iii) Depending on the maximum size of aggregate and fibre concentration, the paste content is determined by mass.
(iv) The fine-to-coarse aggregate ratio varies from 1:1 to 1:3, a ratio of 1:1.5 is a good start for a volume percentage of fibre up to 1.5 and length of fibre up to 40 mm.
(v) For the water–cement ratio and paste content determined in the steps (ii) and (iii), respectively, the cement and water contents may be worked out.
(vi) The fibre content (by mass) is calculated by taking the density of fibres as 7850 kg/m^3.
(vii) The total quantity of the aggregate is determined from:

$$W_A = W_{FRC} - (W_W + W_C + W_F)$$

where W_A, W_{FRC}, W_W, W_C and W_F are the masses of total aggregate, fibre reinforced concrete, water, cement and fibres, respectively.

i.e. the major parameters affecting monomer loading are the moisture and the air in the voids in concrete. The total or in-depth polymer impregnation of concrete, therefore, involves the following states.

(i) Availability of well-designed cement concrete, which is adequately moist cured with optimum strength.
(ii) Removal of moisture by drying the concrete by heating to develop surface temperatures of the order of 120 to 150 °C. The small specimens can be heated in an air-oven. For large *cast–in situ* surfaces a thick blanket of sand (usually 10 mm thick) can be used to prevent a steep *thermal gradient.* Infra-red heaters may be used. About 6 to 8 hours of heating is required to expel a large part of the free water in the concrete.
(iii) Cooling of concrete surfaces to safe levels (about 35 °C) to avoid flammability.
(iv) Removal of air by subjecting the dry concrete specimen to vacuum in a process vessel. The degree of vacuum applied and the duration have significant influence on the quantity of monomer that can be impregnated and therefore, on the depth of impregnation.
(v) Application of monomer by soaking the concrete specimen in it for a sufficiently long time to achieve the desired depth of penetration. The soaking time depends on the viscosity of monomer, preparation of the specimen prior to soaking and the characteristics of the concrete. To reduce the time required to achieve a desired depth of monomer penetration, external pressure using nitrogen gas or air is generally employed.
(vi) Covering the surface with a *plastic sheet* to prevent evaporation of monomer.
(vii) Polymerization by heating the catalyzed monomer to the required temperature levels (usually between 60 and 150 °C depending upon the type of monomer) also called thermal catalytic technique. The heating can be done under water, or by low pressure steam injection, or by infra-red heaters or in an air-oven. Depending on the polymer, 2 to 6 hours are required for this stage. The heating decomposes the catalyst and initiates the polymerization reaction. This reaction is called a *thermal catalytic reaction.* When monomer has penetrated into concrete, polymerization can also be initiated using ionizing radiation such as gamma rays. The polymers, when fully polymerized or cross-linked, are solids occupying the volume in which they have been impregnated. As such, at the impregnation stage, the polymer has to be in a prepolymer liquid form, generally called monomer. The state of polymerization of monomers, or of prepolymer resins, is brought about also by adding initiators, and cross-linking agents.

Polymers can be broadly categorized as *thermoplastics* and *thermosetting resins.* Thermoplastics soften at an elevated temperature (usually between 100

aggregate and the mixture is thoroughly mixed by hand or machine. The thoroughly mixed polymer concrete material is cast in moulds of wood, steel or aluminium, etc. to the required shape or form. Mould releasing agents can be added for easy demoulding. This is then polymerized either at room temperature or at an elevated temperature. The *polymer phase* binds the aggregate to give a strong composite. Polymerization can be achieved by any of the following methods.

(i) Thermal–catalytic reaction
(ii) Catalyst–promoter reaction
(iii) Radiation

In the first method, only the catalyst is added to the monomer (thermoplastic) and polymerization is initiated by decomposing the catalyst by the application of elevated temperatures up to 90 °C. Typical catalysts used for different monomer systems include, benzoyl peroxide, methyl-ethyl-ketone peroxide, benzenesulphonic acid, etc. In the second method, a constituent called *promoter* or *accelerator* is also added, which decomposes the catalyst or accelerates the reaction, at the ambient temperature itself. Typical promoters include cobalt naphthanate, dimethyl-p-toluidine, ferric chloride, etc. Some promoters ensure polymerization at the ambient temperature within an hour. Gamma radiation is applied in the radiation polymerization method. Depending on the method of polymerization and the other conditions, polymerization takes place within a period ranging from a few minutes to a few hours. Special precautions are to be taken in handling and cleaning because the monomers are highly inflammable. Fire safety precautions are to be observed. A thoroughly dry aggregate system is to be used as the monomers may not polymerize in the presence of moisture. Moreover, the catalyst and promoter should never be added to each other as it will result in an explosion. Some of these materials are toxic and are carcinogenic, and have to be handled with extreme care.

The polymer systems which have been successfully used for polymer concrete include methyl-methacrylate, polyester-styrene, epoxy-styrene, styrene and furfuryl acetone. Others are furane, acrylic, polyurethane, urea formaldehyde and phenol formaldehyde, etc. The design considerations for polymer concrete are:

(i) The *binder content* to fill the voids of the aggregate system. Smaller the polymer content greater is the economy.
(ii) *Workability* for easy mixing and placing of cement concrete without bleeding and segregation.
(iii) *Film forming ability* of the polymer, and *bonding* with the aggregate surface to transmit load forces.
(iv) *Economic curing (cross-linking) times* and temperatures.
(v) *Durability* in environments to which the polymer concrete composite is exposed.

CHAPTER
16

High Performance Concretes

16.1 INTRODUCTION

Over the years, to produce concrete with improved properties such as *higher early strength,* there have been gradual increase in the fineness and C_3S content in the general-purpose Portland cement. An increase in C_3S content as compared to the C_2S content has resulted in a more rapid hydration and more rapid development of strength. Since concrete mixtures are proportioned on the basis of 28-day compressive strength, this change has gradually resulted in decreasing the cement content and increasing the water–cement ratio of concrete mixtures with a given consistency. On the other hand a reduction in particle size generally results in increased hydration and a higher compressive strength. However, the greater fineness leads to an increased water demand and a more rapid *heat generation* in concrete structures. In recent years, further improvements in concrete properties have been achieved in the so-called *high performance concrete* (HPC) by improvements involving a combination of *improved compaction, improved paste characteristics* and *aggregate–matrix bond,* and *reduced porosity.* In these systems, a substantial *reduction in water-to-cement ratio* is achieved through the use of *superplasticizers,* further enhancements of some properties have been obtained through the addition of *mineral micro fillers* (*supplementary cementitious* or *pozzolanic materials* such as *silica fume* and *fly ash*). One consequence of lower water–cement ratio is that not all of cement in the concrete mixture participates in the hydration reactions. It is generally accepted that for water–cement ratios less than approximately 0.42, unreacted cement will be present regardless of the *particle size distribution* of cement. Compared to conventional or normal concrete high early-strength concrete containing high cement content tends to crack more easily due to lower creep, higher *thermal* and *drying shrinkages,* and higher *elastic modulus.* There is a close inverse relationship between high strength and early-age cracking in concrete. The deterioration on the other hand is closely associated with

Chemical Interaction

Deterioration may be caused by the chemical reaction between the hardened cement constituents of concrete and the chemicals of a solution. The reaction products formed may be either water soluble and may get removed from the internal structure of concrete by a diffusion process, or the reaction products if insoluble in water may get deposited on the surface of concrete as an amorphous mass having no binding properties, with the result that it can be easily washed out from the concrete surface.

Acids first react with free lime of concrete forming calcium salts and later on attack the hydrosilicates and hydroaluminates forming the corresponding calcium salts, whose solubility will govern the extent of deterioration caused to the concrete.

As can be observed from Fig. 15.2, the hydrochloric acid corrodes the concrete to a greater extent in comparison to the sulphuric acid at low concentrations because H_2SO_4 forms a less soluble $CaSO_4$ on reacting with lime of concrete, which seals the pores of concrete for further permeation and offers resistance to acid corrosion. But at higher concentrations of H_2SO_4, the concrete strength is reduced due to the accumulation of $CaSO_4$ in the pores and the development of internal stresses.

Crystallization

Concrete may get deteriorated by the accumulation or crystallization of salts in its pores, which leads to the development of internal stresses and formation of

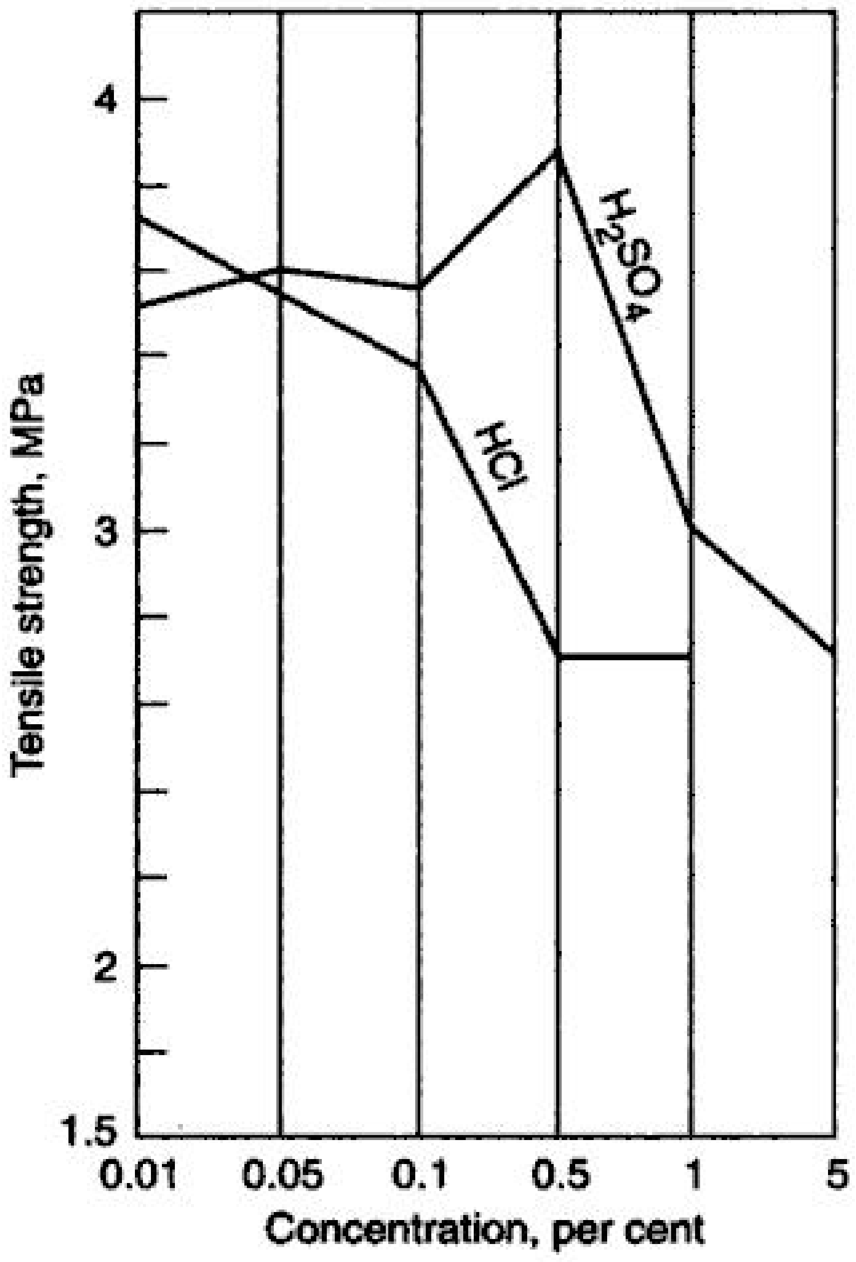


Fig. 15.2 Deterioration of concrete due to acid attack

quantity of hydrogen is plated out at adjacent surfaces of the metal. This thin invisible protective film of hydrogen around the cathode inhibits further progress of corrosion reaction, unless the hydrogen film is removed in some manner.

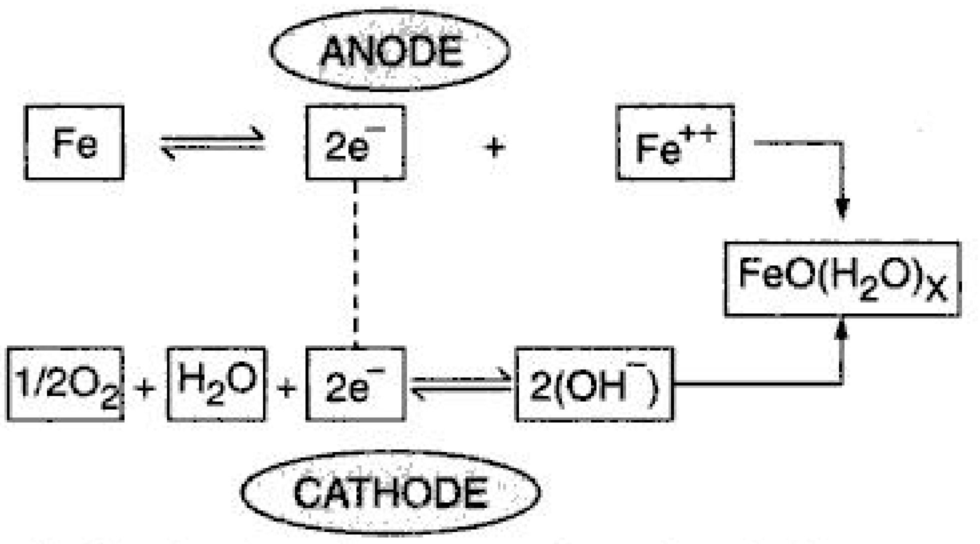
The destruction of hydrogen film may take place in one of the two ways: (i) oxygen depolarization at the cathode, and (ii) evolution of hydrogen as a gas. These processes called cathodic reactions are usually represented by:

$$\frac{1}{2}O_2 + H_2O + 2e \rightarrow 2OH^-$$

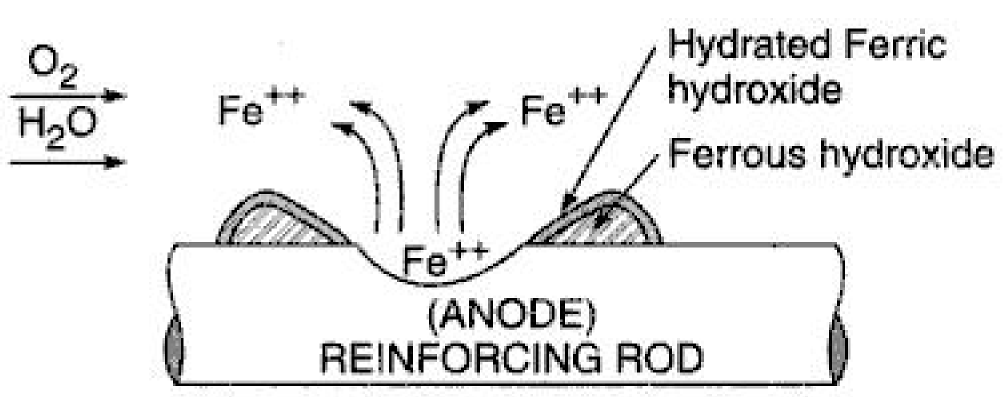
$$2H^+ + 2e^- \rightarrow H_2$$

These cathodic reactions which are often called the secondary reactions control the rate of corrosion of the structural steel. The chemical reactions are depicted in Fig. 15.3. Therefore, any environmental condition which influences these reactions will influence the rate of corrosion. Since cathodic depolarization is dependent on the concentration of dissolved oxygen next to the metal, it is influenced by the degree of aeration, temperature, salt concentration, etc.

The secondary reactions permit the primary reaction to proceed with the accumulation of ferrous ion in the solution which in the presence of water and oxygen are oxidized and precipitated as rust. However, two stages of oxidation may exist depending upon the availability of oxygen. The products of the first stage namely ferrous hydroxide is more soluble than the second stage product, i.e. hydrated ferric hydroxide. The first is usually formed directly at the metal surface and is converted to the latter at a little distance away from the surface where it is in contact with more oxygen as shown in Fig. 15.3(b).



(a) Chemical reactions at anodic and cathodic areas



(b) Formation of rust at anodic region

Fig. 15.3 Chemical reaction resulting into the formation of rust

cracking of concrete structures exposed to severe environmental conditions. To overcome various problems encountered in the field and to achieve better and better performance, a number of further process improvements have been suggested.

It is to be realized that durability of concrete cannot be enhanced without a holistic approach considering the cracking–durability relation. The root causes of many durability problems can be traced to this kind of reductionistic approach ignoring the *cracking–durability relationship* and over emphasizes on the *strength–durability relationship.* A change-over to a holistic approach to control cracking in concrete structures is necessary to create a much closer working relationship between the structural designer, materials engineer, and construction personnel.

There has been significant interest and development in the use of short discrete or continuous fibre reinforcement for improving the pre- and post-cracking behaviour of cementitious composites and/or concrete. *Fibre Reinforced Polymers* (FRP) or sometimes also referred to as *fibre reinforced plastics* are increasingly being accepted as an alternative for uncoated and epoxy-coated steel reinforcement for prestressed and non-prestressed concrete applications.

Although high performance concretes are made with the same basic components as the normal concrete, their much higher qualitative and quantitative performances make them new materials. On the basis of their use, they offer different advantages such as enhanced durability, reduced permeability, higher strength, etc. at an economical cost. Nevertheless, usually the end-product characteristics do not stand-alone but are supplemented by other clauses or sections. The lack of adequate HPC provisions in various national codes of practice is the greatest deterrent in its extensive use. The development of cost-effective state-of-the-art procedures for producing, evaluating, and designing with HPC will instil confidence that the user stated level of performance for each performance characteristic can be reliably achieved in the field. There is no bad concrete; it is just that the concrete is inappropriate for the particular application. The goal is to use concrete with characteristics at appropriate levels to ensure satisfactory performance for the intended service life. A concrete does not perform as desired, when either the specifications are inadequate or they are not followed. Modern QC/QA procedures greatly increase the likelihood that specifications are met when followed. A concrete produced to comply strictly with code requirements should be an HPC. The intent of HPC is not to produce a high cost product, but simply to provide the means for producing concrete that will perform satisfactorily with only a reasonable maintenance cost for the intended service life.

In spite of increasing emphasis being placed on the performance of concrete, the construction industry still operates in a *prescriptive* rather than *performance-based environment* where there is no need for prescribing the mixture ingredients

flexural strengths and *very high ductility* with a *reduced volume of fibres* than SIFCON.

The use of SIMCON appears to be very promising for at least two reasons. First, the very high volume of fibres required to provide significant increases in mechanical properties with SIFCON may not provide economic justification in a large number of practical applications. Secondly, in situations where normal FRC may be economically justified, such as in the pavements, the addition of fibres to the mix and the placement of the fibre-reinforced mix require special care, and considerable extra time and expense. SIMCON overcomes many of these limitations since the fibre mat, normally delivered in large rolls, can be laid out by hand and the slurry simply pumped into place. The use of SIMCON permits fabrication of thin, complex shapes with very high ductility and flexural strength.

Another interesting and useful development in FRC construction has been to provide non-metallic fibres in small, cylindrical bundles, approximately 50 mm high (the length of the fibre) and 55 mm in diameter, wrapped in a water soluble compound. This permits easy addition of the fibres, by hand, into the mixing drum of a mixer, either during charging or at the job site. The wrapper disintegrates, allowing the fibres to disperse into the concrete mixture with little *balling* or *segregation*. Quality control is improved since this makes the quantity of fibres added easy to determine and easy to check, and minimizes the problems in dispersion in the mixer. Further, production rates are maintained with little additional effort.

16.3.3 Other Types of HPC

Super-Workable or Self-Compactable or Flowable Concrete This type of HPC has been developed for the use in situations where vibration is difficult or impossible and where reinforcing steel is highly congested. This *super-workable concrete* has been extensively used in both—bridge structures and buildings

There are well-established procedures for selection of materials, mix proportioning and quality curing for the production of high performance concrete. However, much less control is exercised on the concrete placement. In this regard, the development of *self-compactable non-segregating* concrete utilizing HRWR is a significant step towards achieving high performance concrete through *automation*.

Low Heat of Hydration Concrete A number of different combinations using Portland cement, finely ground burnt limestone powder, fly ash, GGBFS, silica fume, etc. can be used for a *low heat of hydration* application. Low heat of hydration mixes typically exhibit considerably *extended setting times*, with final set at about 20 hours, and *low early strengths*. This is due both to use of large quantities of HRWR and mineral additives or blend cements. A mix with large quantity of HRWR in conjunction with a *blended cement* composed of 30 per

engineer can specify a mixture to yield a desired concrete service life. Each parameter can be independently specified by a grade. The grades start at low performance levels and small enough increments are defined to enable to specify higher quality concrete incrementally. The strength grades start at a performance level that is easily attainable and spans to a superior grade. The definition covers all grades of concrete that can be readily used by the concrete industry.

Table 16.2 does not represent a comprehensive list of all characteristics that a good concrete should exhibit. It does list characteristics that can quantifiably be divided into different performance groups. Other characteristics should be checked. For example, HPC aggregates should be tested for detrimental alkali–silica reactivity. Due consideration should also be paid to (but not necessarily limited to) acidic-environments and sulphate-attacks. For a given high performance concrete, the mix design is specified by a grade for each desired performance characteristic. For example, a concrete may perform adequately at Grade-4 in strength and elasticity, Grade-3 in shrinkage and scaling resistance and Grade-2 in all other categories.

In view of diversity of strength needs and the variation of strengths used in practice, SHRP has suggested a wide range of strength grades starting at 40 MPa for Grade-1 to greater than 95 MPa for Grade-4. Bridge engineers currently specifying strengths less than Grade-1 may begin the transition to a higher durability and strength concrete by specifying minimum HPC performance grades. The highest level is specified to define the state-of-the-art in highway concrete usage. Static modulus of elasticity grades ranges from a low of 28 GPa for Grade-1 to greater than 50 GPa for Grade-3.

The *general field environment* to which the concrete may be exposed during its working life has been classified in IS:456–2000 into five levels of severity, i.e. mild, moderate, severe, very severe and extreme as described in Table 16.3. Compliance to specified stipulations regarding *minimum cement content*, *maximum water–cement ratio* and *minimum grade of concrete* will ensure adequate performance levels in the given exposure conditions.

IS:456–2000 stipulations for following exposures are:

Freezing–Thawing Where freezing–thawing actions under wet conditions exist, enhanced durability can be obtained by the use of suitable air-entraining admixtures. When concrete lower than grade M50 is used under these conditions, the mean total air content by volume of the fresh concrete at the time of delivery into the construction should be:

Nominal maximum size of aggregate, mm	20	40
Entrained air, per cent	5 ± 1	4 ± 1

Since air-entrainment reduces the strength, suitable adjustments may be made in the mix design for achieving required strength.

practices. A substantial reduction in water requirement can be achieved by using a well-graded aggregate.

(ii) Paste Characteristics It is generally accepted that the most important parameter affecting concrete strength is the *water–cementitious material* ratio, sometimes referred to as the *water–binder ratio*. The strength of concrete is dependent largely on the *capillary porosity* or *gel/space ratio*, but these quantities are not easy to measure or predict. It is generally believed that the capillary porosity of a properly compacted concrete is controlled by the water–cementitious material ratio and degree of hydration. Most of the high performance concretes are produced with a water–cementitious material ratio of 0.40 or less. The practical use of very low water–cementitious material ratio concretes has been made possible by use of both conventional and *high-range water reducers,* which permit production of workable concrete with very low water contents. However, in view of cracking–deterioration interaction it should be noted that the belief that the durability of concrete can be enhanced by reducing the water–cementitious material ratio is erroneous because *it is not the water–cementitious material ratio but the water content that is more important for the control of total void space and hence cracking*. A reduction in the water content will bring about a corresponding reduction in the cement content at a given value of strength, which in turn, will reduce thermal contraction, autogenous and drying shrinkages of concrete. From standpoint of durability, it is apparent that a direct control on the maximum allowable water content in the concrete mixture is essential.

Mineral Additives The *fly ash* (FA), *the ground granulated blast-furnace-slag* (GGBFS) and *the silica fume* (SF) has been used widely as supplementary cementitious materials in high performance concrete. These mineral additives, typically fly ash and silica fume (also called condensed silica fume or micro silica), reduce the *permeability* of concrete to carbon dioxide (CO_2) and chloride-ion penetration without much change in the *total porosity*. These pozzolanas react with OPC in two ways—by altering hydration process through alkali activated reaction kinetics of a pozzolanas called pozzolanic reaction and by micro filler effect. In pozzolanic reaction the pozzolanas react with calcium hydroxide, $Ca(OH)_2$ (free-lime) liberated during hydration of cement, which comprises up to 25 per cent of the hydration product, and the water to fill voids with more calcium–silicate–hydrate (non-evaporable water) that binds the aggregate particles together. The pozzolanas may also react with other alkalis such as sodium- and potassium-hydroxides present in the cement paste. These reactions reduce permeability, decrease the amounts of otherwise harmful free lime and other alkalis in the paste, decrease free water content, thus increase the strength and improve the durability.

Fly ash used as a *partial replacement for cement* in concrete, provides very good performance. Concrete is durable with continued increase in compressive strength beyond 28 days. There is little evidence of *carbonation,* it has low to average *permeability* and good *resistance* to *chloride-ion penetration.* Chloride-

compaction. However, such concrete suffer from the loss of *entrained air* especially during pumping to high lifts. Air loss during pumping could be reduced to acceptable levels when 9 to 10 per cent of the binary OPC–SF cement is replaced with FA.

The rheological synergistic action of ternary and quaternary cements in *self-compacting concrete* can be obtained by using such cements in larger volumes, which increases the powder content of concrete and thus provides a high *colloidal volume* that combats *segregation* (a major problem with SCC) without the need of excessive amounts of costly viscosity-enhancing admixture.

Setting Time

Setting time can vary dramatically depending on the application and the presence of set modifying admixtures. Concretes for applications with early strength requirements and concretes containing one of the many non-retarding HRWRs can lead to mixtures with *rapid slump loss* and reduced working time. This is particularly true in warmer construction periods and when the concrete temperature has been kept high to promote rapid strength gain.

The use of large quantities of HRWR or other water reducing admixtures can significantly extend setting times and therefore reduce very early strengths even though strengths at more than 24 hours may be relatively high. Dosage has to be monitored closely for the mixtures containing substantial quantities of *mineral additives* so as not to overdose the Portland cement if the chemical admixture is added on the basis of total cementitious material.

Curing

The compressive strength of HSC is less sensitive to temperature and relative humidity than the *normal strength concrete*. However, tensile strength of HSC has been found to be more sensitive. The concrete containing very large quantities of *ground granulated blast-furnace-slag* requires longer moist curing times to develop adequate strength and is more sensitive to drying than plain Portland cement concretes.

The higher internal temperatures frequently found with high early strength HPC can lead to a rapid strength gain in concrete accompanied by a consequent gain in elastic modulus. The large differential temperatures occurring within a stiffer concrete will create higher stresses and can cause more pronounced *cracking* than with normal concrete. These cracks will occur, regardless of the method of curing, due to stress caused by differential temperatures.

The higher initial curing temperatures associated with HPC with high cement contents require that slabs and pavements be sawed earlier than usual to prevent cracking. Cracks which form in the first 24 hours after placement have occasionally been mislabelled as *shrinkage cracks*. At these ages the concrete is too young to have undergone any significant drying shrinkage.

(v) Fracture Modes In normal concrete *micro-cracking* begins as soon as the compressive stress reaches about 30 to 40 per cent of the *ultimate compressive stress*. At a stress level of 80–90 per cent cracks become interconnected. Whereas in HPC little micro-cracking appears until the compressive stress reaches 65 to 75 per cent of the ultimate stress. At a stress level of 80–90 per cent cracks occur mainly in the *transition zone* and are not yet fully interconnected. The fracture modes of these concretes are also different. The typical fracture surface of a normal concrete is rough and the *intergranular fracture* occurs along the *aggregate–matrix interface*; the aggregates are not broken. On the other hand the typical fracture surface of an HPC is smooth and the cracks pass without any discontinuity through both the matrix and the aggregates. This *transgranular fracture* is more sudden.

The performance of concrete is related to the *fracture energy* of concrete, which is defined as the energy dissipated per unit area during the formation of a crack. The energy is dissipated within the *fracture process zone*, the region in front of a crack tip where the stress decreases as the crack opens. The fracture energy is governed principally by the properties of the coarse aggregate. Higher-strength aggregates produce concrete with higher fracture energies.

(vi) Strain Capacity The strain capacity of concrete can be measured either in compression or in tension. In the *compression mode*, it can be measured either by concentric or by eccentric compression testing. In the *tension mode*, the strain capacity can be measured either for direct tension or indirect tension.

(vii) Stress–Strain Behaviour in Compression The stress–strain behaviour is dependent on a number of parameters which include material variables such as aggregate type and size, and testing variables such as age at testing, loading rate, strain gradient and others. Higher strength and corresponding strain are achieved for crushed aggregate from fine-grained diabase and limestone as compared to concretes made from smooth river gravel and from crushed granite that contains inclusions of a soft mineral.

For a concrete of higher strength, the shape of the ascending part of the complete stress–strain curve becomes more linear and steeper, the strain at maximum stress is slightly higher, and the slope of the descending part becomes steeper. This is true for both the *normal weight* and *light-weight aggregate concretes*.

(viii) Poisson's Ratio Poisson's ratio under uniaxial loading conditions is defined as the ratio of lateral strain to strain in the direction of loading. In the inelastic range, due to *volume dilation* resulting from internal *micro-cracking*, the apparent Poisson's ratio is not constant but is an increasing function of the axial strain. In the inelastic range, the relative increase in lateral strains is less for higher-strength concrete than for concrete of lower strength. That is, a higher-strength concrete will exhibit lesser volume dilation than a lower-strength concrete. This implies that *lesser internal micro-cracking occurs in high strength concretes*.

With a better appreciation of socio-economic implications of durability and for sustainable development, increasing emphasis is being placed on concrete durability than on its strength. In many applications, high strength concrete is used only because of its high durability quality rather than the need for its strength. Durability of concrete is *material specific* property and dependent on test methods. Durability factor involves assessment of parameters such as permeability of concrete, frost resistance, salt resistance, corrosion resistance, bond between concrete and reinforcement, resistance to alkali-aggregate reaction, etc.

Aggressive agents that attack concrete can be classified into two general categories: external or internal. External agents such as chloride-ions, carbon dioxide, freezing–thawing cycles, bacteria and abrasives actions attack concrete on being brought into contact with it while internal agents such as alkali-reactive particles in the aggregate, chloride-ions present in certain accelerators attack concrete by reacting chemically with various concrete components.

Internal Attack

Alkali-aggregate reaction The alkali–silica reaction (ASR) is the most frequently encountered phenomenon. A humid environment, a high concentration of alkalis (Na_2O and K_2O) in the interstitial liquid phases of concrete, and alkali-reactive particles (siliceous materials) in the aggregates are the main factors responsible for *alkali–aggregate reactions*. Reactions are complex and heterogeneous. Chemical and physical processes interact in the formation of an alkaline–silicate-gel either around the aggregates or in the *cracks* and *pores* of cement paste, which expands on crystallization developing high internal stresses, which crack the cement paste.

Use of *mineral additives* especially silica fume, reduces expansions due to alkali–silica reactions. They react with enough of the alkalis to reduce the volume of ASR products. They may also limit the amount of moisture available for reaction by reducing paste porosity. The type of pozzolana, its alkali content, chemical composition, and dosage, all determine how effectively it will control expansion caused by ASR. Generally, pozzolanas with higher silicon dioxide and lower lime contents are effective. Required pozzolana dosage varies, depending on chemical composition of the ash, alkali content of the concrete, and aggregate reactivity. Portland blast-furnace-slag cement with a minimum of 65 per cent slag and pozzolanic cements with a minimum of 30 per cent pozzolanic material provide sufficient protection against alkali–silica reactions with any kind of aggregates, regardless of alkali content of cement. Low alkali cement with C_3A content in the range of 6–10 per cent having alkali (as Na_2O less than 0.60 per cent) could be used when concrete is subjected to marine environments.

The cements with less than 0.6 per cent equivalent Na_2O are designated as low alkali and with more than 0.6 per cent equivalent Na_2O as high alkali. In practice, cement alkali contents of 0.6 per cent or less are usually found sufficient to prevent damage due to the alkali–aggregate reaction irrespective of

value of 0.2 mm and the air-voids should be of small size with their diameter being in the range of 0.05 to 1.25 mm to ensure that the required spacing factor is obtained with low air contents. Moreover, the frost resistance of concrete is related not only to the *water–binder ratio*, but also to the over all quality of the paste. The most important factor for frost durability is not the *total porosity* of the concrete or of the paste, but the *size, distribution of the capillary pores*, which determines the amount of freezable water and the *paste permeability*. The damage in the paste is characterized by *surface scaling,* while in mortar and concrete, distress appears in form of a few large cracks. The resistance of concrete to freezing–thawing, and wetting–drying cycles is superior for high performance concrete than that for normal concrete of similar workability and strength. For the given number of cycles long freezing–thawing cycles are more severe than the short freezing–thawing cycles.

A maximum *water–cementitious material ratio* (w/cm) of 0.40 for concrete exposed to freezing–thawing conditions and deicing salts is recommended since at a w/cm less than 0.40, a properly consolidated cement paste has a *low permeability*. In the range of 0.40 to 0.55, permeability begins to increase significantly. For an increase in the w/cm above 0.55, permeability increases at a very high rate due to an increase in *capillary channels* within the cement hydrate phases.

Properly *air-entrained concretes* containing *superplasticizers* can have adequate freezing–thawing resistance even at spacing factors greater than 0.2 mm. The non-air-entrained *high strength concretes* with water–cementitious material ratio of less than 0.24 (critical value) is frost resistant. For values of water–binder ratio higher than 0.30, the use of silica fume may be detrimental. The use of air-entrainment is necessary if *water–binder ratio* is higher than 0.30. Some fly ashes with high carbon contents reduce air content. Thus it is important to ensure that concrete containing *fly ash* also has desired *air-void-system*. However, for properly air-entrained concrete, *silica fume* does not seem to have any detrimental effect.

The very low water–cement ratio, for an adequately cured concrete, can reduce or even eliminate the amount of freezable water in the pores for practical temperature ranges. These mixes will also dramatically reduce the ingress of water, therefore, reducing the amount of damage due to physically freezing water in the concrete.

The high strength light-weight concretes are extremely resistant to frost action. The use of calcium chloride should be avoided, as it contributes to reduced freezing–thawing resistance. A light-weight coarse aggregate with its surface coated by high molecular paraffin produces light–weight concrete with high durability against freezing–thawing, abrasion and fire.

The addition of fibres themselves has no significant effect on the freezing–thawing resistance of concrete, i.e. the concretes that are not resistant to freezing–thawing will not have improved resistance by the addition of fibres.

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It is generally accepted that the type and amount of fibres currently used do not significantly enhance the *first crack tensile strength* of the fibre reinforced composite (FRC). Many of the current applications of fibre reinforced concrete involve the use of fibres ranging around 1.0 per cent by volume of concrete. Recently, it has been possible to incorporate relatively large volumes (ranging up to 15 per cent) of steel, glass, and *synthetic* (*polypropylene*) fibres in SIFCON and SIMCON systems. In these concretes with such large volume of aligned fibres, there is substantial enhancement of the tensile load carrying capacity of the matrix. This may be attributed to the fact that fibres suppress the localization of *micro-cracks* into *macro-cracks* and consequently the tensile strength of the matrix increases.

Cementitious composites with higher volume percentages (in the range of 10 to 15 per cent) of fibres termed as *slurry infiltrated concrete* (SIFCON) when used over reinforced concrete beams lead to *ductility indexes* exceeding three times those obtained without it. *Crack widths* are very much smaller than in conventional reinforced concrete. There is no need for stirrups in flexural members with SIFCON matrix.

Application of *steel fibre mats* to reinforce concrete matrix called *slurry infiltrated mat concrete* (SIMCON) produces concrete components with *extremely high flexural strength*. The advantage of steel fibre mats over a large volume of discrete fibres is that the mat configuration provides inherent strength and can utilize fibres with much *higher aspect ratios*. The fibre volume is less than half that required for SIFCON, while achieving similar *flexural strength* and *energy absorption capacity*. The typical *aspect ratio* for FRC range from 40 to 100, although special handling procedures may be required as the aspect ratio approaches 100. SIMCON utilizes fibres with aspect ratios exceeding 500.

Since the mat is already in a pre-formed shape, handling problems are minimized and balling does not become a factor. The superior performance of the SIMCON over SIFCON is related to the *bonding of the mat fibres* in the composite. In the standard SIFCON, the relatively short embedment length of 25 mm results in fibre pull-out as the primary failure mode. In the SIMCON composites, the failure mode comprises *multiple cracks* and *ultimate failure* occurs through *fibre breakage* in the *high tensile stress zones* of one or more of the crack planes. In the mat reinforced composites, the *yield strength* of the steel is fully utilized.

Compression The compressive strength of fibre reinforced concrete is relatively less affected by the presence of fibres as compared to the properties under tension and flexure. The use of steel fibres in lower strength concrete mixtures increases their compressive strength significantly compared to plain unreinforced matrices and is directly related to volume fraction of steel fibre used. Fibres improve the compressive behaviour by enhancing the *toughness*. The magnitude of the increase is dependent on the fibre shape and the content. This increase is more for hooked-end steel fibres in comparison with straight steel glass or polypropylene fibres.

CHAPTER

17

Repair Technology for Concrete Structures

17.1 INTRODUCTION

Though concrete is a relatively durable building material, it may suffer *damage* or *distress* during its life period due to a number of reasons. Because of the varying conditions under which it is produced at various locations, the quality of concrete suffers occasionally either during production or during service conditions resulting in distress. The structural causes of distress of concrete may include externally applied and environmental loads exceeding the design stipulations, accidents and subsidences. Sometimes distress in a structure is brought about by *poor construction practices, error in design and detailing,* and *construction overloads.* The other causes may be *drying shrinkage, thermal stress, weathering, chemical reactions* and *corrosion of reinforcement.* In addition to the distress in hardened concrete the plastic concrete may also suffer damage due to *plastic shrinkage* and *settlement cracking.* Sometimes on stripping off the forms a number of surface defects such as bulges, ridges, *honeycombing, bolt-holes,* etc. are noticed on the fresh concrete members. Such defects can be avoided to a large extent by providing a watertight and rigid formwork in such a way that stripping can be done without the use of crowbars or other tools. In addition to these defects, *blow-holes* develop during concreting operations due to improper design of *formwork.* These are formed in the surface of concrete by trapped air and water bubbles against the face of formwork. These can be reduced if the form face is slightly absorbent and adequate compaction is provided. In case the blow-holes are exceptionally large, or if a smooth surface is required, they must be filled with 1 : 1 or 1 : 2 cement–sand mortar. The sand should be sieved through a 300-µm or 600-µm sieve, depending upon the smoothness of the finish required. Crushed limestone dust is preferable. The mortar should be rubbed over the affected area with a rubber-faced float,

(vi) observations on the similar structures in the same locality,
(vii) study of specifications, method of construction used and the test results at the site, if any,
(viii) views of the designer, builder, occupants of the building, if any, and
(ix) weather during which the structure has been constructed.

From the above discussion it is evident that the cracking is a complex phenomenon. The various aspects of the problem are discussed as follows.

The latent defects in a concrete structure may be caused by inadequacy of design, materials or construction practices which may not become evident until sometime after its completion. The immediate cause of deterioration may be a chemical action or corrosion of reinforcement, but in majority of the cases the basic cause may be traced back to something such as unrealistic detailing or poor workmanship.

The incompatible dimensional changes caused by drying shrinkage and thermal movements during and after the hardening period may also cause cracks in concrete members. Before any repair work is taken in hand, the cause of damage must be clearly identified, for which careful investigation is required. Some of common causes are discussed in the following sub-sections.

17.2.1 Cracking of Plastic Concrete

When the exposed surfaces of freshly placed concrete are subjected to a very rapid loss of moisture caused by low humidity, wind, and/or high temperature, the surface concrete shrinks. Due to restraint provided by the concrete below the drying surface layer, tensile stresses develop in the weak, stiffening plastic concrete, resulting in shallow cracks that are usually short, discontinuous running in all directions and very seldom extend to the free edge. In an unreinforced slab they are typically diagonal as shown in Fig. 17.1(a). In the presence of reinforcement their pattern may be modified. Plastic shrinkage usually occurs prior to final finishing before curing starts. The cracks are often fairly wide at the surface. They range from few centimetres to many metres in floors or slabs or other elements with large surface areas. *Plastic shrinkage cracks* may extend the full depth of elevated thin structural elements.

Plastic shrinkage cracks can be controlled by reducing the relative volume change between the surface and the interior concrete by preventing a rapid moisture loss due to hot weather and dry winds. This can be accomplished by using fog nozzles to saturate the air above the surface and use of plastic sheeting to cover the surface between the final finishing operations. Wind breakers reducing the wind velocity, and sunshades reducing the surface temperatures are also helpful, and it is good practice to schedule flatwork after the walls have been erected. The remedial measures after the cracks have formed usually consists of sealing them against entry of water by brushing in cement or low viscosity polymer.

continuous inspection during all phases of construction to supplement the careful design and detailing.

Cracks due to Externally Applied Loads

Load-induced tensile stresses may result in cracks in concrete elements. A design procedure specifying the use of reinforcing steel, not only to carry tensile forces, but also be obtain both an adequate crack distribution and a reasonable limit on crack width is recommended. Flexural and tensile crack widths can be expected to increase with time for members subjected to either sustained or repetitive loading. A well-distributed reinforcing arrangement offers the best protection against undesirable cracking.

17.3 EVALUATION OF CRACKS

As in the case of a medical practitioner prescribing medicine without thoroughly examining the patient, it is difficult for a repair engineer to advocate any repair technology without making a thorough investigation. Before proceeding with repair, the investigations should be made to determine the location and extent of cracking, the causes of damage, and the objectives of repair. Calculation can be made to determine stresses due to applied loads. For detailed information, the history of the structure, structural drawings and specifications, and construction and maintenance records should be reviewed.

The objectives of repair include restoration and enhancement of durability, structural strength, functional requirements and aesthetics. The evaluation of cracks is necessary for the following purposes:

(i) To identify the cause of cracking.
(ii) To assess the structure for its safety and serviceability.
(iii) To establish the extent of the cracking.
(iv) To establish the likely extent of further deterioration.
(v) To study the suitability of various remedial measures.
(vi) To make a final assessment for serviceability after repairs.

Apart from visual inspection, tapping the surface and listening to the sound for hollow areas may be one of the simplest methods of identifying the weak spots. The suspected areas are then opened up by chipping the weak concrete for further assessment.

The comparative strength of concrete in the structure may be assessed to a reasonable accuracy by *non-destructive testing* and by the tests on the *cores* extracted from the concrete. The commonly used non-destructive tests are the *rebound-hammer* test and *ultrasonic-pulse-velocity test.*

17.3.1 Visual Examination

The appearance of concrete surface may suggest the possibility of chemical attack by a general softening and leaching of matrix, or in case of sulphate attack

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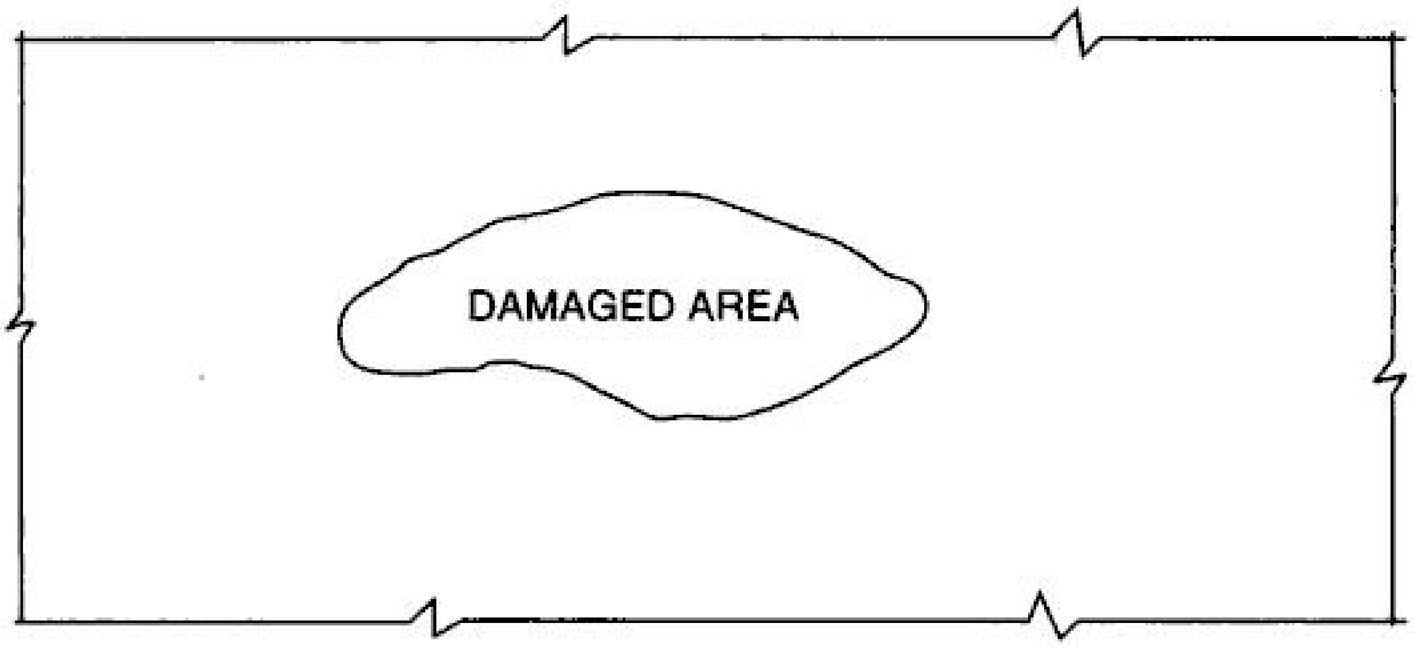
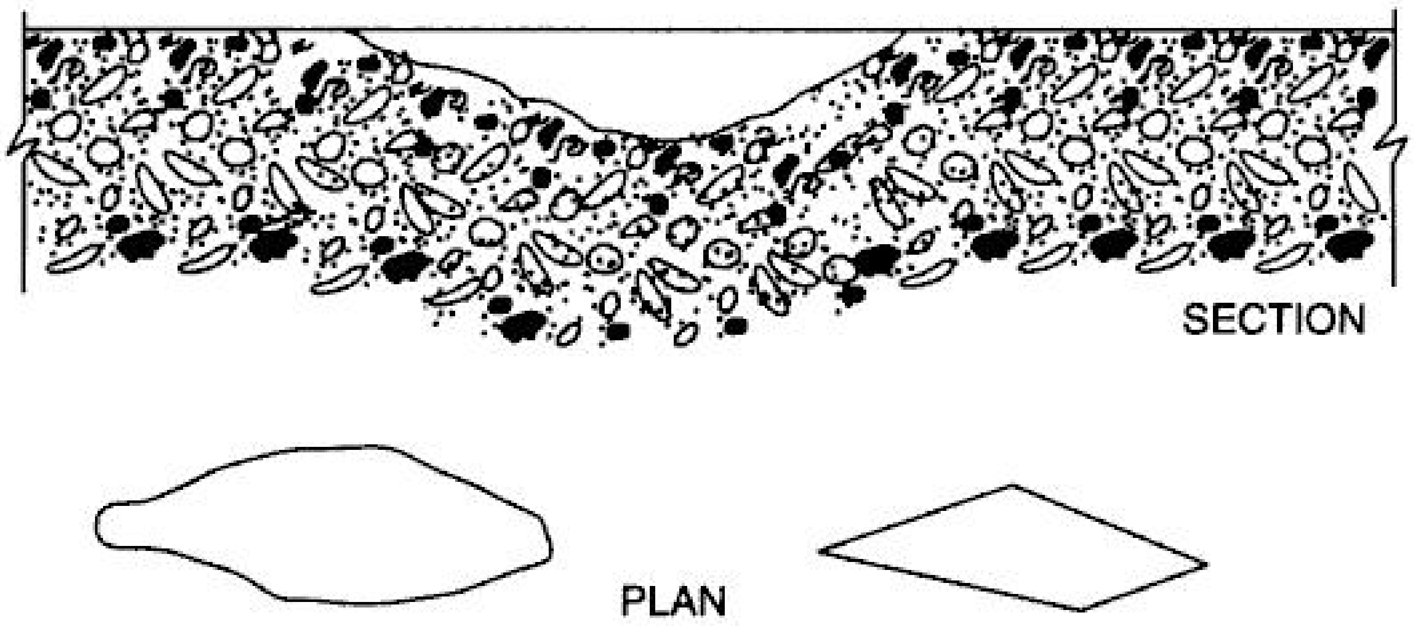
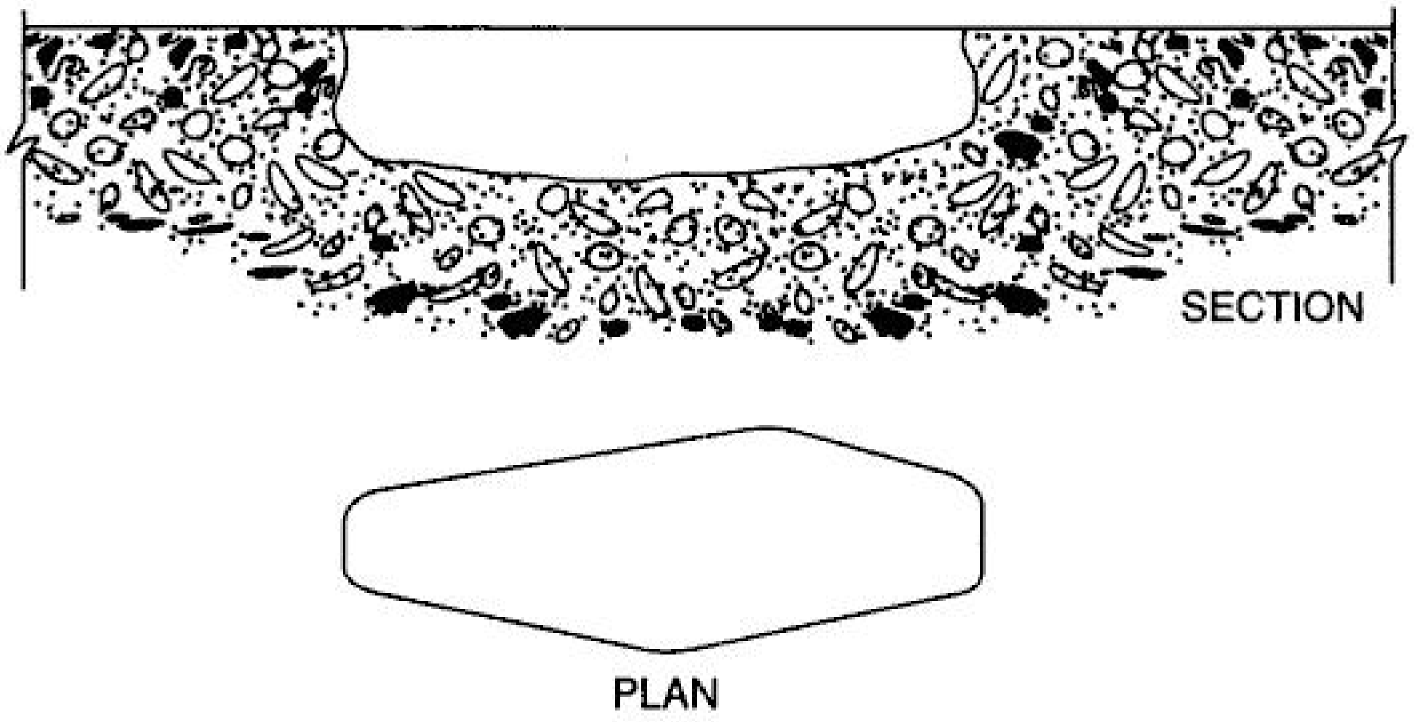


(a) Plan of damaged area



(b) Incorrect method of cutting out



(c) Correct method of cutting out

Fig. 17.4 Preparation of surface by cutting out the damaged area

A vigorous scrubbing action with a stiff broom should be carried out during the washing procedure. The surface should be washed off thoroughly with a pressure hose to remove all traces of loosened oil and as well as the cleaning solution.

The *laitance* which can be detected by the presence of fine powder on surface, when it is scrapped with the knife blade, may be removed by *acid*

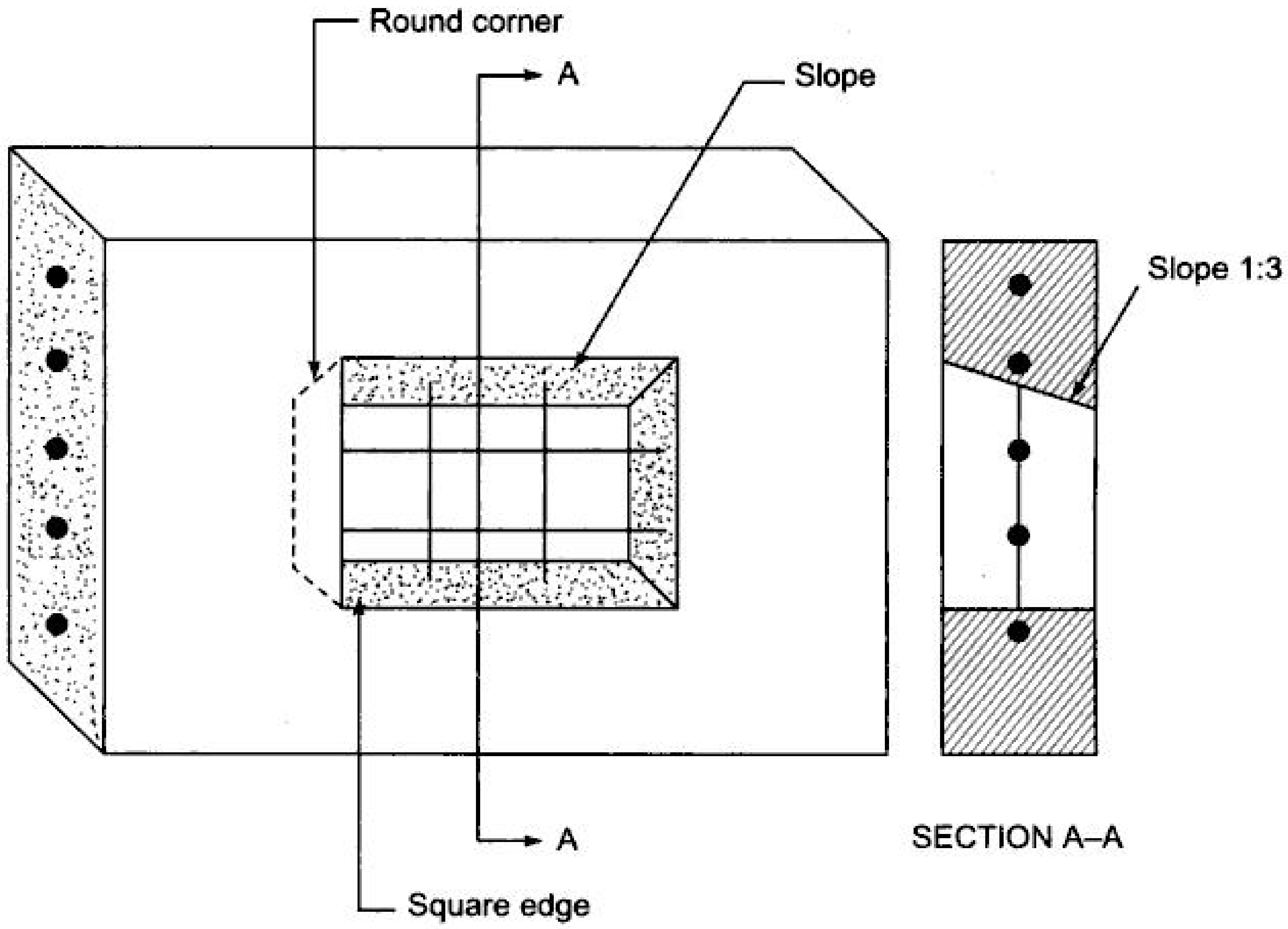


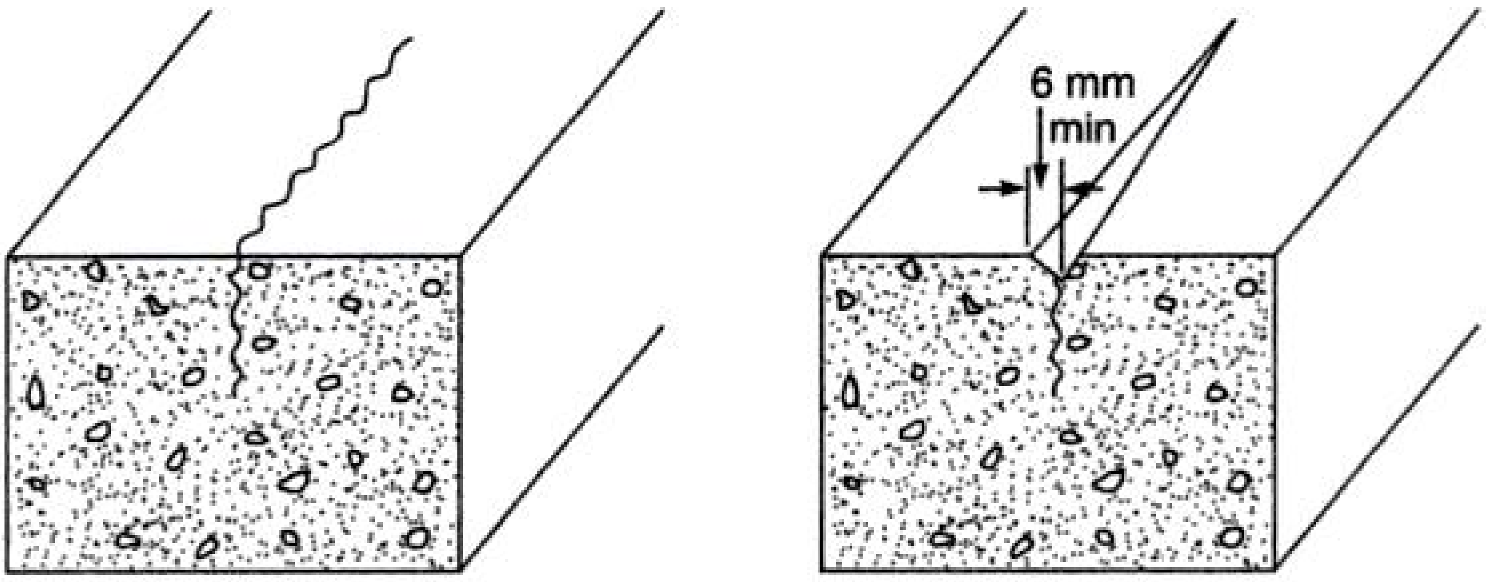
Fig. 17.5 Repair by concrete replacement

(c) *Mortar replacement* The method is suitable for the cavities which are too wide for drypack or too shallow for concrete replacement. Generally it is used for shallow depressions no deeper than that for the side of the reinforcing bars nearest to the surface. For replacement of deteriorated concrete, this method is suitable for minor restorations. The mortar replacement can be done by hand or can be applied pneumatically by using a small pressure gun.

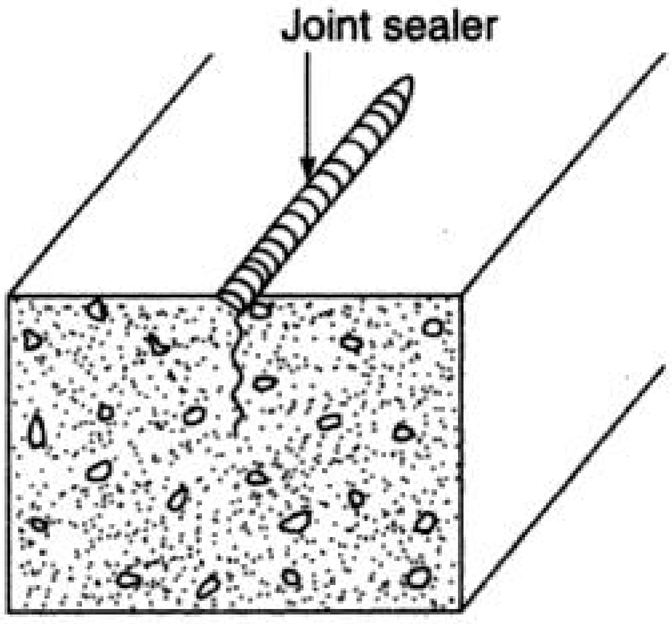
It is preferable to *pre-shrink* the repair mortar by mixing it to a plastic consistency as long in advance of its use as cement permits (the pre-shrinking time ranges from 60 to 120 minutes). For hand placing, the mortar should have the same proportions as the mortar used in the mix of which the structure is made. In the case of a pressure gun, the ratio recommended is 1 part of cement to 4 parts of sand.

In case the hole being repaired is deeper than 25 mm, the mortar should be applied in layers not exceeding 15 mm in thickness to avoid sagging loss in the bond. The subsequent layers are laid at an interval of 30 minutes or more. The final layer placed slightly overflowing the hole is struck off level with the surface.

(d) *Grouting* The wide and deep cracks may be repaired by filling them with Portland cement grout. The grout mixtures may contain cement and water or cement, sand and water, depending upon the width of the crack. However, the water–cement ratio should be kept as low as practicable to maximize strength and minimize shrinkage. Water-reducing admixtures may also be used to



ORIGINAL CRACK ROOTING



SEALING

Fig. 17.8 Crack repair by routing and sealing

The prepared routed crack is filled with a suitable field moulded flexible sealant with strain capacity being at least as large as the one to be accommodated. A wide crack spreads movement over a greater width so that the resulting strain is compatible with sealant to be used. The sealant must adhere to the sides of the chase but debonded from bottom so that the movement in the crack spreads over the full width of the chase. This can be achieved by providing a *bond breaker* or *debonding strip* of a material such as polyethylene or *pressure sensitive tape* at the bottom of the chase before sealant is applied. This debonding strip does not bond to the sealant before or during cure and allows the sealant to change shape without stress concentration at the bottom. The dimensions of the seal are an integral part of its performance, Figure 17.9 shows a sectional view of a typical flexible sealing or movement joint. With an increase in chase width, the crack movement which induces shear or tension in sealant will exert considerably reduced stress on the adhesive interface with concrete, and thus enabling the face seal to cope with extensive movement.

17.6.3 Providing Additional Steel

The cracked reinforced elements (usually bridge decks) can be successfully repaired using epoxy injection and reinforcing bars. The technique consists of sealing the crack, drilling holes of 20 mm diameter at 45° to the element surface

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The collar can also be used as mid-column bearing surface, acting as circumferential beam to distribute the concentrated load around the column. The collar is subjected to shear and bending along the collar circumference as well as direct bearing stress under concentrated load. Thus in addition to shear transfer reinforcement, the collar should be provided with reinforcement for shear and moment within the collar. The repair can be used as an alternate load path from the column to the collar and then to the connecting structural component. Column collars can be provided below the slab to act as *column capital* to improve punching shear strength of the slab column connection as shown in Fig. 17.13. The collars are reinforced with circular ties and with dowel bars embedded 150 mm into columns.

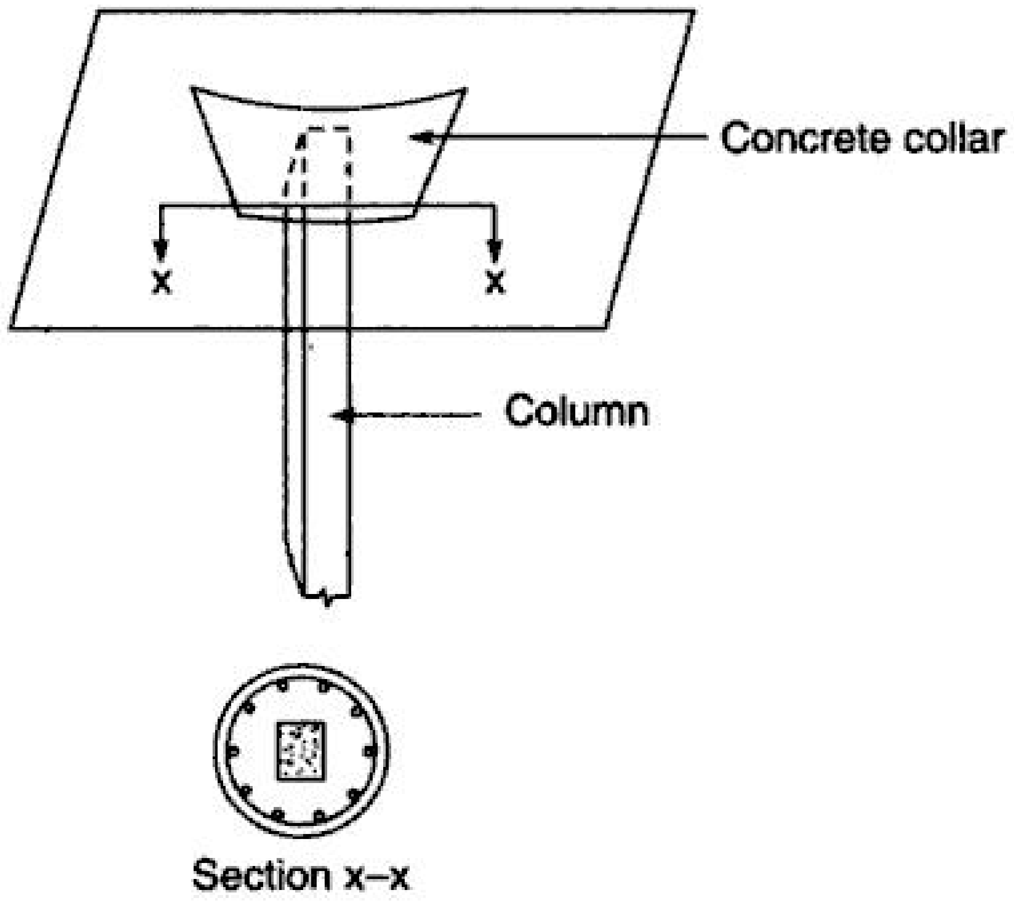

Fig. 17.13 Strengthening of a slab and column connection using a concrete collar

For repairing the bridge piers, a repair scheme using steel-encased column collars can be used. In this design, the steel shell provides the circular reinforcement.

Unless the shear transfer strength is verified by load tests, the collar design and construction should meet the following criteria.

(i) The collar height should be at least 0.8 times of the original column cross-sectional dimension.
(ii) The collar reinforcement is located at or near the outside face. The steel is lapped or welded for full development of strength.
(iii) The concrete strength should be at least 22.5 MPa.
(iv) The column surface should be roughened in the shear transfer zone by bush-hammer. The surface of the column is then cleaned by a wire brush and high-pressure air to remove any loose concrete before placing the concrete in the formwork. Fibre-glass-reinforcement plastics, ferrocement,

wear or damage so as to obtain a sound clean concrete surface suitable for repair. Subsequent to this *scarification* of the repair area, the sounding technique can be used to locate *surface delaminations* by using small chipping guns.

The reinforcement in the delamination area should be exposed and chipping continued until all concrete within 12 mm of the entire exposed portion of the reinforcing bar is removed. The prepared surface should be resounded to ensure that all delaminated unsound concrete has been removed. The exposed reinforcement should be sand-blasted to remove all corrosion by-products. Finally, the entire repair area is blown off with compressed air to remove any loose corrosion particles, concrete, blasting sand and dust.

In case of cement–mortar repair the prepared reinforcement surface is coated with a cement paste layer which will provide additional protection to the reinforcement. After the coated surface has cured, the repair area is thoroughly kept wet for 24 hours if possible. All surface water must be removed from the area before filling in the cementitious material consisting of 1 : 3 cement–sand mix with sufficient water for firm pressing by hand. When the quantities required for repair are small, the proprietary materials may be used which are carefully batched and their quality controlled. Because of higher workability they require only hand tamping. For other cement–sand mixes vibrating hammers with a square plate on foot are often used, but for a large area a short beam fitted with form vibrator may be used to press the material into the repair area. The repair is finished off with a hand trowel and kept covered with polythene for 7 days.

Dry cementitious mortar materials along with a polymer, such as styrene butadiene rubber latex, can be used for high quality local repairs. The following procedure may be adopted.

The exposed concrete and reinforcement surfaces are coated with a primer compatible with the repair system. The primer can be applied by rolling it on to the surface with a paint roller and allowed to cure. The coat of primer on the reinforcement provides additional protection against corrosion. After the primed surface is adequately cured, the surface becomes impermeable to moisture and could remain unprotected from environmental effects.

An initial bedding layer of polymer concrete is placed in deep areas around the exposed reinforcement. The purpose of this initial bedding layer is to assure that the exposed reinforcement be encapsulated by the polymer concrete. In deeply removed areas beneath the reinforcing bars the polymer concrete is spaded to remove the air pockets. The chipped areas are then back-filled with sand-loaded polymer concrete. A skin coat of neat polymer concrete followed by a 6 mm layer is applied over the exposed reinforcement.

The material is allowed to cure before proceeding to the application of the second layer of polymer concrete to build-up the cover thickness in the area over the reinforcement. The final lift of polymer concrete is applied over the entire repair area to provide a minimum thickness of 6 mm. A minimum of 12 mm polymer cover is desired over the exposed reinforcement.

requires injection of sealant (grout) to fill water passages completely, and it is necessary to attain a relatively high flow velocity to achieve this, because of the short *pot-life* or working time of the typical repair material. The first basic step is to restrict or confine the water flow to a tube through which the sealant may be introduced. Once the flow of water has been controlled, the connection between the tube and concrete must be made strong enough to withstand the injection pressure.

However, due to possibility of concrete being stressed during injection, it is preferable to maintain lower pressures. A typical direct method entails the injection of material up the pressure gradient from the down stream side. On the other hand an indirect method involves the introduction of the sealant on the pressure side, so that the pathways are filled under the acting hydrostatic head. The direct methods are very slow due to sealant being pumped slowly through very narrow passages against pressure, and the pressure cannot be maintained for long enough to achieve complete penetration. In many cases water may find another finer pathway leading from the same source. In contrast the indirect methods enable the work to be completed quickly because surface seals are not required and mechanical anchorages can be used.

17.9 UNDERWATER REPAIRS

Many of the methods discussed in the preceding sections for above-water (dry) repairs may also be used underwater with only minor modifications. However, the materials specified for use in air are often unsuitable for underwater application. The special features of underwater repair are:

1. Due to high cost and complexity of underwater working, the repair operations need be made as simple as possible. The choice of repair technique is influenced by the available method of access.
2. Adequate preparation of damaged area may require specially adapted techniques.
3. The repair materials must be compatible with underwater application both during placing and curing. Cementitious systems have been found to be better suited for underwater use.
4. Formwork and placement method adopted must minimize mixing between repair material and water.
5. Underwater supervision of repair operations is difficult and costly.

Generally, laboratory trials on both materials and repair methods are used to identify possible problem areas and ensure smooth site operations. Before a repair is undertaken it is necessary to clean the damaged area of marine encrustation (contaminants) to allow detailed inspection to assess the extent of damage. In case of smaller areas, this can be accomplished by using mechanical wire brushes, needle guns or scabbling tools. However, for larger areas a high pressure jet may provide a solution. Once the area has been cleaned, the extent

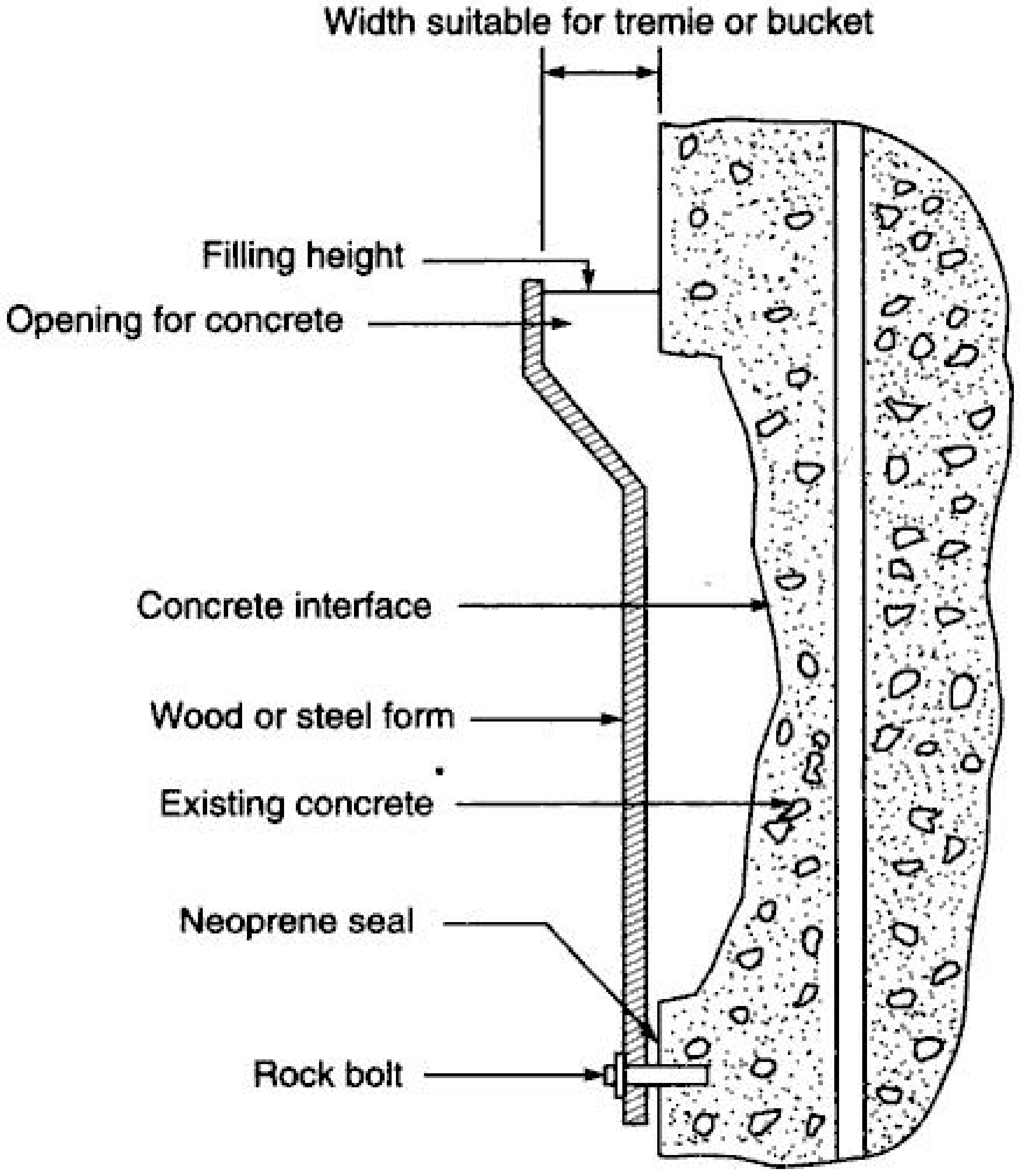

Fig. 17.16 Typical formwork details for underwater vertical repair

a slight deformation may occur. In the first case, the whole of damaged portion has to be replaced during restoration of structure while in the latter, only repair and finishing may be required. The extent of damage caused to the structure during a fire depends on the duration of fire, and the temperature to which the structure was subjected during the fire.

High temperature during a fire reduces the strength of reinforced concrete structures due to change in the strength and deformability of materials, reduction in cross-sectional dimensions, weakening of bond between the reinforcement and concrete, which determines structural action under the load. The maximum temperature reached during a fire is normally estimated indirectly, e.g. from the melting of metallic or other non-combustible articles. A temperature of 1000–1100 °C has been observed during fire in residential and administrative buildings. The duration of these fires was mostly between 1 and 2 hours. It has been observed that during fires in theatres and departmental stores, temperature rises up to 1100–1200 °C and the fire duration exceeded 2–3 hours in some cases. Still higher temperatures have been observed during fires in industrial buildings and ware-houses in which considerable quantities of solid and liquid combustible materials were processed or stored. During a fire in a store of combustible liquid and lubricants which lasted more than 2 hours, temperature of 1300 °C has been reported. Thus duration of a fire and maximum temperature reached can vary

either toward mid span, or upward to the location of the concentrated load resulting in delamination of the covercrete (anchorage failure).

17.11.2 Compression Members

Fibre-reinforced polymer (FRP) plate jackets and steel hoops have been successfully used in the past to enhance the shear strength and curvature ductility capacity in reinforced concrete compression members. The enhancement is due to the dual confinement effects provided by an external FRP jacket and by internal steel hoops. The jackets are also very effective in preventing longitudinal bar buckling. FRP jackets are relatively easy to instal and are cost-competitive when compared with conventional options such as concrete or steel jacketing, particularly when access to the columns is limited. FRP jacketing of columns in deficient buildings is an increasingly attractive retrofit option in seismically active areas.

17.12 STRENGTHENING OF DEFICIENT STRUCTURES

In case of *deficient* or *under-designed* structures the distress may appear in the form of deflections exceeding the *serviceability limits*, i.e. excessive sagging, accompanied by micro-cracking along the length of beams in their top fibres. The structural behaviour of members is dependent on their stiffness. In order to assess the magnitude of deficiency in design the first step is to compute flexural stiffness of the elements with their existing sizes. Before embarking on strengthening of flexural members like beams and slabs, it is to be ensured that columns and foundations are adequate. To illustrate the procedure consider the case of a framed structure where slabs and beams have been found to be under-designed in thickness, however, columns and their foundations are adequate. In order to *strengthen* or *enhance the flexural stiffness of beams and slabs*, it is proposed to increase the cross-section of beams and thickness of slab.

17.12.1 Strengthening of Beams

Before taking up the strengthening of a beam, the load acting on it should be reduced by removing the tiles, bed mortar, etc. from the slab. In addition props may be erected at mid span of each slab and tightened in such a manner that slab is not damaged. After chipping off of the existing plaster on the beam, *additional reinforcement* at the bottom of beam together with new stirrups (if required) are provided. The bars are passed through or inserted in the supporting columns through holes of appropriate diameter drilled in the columns. The spaces between bars and surrounding holes are filled with epoxy grout to ensure a good bond.

Expanded wire mesh is fixed and anchored on three sides of the beam as shown in Fig. 17.17. To ensure a good bond between old concrete and *polymer-modified mortar*, an *epoxy bond coat* is applied to the concrete surface. While the bond coat is still fresh, a layer of polymer modified mortar is applied. The

(a) reacts with water only (b) hydrates rapidly (c) hardens rapidly (d) generates less heat of hydration (e) has no resistance to sulphate attack

19. Snowcem is

    (a) chalk powder (b) powdered lime (c) mixture of chalk powder and lime (d) coloured-cement (e) none of the above

20. In testing the Portland cement for the loss on ignition, the sample is heated to

    (a) 100 °C (b) 250 °C (c) 500–800 °C (d) 900–1000 °C (e) 1250 °C

21. In the case of Portland cement the loss on ignition should be

    (a) less than 4 per cent (b) less than 10 per cent (c) within 10 to 15 per cent (d) less than 20 per cent (e) more than 20 per cent

22. During the test of OPC for loss on ignition, the loss in weight occurs due to

    (a) decomposition of silicates (b) chemical reaction (c) burning of constituents (d) melting of tricalcium aluminate (e) evaporation of moisture and carbon dioxide

23. The insoluble residue in cement should be

    (a) between 10 and 15 per cent (b) less than 10 per cent (c) between 5 and 10 per cent (d) between 1.5 and 5 per cent (e) less than 0.85 per cent

24. Total heat of hydration of cement is independent of

    (a) ambient temperature (b) composition of cement (c) fineness of cement (d) all of the above

25. The length of time for which the concrete mixture remains plastic predominantly depends on the

    (a) setting time of cement (b) amount of mixing water (c) atmospheric temperature (d) equally on all of the above

26. Initial setting time is maximum for

    (a) Portland–pozzolana cement (b) Portland–slag cement (c) low-heat Portland cement (d) high strength Portland cement

27. The setting time of cement is influenced by

    (a) percentage of water and its temperature (b) temperature and humidity of air (c) amount of kneading the paste (d) all of the above (e) none of the above

28. For ordinary Portland cement

    (a) initial setting time should not be less than 5 minutes and final setting time should not be more than 24 hours (b) initial setting time should not be less than 30 minutes and final setting time should not be more than 600 minutes (c) initial setting time should not be less than 60 minutes and final setting time should not be more than 600 minutes (d) initial setting time should not be less than 5 minutes and final setting time should not be more than 600 minutes (e) none of the above

73. Autoclave method is used to determine
    (a) residue (b) expansion (c) heat of hydration (d) sulphur content (e) none of the above
74. Le Chatelier's method can be used to determine
    (a) unsoundness of cement (b) soundness of cement (c) fineness of aggregate (d) sulphur content (e) all of the above
75. The specific surface of OPC is determined by
    (a) Le Chatelier's apparatus (b) air-permeability method (c) autoclave method (d) sieve analysis (e) photo-calorimeter method
76. The specific surface of cement is expressed in
    (a) mm^2 (b) mm^2/g (c) g/mm^2 (d) mm^3/g mm (e) any of the above
77. The hydration of concrete ceases at the temperature of
    (a) 0°F (b) 0°C (c) 11°F (d) 11°C (e) none of the above
78. The average specific surface of cement is closer to
    (a) 100 000 mm^2/g (b) 200 000 mm^2/g (c) 300 000 mm^2/g (d) 400 000 mm^2/g (e) 500 000 mm^2/g.
79. Initial setting time of concrete ceases at
    (a) –10 °C (b) –4 °C (c) 0 °C (d) 4 °C (e) none of the above
80. Which of the following statement(s) is are incorrect?
    (a) Calcium chloride should not be used in pre-stressed concrete (b) Strength of concrete increases below freezing point of water (c) Hardening of concrete takes place rapidly in hot weather (d) The ingredients of concrete should be mixed within three minutes (e) All of the above
81. While ______ is a calcareous material ______ is an argillaceous material.
    (a) limestone, shale (b) clay, limestone (c) shale, limestone (d) slate, laterite (e) marl, chalk
82. The colour of ordinary Portland cement is ______ and that of Portland–pozzolana cement is ______
    (a) white, black (b) brown, grey (c) grey, light grey (d) white, grey (e) grey, black
83. White cement is the ______ cement and low-heat cement is used in ______ structures.
    (a) cheapest, thin (b) costliest, thick (c) costliest, thin (d) cheapest, thick
84. For fineness test of cement IS sieve of ______ is used.
    (a) 90 μm (b) 9 μm (c) 150 μm (d) 300 μm (e) 600 μm

AGGREGATES

85. Aggregate is used in concrete because
    (a) it is a relatively inert material and is cheaper than cement (b) it imparts volume stability and durability to the concrete (c) it provides bulk to the

(a) power requirement in the mixing plant (b) reproducibility of concrete batches (c) time of mixing (d) homogeneity of fresh concrete (e) all of the above

134. Workability of fresh concrete is most appropriately defined by

(a) the composite property satisfying the requirements of mixability, stability, transportability, placeability, mobility, compactability and finishability (b) ease and homogeneity with which it can be mixed. placed, compacted and finished (c) its consistency and plasticity (d) its slump and compaction factor values (e) all of the above

135. The empirical test used for assessing the workability of fresh concrete is

(a) the slump test (b) the compacting factor test (c) the vee-bee consistency test (d) the flow test (e) all of the above

136. The conventional empirical tests for assessing workability of concrete suffer from the drawback that

(a) they measure only a particular aspect of workability, i.e. these tests are single-point tests (b) they are operator sensitive (c) none of these tests is capable of dealing with the whole range of workabilities (d) they are not amenable to physical idealized modelling (e) all of the above

137. Identify the incorrect statement(s) with regard to the workability of fresh concrete.

(a) The change in workability due to a relative change in the water content in concrete is dependent on the mix ratio. (b) An increase in water content may result in a monotonous increase in workability. (c) A high water content may result in segregation and bleeding. (d) Water content is limited to a value given by the water–cement ratio. (e) All of the above

138. Which of the following statement(s) is/are incorrect?

(a) The use of a larger size and/or rounded aggregate gives higher workability. (b) For the same water content, use of finer sand increases the workability. (c) The grading of fine aggregate is more critical than the grading of coarse aggregate for workability. (d) For high-strength concrete a coarser grading is preferred. (e) fineness of cement has an influence on bleeding.

139. Identify the incorrect statement(s).

(a) The segregation of coarse particles in a lean dry mix may be corrected by adding a small quantity of water to it. (b) The tendency to segregate can be minimized by reducing the height of drop of concrete. (c) The separation of cement paste from the concrete mix is termed segregation. (d) The aim is to have minimum possible workability consistent with satisfactory placement and compaction of concrete. (e) All of the above

140. Slump test is the most widely used field test primarily because

(a) it indicates the behaviour of fresh concrete under action of gravitational forces (b) of the simplicity of apparatus and test procedure

(a) deterioration from environmental conditions (b) internal desruptive forces (c) chemical attack (d) all of the above

184. The inelastic behaviour of concrete is due to the
    (a) shrinkage in concrete (b) propagation of bond and mortar cracks (c) presence of macro and micro cracks (d) use of aggregates (e) all of the above
185. The thermal conductivity of concrete decreases with the
    (a) light weight concretes (b) increase in the water–cement ratio (c) decrease in the cement content (d) all of the above (e) none of the above
186. For cement concrete the stress–strain curve is linear approximately up to
    (a) 1/4 of ultimate stress (b) 1/3 of ultimate stress (c) 1/2 of ultimate stress (d) 5/8 of ultimate stress (e) 3/4 of ultimate stress
187. The modulus of elasticity of concrete improves with
    (a) age (b) high water–cement ratio (c) shorter curing periods (d) better compaction (e) all of the above
188. Shrinkage of concrete can be reduced by using
    (a) low water–cement ratio (b) water-tight and non-absorbent formwork (c) presaturated aggregates (d) all of the above (e) none of the above
189. The strength of concrete mainly depends upon
    (a) quality of fine aggregate (b) quality of coarse aggregate (c) fineness of cement (d) water–cement ratio (e) none of the above
190. The thermal coefficient of expansion of concrete is approximately
    (a) 3×10^{-8} per °C (b) 3×10^{-6} per °C (c) 3×10^{-5} per °C (d) 3×10^{-4} per °C (e) 3×10^{-3} per °C
191. Creep in concrete is undesirable particularly in
    (a) continuous beams (b) reinforced concrete columns (c) pre-stressed concrete structures (d) all of the above (e) none of the above
192. The knowledge of the flexural tensile strength is useful in design of
    (a) reinforced concrete members (b) pavement slabs and airfield runways (c) prestressed concrete structures (d) water-retaining structures (e) all of the above
193. Compressive strength of concrete is the most important property because
    (a) it depends upon the water–cement ratio (b) it is related to the structure of hardened cement paste and gives the overall quality of concrete (c) it indicates the extent of voids in the concrete (d) it affects the permeability and durability of concrete (e) none of the above
194. The concrete may attain its 100 per cent compressive strength after
    (a) 7 days (b) 14 days (c) 28 days (d) 1 year (e) 3 years
195. The strength of concrete is decreased by
    (a) vibration (b) impact (c) fatigue (d) all of the above (e) none of the above
196. The permissible stress for concrete subjected to fatigue should be

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246. When water is added in an increasing amount to a fixed mass of dry mortar mix, the volume of mortar
(a) initially increases then decreases to a minimum value (b) does not change as the water simply fills the voids (c) decreases (d) increases (e) increases proportionately more than the volume of water added

247. The volume of water which corresponds to a minimum volume of mortar is termed
(a) saturation water content (b) basic water content (c) lowest water content (d) highest water content (e) hygroscopic water content

248. The amount of water mixed in mortar should be always
(a) more than the basic water content (b) equal to the basic water content (c) less than the basic water content (d) 50 per cent of the basic water content (e) none of the above

249. A water content of 1.25 for a mortar mix means
(a) 1.25 litre of water has been added per litre of cement (b) 1.25 litre of water has been added per litre of mortar (c) 25 per cent more water has been added than the basic water content requirements (d) 1 litre of water has been added in 1.25 litre of mortar (e) none of the above

250. The nominal mix corresponding to M20 grade concrete is
(a) 1:1:2 (b) 1:1.5:3 (c) 1:2:3 (d) 1:2:4 (e) 1:3:6

251. The grade of concrete corresponding to nominal mix proportions of 1:3:6 is
(a) M35 (b) M25 (c) M15 (d) M10 (e) M7.5

252. The total number of grades of ordinary concrete stipulated in IS: 456–2000 are
(a) 10 (b) 8 (c) 3 (d) 6 (e) 5

253. The volume of sand per cubic metre of 1:2:4 (by volume) concrete would be approximately
(a) 0.2 to 0.4 (b) 0.4 to 0.6 (c) 0.6 to 0.9 (d) 0.8 to 1.0 (e) none of the above

254. The number of bags of cement required per cubic metre of 1:2:4 concrete, would be approximately
(a) 5 to 6 (b) 4 to 5 (c) 3 to 4 (d) 2 to 3 (e) 1 to 2

255. For slabs and beams, the concrete of nominal mix generally used is
(a) 1:1:2 (b) 1:1.5:3 (c) 1:2:4 (d) 1:3:6 (e) 1:2:3

256. For water retaining structures the nominal mix generally used is
(a) 1:1:2 (b) 1:1.5: 3 (c) 1:1.5: 4 (d) 1:2:4 (e) 1:2:6

257. To take into account the variation of individual samples, the laboratory design strength can be obtained by increasing the target mean strength by (per cent)
(a) 5 (b) 5 to 10 (c) 10 to 15 (d) 15 to 20 (e) 20 to 25

5 °C (c) Underwater concreting (d) Any of the above (e) None of the above

312. Concreting in hot weather

(a) reduces handling time of fresh concrete and strength of hardened concrete (b) increases tendency to cracking (c) make it difficult to control air content (d) all of the above (e) none of the above

313. In hot weather concreting it is recommended to

(a) use cold mixing water (b) have minimum cement content consistent with other functional requirements (c) use cements with lower heat of hydration and use water reducing admixtures (d) reduce period between mixing and placement to an absolute minimum (e) all of the above

314. Concreting in cold weather

(a) reduces rate of development of strength (b) delays removal of formwork (c) temperature differential within the concrete mass may promote cracking (d) freezing and thawing during the prehardening period may reduce strength by 50 per cent (e) all of the above

315. In cold weather concreting it is recommended to

(a) heat the water for mixing (b) use insulating formwork and delay its removal (c) use additional quantity of cement (d) use air-entraining agents (e) all of the above

316. In cold weather curing of concrete should be continued for

(a) 7 days (b) 14 days (c) 21 days (d) 28 days (e) 45 days

317. For placing the concrete underwater the principal technique(s) used are

(a) tremie method (b) bucket placing (c) placing in bags (d) prepacked concrete (e) any of the above

318. The timber formwork for concrete should be made of

(a) teak wood (b) Shisham wood (c) soft wood planks (d) green timber (e) hard wood

319. For a concrete slab for a 3.75 × 4.75 m room the stripping time of form should be

(a) 3 days (b) 7 days (c) 14 days (d) 21 days (e) 28 days

320. To take care of any sag in the beams, the forms are given a camber of

(a) 1:200 (b) 1:300 (c) 1:500 (d) 1:650 (e) 1:750

321. For a medium income group big housing project which type of formwork you would recommend?

(a) timber formwork (b) plywood formwork (c) steel formwork (d) other type

INSPECTION AND TESTING

322. Identify the incorrect statement(s).

(a) The testing of representative concrete does not give the quality of actual in-place concrete (b) Quality control can be exercised by testing

187. (a)	188. (d)	189. (d)	190. (d)	191. (c)	192. (b)
193. (b)	194. (e)	195. (d)	196. (b)	197. (e)	198. (c)
199. (c)	200. (c)	201. (e)	202. (c)	203. (d)	204. (c)
205. (d)	206. (b)	207. (c)	208. (d)	209. (d)	210. (d)
211. (d)	212. (d)	213. (d)	214. (b)	215. (a)	216. (b)
217. (b)	218. (d)	219. (b)	220. (a)	221. (c)	222. (c)
223. (d)	224. (d)	225. (e)	226. (e)	227. (d)	228. (d)
229. (f)	230. (b)	231. (b)	232. (f)	233. (c)	234. (e)
235. (d)	236. (b)	237. (e)	238. (d)	239. (d)	240. (e)
241. (d)	242. (b)	243. (b)	244. (a)	245. (c)	246. (a)
247. (b)	248. (a)	249. (c)	250. (b)	251. (d)	252. (c)
253. (a)	254. (a)	255. (b)	256. (a)	257. (c)	258. (a)
259. (d)	260. (b)	261. (d)	262. (b)	263. (c)	264. (b)
265. (a)	266. (d)	267. (c)	268. (d)	269. (d)	270. (a)
271. (d)	272. (e)	273. (c)	274. (b)	275. (e)	276. (b)
277. (f)	278. (a)	279. (d)	280. (c)	281. (d)	282. (b)
283. (b)	284. (e)	285. (a)	286. (e)	287. (e)	288. (e)
289. (d)	290a. (e)	290b. (a)	291. (a)	292. (c)	293. (b)
294. (d)	295. (d)	296. (d)	297. (c)	298. (b)	299. (b)
300. (d)	301. (b)	302. (b)	303. (b)	304. (d)	305. (c)
306. (b)	307. (b)	308. (c)	309. (c)	310. (b)	311. (d)
312. (d)	313. (e)	314. (e)	315. (e)	316. (d)	317. (e)
318. (c)	319. (c)	320. (c)	321. (c)	322. (b)	323. (d)
324. (c)	325. (b)	326. (c)	327. (e)	328. (a)	329. (e)
330. (c)	331. (a)	332. (b)	333. (e)	334. (c)	335. (a)
336. (b)	337. (e)	338. (e)	339. (b)	340. (c)	341. (d)
342. (e)	343. (a)	344. (b)	345. (c)	346. (c)	347. (a)
348. (b)	349. (a)	350. (c)	351. (d)	352. (e)	353. (e)
354. (a)	355. (e)	356. (b)	357. (d)	358. (a)	359. (c)
360. (a)	361. (b)	362. (d)	363. (e)	364. (c)	365. (b)
366. (d)	367. (e)	368. (d)	369. (e)	370. (e)	371. (c)
372. (e)	373. (d)	374. (a)	375. (e)	376. (e)	377. (c)
378. (e)	379. (c)	380. (a)	381. (c)	382. (d)	383. (a)
384. (e)	385. (a)	386. (a)	387. (b)	388. (c)	389. (d)
390. (e)	391. (c)	392. (e)	393. (e)	394. (e)	395. (c)
396. (e)	397. (e)	398. (c)	399. (b)	400. (a)	401. (b)
402. (b)	403. (c)	404. (a)	405. (a)	406. (a)	407. (b)
408. (e)	409. (a)				

(b) setting of cement in a bag
(c) strengthening of cement in a bag
(d) none of the above

28. Which of the following cements has maximum percentage of C_3S
(a) ordinary Portland cement (b) low heat cement
(c) sulphate resisting cement (d) rapid hardening cement

29. The resin-impregnated concrete
(a) reduces the porosity (b) increases the durability
(c) both of the above (d) none of the above

30. Workability of concrete can be increased by the
(a) increase in maximum size of aggregate
(b) decrease in temperature
(c) use of round aggregate which has smooth surface texture
(d) all the above

31. Higher percentage of C_3S makes cement
(i) hydrate more quickly
(ii) generate heat more rapidly
(iii) develop early strength
The correct answer is
(a) both (i) and (iii) (b) both (ii) and (iii)
(c) (i), (ii) and (iii) (d) none of the above

32. Which of the following statements is a correct statement
(a) rich mixes are less prone to bleeding than lean ones
(b) lean mixes are less prone to bleeding than rich ones
(c) bleeding is decreased by decreasing fineness of cement
(d) both (a) and (c)

33. White cement should have minimum percentage of
(a) silica (b) magnesium oxide
(c) calcium oxide (d) iron oxide

34. Bleeding can be reduced by
(a) addition of pozzolanas
(b) addition of aluminium powder
(c) increasing the fineness of cement
(d) all of the above

35. The increase in the strength of concrete with time is
(a) linear (b) non-linear
(c) asymptotic (d) all of the above

36. Calcium sulpho-aluminate is produced due to reaction of hydrated tricalcium aluminate with
(a) gypsum (b) water
(c) both of the above (d) none of the above

(c)	4	3	1	2
(d)	3	4	1	2

86. The nature of pigments used in coloured cement is chemically
    (a) inactive (b) active
    (c) alkaline (d) acidic

87. Efflorescence in cement is caused due to an excess of
    (a) alkalis (b) iron oxide
    (c) silica (d) alumina

88. Assertion A: In slurry infiltrated fibre concrete (SIFCON), there is substantial enhancement of tensile load carrying capacity
    Reason R: Fibres suppress the localization of micro-cracks into macro-cracks
    (a) both (A) and (R) are true and (R) is correct explanation of (A)
    (b) both (A) and (R) are true and (R) is an incorrect explanation of (A)
    (c) (A) is true and (R) is false
    (d) (A) is false and (R) is true

89. Match list-I with list-II and select the correct answer using the codes given below the lists
    List-I: (test)
    A. Vicat-apparatus B. Le-chatelier apparatus
    C. Slump test D. Fineness modulus
    List-II: (property)
    1. Soundness of cement
    2. Initial setting time of cement
    3. Workability of concrete
    4. Relative size of aggregates
    Codes:

	A	B	C	D
(a)	1	2	3	4
(b)	1	4	3	2
(c)	3	4	2	1
(d)	2	1	3	4

90. The product formed on hydration of C_2S, in the abbreviated symbols is
    (a) C_2SH (b) C_3SH
    (c) $C_2S_3H_2$ (d) $C_3S_2H_3$

91. The commercial name of white and coloured cements in India is
    (a) colocrete (b) silvicrete
    (c) snocem (d) all the above

92. Workability tests most suitable for concrete of very low workability are
    (i) vee-bee test (ii) slump test
    (iii) compaction factor test

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Reason R: The concrete mixtures are proportioned on the basis of 28-day compressive strength

(a) both (A) and (R) are true and (R) is correct explanation of (A)
(b) both (A) and (R) are true and (R) is an incorrect explanation of (A)
(c) (A) is true and (R) is false
(d) (A) is false and (R) is true

180. Epoxy injection technique is used for
(a) repairing the water retaining structures
(b) sealing of large cracks
(c) sealing the narrow cracks
(d) all of the above

181. Ready mixed concrete (RMC) is:
(i) specified in terms of performance parameters
(ii) produced under factory conditions
(iii) produced and supplied by weight
The correct answer is
(a) both (i) and (ii)
(b) both (i) and (iii)
(c) both (ii) and (iii)
(d) (i), (ii) and (iii)

182. For compaction of concrete, surface vibrator can be used for
(i) columns
(ii) slabs
(iii) raft foundation
The correct answer is
(a) only (i)
(b) only (ii)
(c) both (i) and (ii)
(d) both (ii) and (iii)

183. Number of samples for 31–50 m^3 of concrete work shall be
(a) 2
(b) 3
(c) 4
(d) 5

184. Match list-I with list-II and select the correct answer using the codes given below the lists

List-I: (property enhancement)

A. Self compactability
B. Freeze–thaw durability
C. Paste–refinement
D. Concrete–concrete bond

List-II: (material)

1. High-range water reducer
2. Epoxy resin
3. Mineral additive
4. Air-entraining admixture

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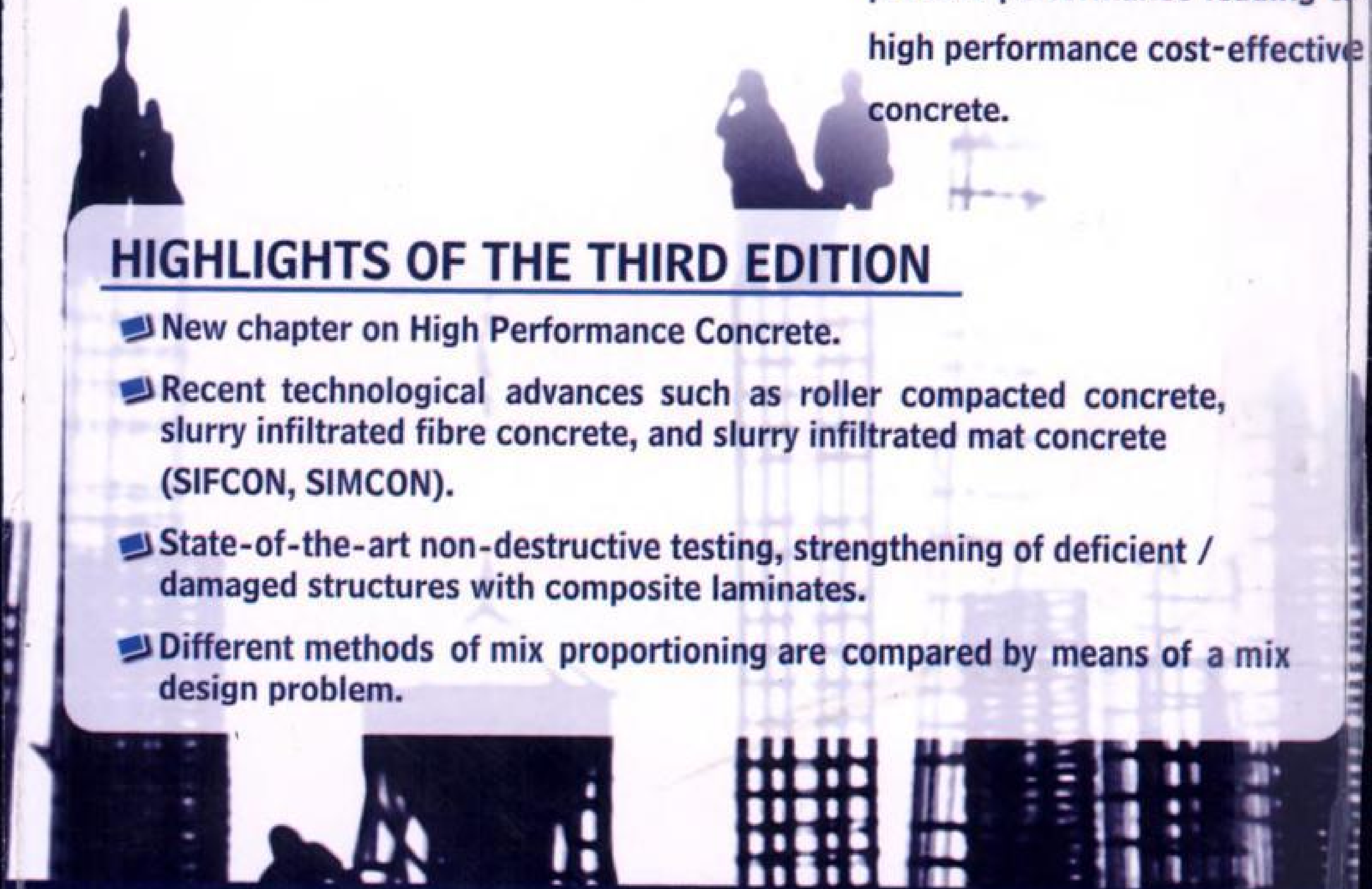

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