Foreword

National standards and codes in a country are the indication of growth and development. For so many years in Iran, great deal of efforts have been devoted to preparation of guidelines and codes of practice covering various technical and engineering disciplines.

The main objective to preparation of a standard is to present a collection of requirements and regulations with the aid of which relative problems may be resolved, and as stated in the introductory text of this code:

> The purpose of this code is to present the minimum required rules and regulations through observance of which a reasonable degree of safety, serviceability and durability can be ensured for structures concerned in this code.

The following issues regarding the present code shall be pointed out:

- In codification of this code the followings were considered:

  climatic conditions of the country, ease of application, and observance of the latest methods of analysis and design.

- The code titled “General Requirements and Conventional Buildings” consists of two parts:

  Part one is titled “General, Materials, and Construction Requirements” nine chapters;

  Part two titled “Principles of Design and Analysis” - eleven chapters.

- The abbreviation symbols used in this code are chosen in accordance with uniform symbols approved by the International Standard Organization(ISO).

- Specifications and standards quoted in this code are organized and numbered by the Technical, Criteria Codification and Earthquake Risk Reduction Affairs Bureau. Until the said codes are not codified and published by the Bureau, other equivalent standards may be used.

- In case the contractual texts are prepared in two languages , Farsi text shall govern.

- Translation of part one of the Iranian Concrete Code from Farsi into English is carried out partly by Dr. Fereidoun Amini and partly by Mr. Touradj Sadighian and is edited by Mr. Touradj Sadighian. Part two is done by Mr. Touradj Sadighian.

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State Management and Planning Organization
Office of Technical Affairs Deputy
Technical, Criteria Codification and Earthquake Risk Reduction Affairs Bureau
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PART ONE

General, Materials and Construction Requirements

PART TWO

Principles of Analysis and Design

CONCRETE CODE OF IRAN (CCI)

First Topic

General Requirements and Conventional Buildings

The Islamic Republic of Iran

State Management and Planning Organization

CONCRETE CODE of IRAN
(CCI)

Publication No. 120
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CHAPTER ONE

GENERAL

❒1.1 Objective

The objective of this code is to provide minimum requirements whereby an appropriate degree of safety, serviceability and durability of structures dealt with herein will be safeguarded.

❒1.2 Scope

1.2.1 The Provisions and instructions of this code shall apply for design, analysis, calculation, construction, controlling the specifications of constituent materials and construction quality of conventional concrete structures. The first two parts of this code contain requirements and regulations for reinforced concrete structures made with ordinary aggregates and portland or blended cement with a characteristic strength of at least 16 MPa (N/mm^2).

1.2.2 The provisions and regulations of this code, where applicable shall apply for specific structures and elements, including the following items. Other provisions specific to these structures form the subject of the future parts of this code:

a) Plain and under–reinforced concrete structures or members.
b) Reinforced concrete structures or members made of lightweight or heavy aggregates.

c) Reinforced concrete structures or elements made with aerated (cellular) concrete.
d) Prestressed concrete structures or elements.
e) Precast concrete elements or structures.
f) Composite steel-concrete structures or elements.
g) Fibre reinforced concrete structures or elements.
h) Reinforced concrete structures or elements exposed to high temperatures.
i) Special structures such as fluid storage tanks, silos, dams, blast–resistant structures, nuclear power plants, chimneys, bridges, and structures such as arches, which require specialized design.

❐1.3 Design bases

1.3.1 In this code, for attainment of safety and serviceability, the basis of design of structures and their control is that of " limit states " . The general design method is semi–probabilistic. Probability aspects are incorporated in calculations by applying partial safety factors to characteristic values of loads, actions effective on the structures, in accordance with loading codes, and characteristic strengths of concrete and steel.

1.3.2 Durability of structures is achieved through considering exposure conditions in the design, and selection of member shape appropriate to these conditions, observance of operational technical specifications such as quality, minimum cement content, water quality, water per cement ratio, type and quality of aggregates, maximum deleterious content of concrete, mix proportions, pouring and placement conditions, curing, cover layer thickness and construction joints.

❒1.4 Analysis bases

In this code, the permitted methods of analysis of structures and elements are as follows:

a) Linear analysis
b) Linear analysis with limited redistribution
c) Non – Linear analysis
d) Plastic analysis

❒1.5 Special provisions to secure safety against earthquake

Three ductility limits have been considered in this code as the response of reinforced concrete structures to earthquake induced effects

a) Low ductility limit
b) Medium ductility limit
c) High ductility limit

To fulfill the three ductility limits, low, medium and high, observance of special provisions presented in the chapter on "Special Considerations for Earthquake Resistant Design" is mandatory.

❒1.6 Units

The unit system adopted in this code is in compliance with the international units system (S.I.)

The units that have been used throughout this code are as follows:

a) length – Meter (m) and millimeter (mm)
b) Area – Square meter (m^2) and square millimeter (mm^2)
c) Concentrated load– Kilonewton (KN), and for distributed load Kilonewtons per meter (KN/m) and Kilonewtons per square meter (KN/m^2) which is equivalent to one kilopascal (KPa).
d) Specific mass (unit volume mass), kilogram per cubic meter (Kg/m^3).
e) Specific weight (unit volume weight) kilonewton/cubic meter (KN/m^3)

f) Stress and strength – megapascal (MPa) which is equivalent to newton per square millimeter (N/mm^2)
g) Moment – kilonewton per meter (KN/m)
h) Temperature – degree Celcius ($^\circ c$)

❐1.7 Symbols

All symbols used throughout this code are broadly in accordance with those used in the International Standards Organization (ISO).

❐1.8 Relevant codes and standards

Wherever this code is in conflict with the codes, guidelines and operational specifications of the Institute of Standards and Industrial Research or the Technical Criteria and Standards Bureau (T.C.S.B.) of the Management and Planning Organization of Iran, this code shall govern in all matters concerning design construction and material properties.

CHAPTER TWO

GENERAL REQUIREMENTS FOR SUBMISSION, INSPECTION AND DESIGN APPROVAL

❐2.1 Submission of design calculations, drawings and technical documents

2.1.1 Preliminary drawings for reinforced concrete structures shall be based on architects' drawings on which all dimensions, heights and other main particulars of the structure have been clearly shown. One copy of the same architects drawings on which structural design calculations have been based and bear the seal of the structural engineer together with structural drawing shall be submitted to the supervisory authorities.

2.1.2 The preliminary drawings which are submitted for approval shall be accompanied by the structural design calculations containing the following information:

a) Brief description of the main particulars of the building i.e its intended use, location, number of storeys and height.
b) Studies and assumptions made in determining soil bearing capacity, ground water level and other geotechnical aspects, as necessary.
c) Specification of materials used in the concrete such as steel, type of cement and characteristic strengths of concrete at stated standard ages or stages of construction for which each part of the structure has been designed.

d) Assumptions made in computation of dead loads, live loads including vertical loads, snow loads, wind loads, earthquake loads and generally all loads that have been used in the calculations.
e) Sketches of plans and loaded frames.
f) Methods used for analysis and design, stresses and the spatial factors and coefficients on which design calculation have been used.
g) Name and date of issue of all other relevant codes and supplements that have been used in design.
h) Detailed design calculations complemented with sketches and explanatory notes, with clearly identified results to facilitate checking.When automatic data processing is used, design assumptions and identified input and output data shall be submitted with the calculations.

2.1.3 Depending on the case, three types of drawings are prepared for construction of buildings:

2.1.3.1 Design drawings on which general arrangements and geometry of the structure , section sizes and cross-sectional area of steel have been shown.These drawings do not contain full construction details and must be upgraded to construction drawings prior to commencement of the works.

2.1.3.2 Construction drawings – in addition to the information shown on design drawings, construction drawings include construction details such as diameter, number and length of reinforcement steel, location of interruption and splices of reinforcement as to enable construction without ambiguity. Construction drawing for reinforced concrete structures shall be prepared by a competent structural engineer , in full consideration of the conditions listed below, and submitted to the checking authorities.

a) Drawings shall be prepared clearly and with acceptable scale.

b) Soil bearing strength taken as basis for design calculations, and the mechanical specifications of concrete and steel shall be stated.
c) Dimensions and location of all structural members, dimensions and location of all openings and punchings shall be given in the drawings.
d) All details and sections that are necessary for preparation of site drawings, diameter of reinforcement bars, location of bending, cutting and splicing of reinforcement and all relevant dimensions shall be provided. Part of the information may be given in the bar table.
e) The thickness of concrete cover on reinforcement steel, the diameter of the largest usable aggregates in concrete and maximum water per cement ratio shall be given in the drawings.
f) The location of the expansion, contraction and construction joints and their details shall be in the drawings.
g) Preparation of bar tables and determination of weights of different types of reinforcement are among obligations of the design engineer to the client. However, meeting the obligations is not mandatory to acquire construction permit, unless some of the information regarding reinforcement is not shown on the construction drawings, but is only shown on these tables.

2.1.3.3 Site drawings-these drawing are prepared for special and critical parts of the structure. They are based on details shown on construction drawings and are drawn to a large scale. These drawings must be prepared according to the needs of the site as construction works proceed and must be confirmed by the inspecting engineer.

❐2.2 Inspection

2.2.1 Concrete construction shall be inspected throughout the various stages of construction by competent engineers. It is recommended that inspection is carried out by the structural engineer or his/her competent representatives.

2.2.2 There must always be kept on site a log in which records of the following are kept:

a) Quality and proportions of concrete materials.
b) Dates of construction and removal of forms, placing of reinforcement and placing of concrete.
c) Weather conditions such as ambient temperatures and rain.
d) Results of tests carried out on various samples.
e) Any significant construction loading on completed floors, walls and other members.
f) General progress of work.
g) The site log can be substituted by daily reports containing the above information.

2.2.3 When the ambient temperatures falls below $5\ ^{\circ}$c or rises above $30\ ^{\circ}$c, a complete record shall be kept of concrete temperatures and of protection given to concrete during placement and curing.

2.2.4 The site log (or the daily reports) shall be signed daily by inspecting engineer and shall be kept on site and be available to the building officials during progress of work.

The site log together with final drawings should be kept and preserved by the project owner (i.e. ownership documents) after completion of the project. Computerized storage of these information is required for major buildings.

2.2.5 Building officials must report to inspecting engineer in charge, any incompliance with construction drawings and technical principles during progress of work. If the defects are deemed to endanger the safety of the structure , building officials must seek the advice of the preliminary technical commission immediately (see note 2).
The preliminary technical commissions shall be set up on site at once and, if necessary, will issue instructions to stop all or part of the work and will refer the case to the final technical commission for full and final assessment. The final technical commission will consider the relevant issue s, and, if necessary, will carry out on-site assessments, will take down details of existing member dimensions , will order necessary tests to examine quality of materials and safety, and makes appropriate decisions.
Note 1 : Building officials are competent representatives who are authorized by the laws and regulations of the country and local municipality to inspect design and construction , and are generally persons who are responsible for the supreme inspection and enforcements of this code.

Note 2: Members of the preliminary and final technical commissions will be appointed according to the laws and regulations of the country and local municipality. In their absence, the client , in agreement with the inspection system, will select persons from among expert engineers.

❐2.3 Load tests

2.3.1 When the conditions of the structure are such that building officials doubt its safety , and when assessment by technical calculations does not lead to removal of ambiguity and doubt about safety, building officials, through the preliminary technical commission and upon confirmation by the final technical commission , can instruct load testing of all or the part of the structure that is considered questionable.

2.3.2 A load test shall be made under control of the technical commissions not less than 8 weeks after construction of the part in question. When the engineer, the client, the contractor , and all involved parties mutually agree, the test may be made at an earlier age.

❐2.4 Approval of special systems for design and construction

Sponsors of any newly developed and proposed system of analysis and design, safety assessment or construction, application of which is not covered by or does not conform to the criteria of this code, but their validity has been confirmed by scientific justification, can ask the T.C.S.B. to take the matter to a technical committee composed of T.C.S.B. experts and at least three other experts in the relevant field. The technical committee, after considering the case and making the necessary tests and compiling theoretical justifications will make a judgment on accepting or rejecting the proposals. In the case of acceptance , the committee will make a decision and issue its requirements and conditions. The requirements and conditions issued by the committee shall be deemed to be those of this code.

Assessment of safety of special elements by tests and without technical calculations may be acceptable subject to approval of the proposed test and test method by the technical committee.

CHAPTER THREE

CONSTITUENT MATERIALS OF CONCRETE

❒3.1 Selection and approval of materials

3.1.1 Design considerations

Materials used in concrete should satisfy the design requirements for the structural performance, durability, safety and appearance of the finished structure, taking full account of the environment to which it will be subjected. The amount of potentially harmful impurities in the constituent materials should be taken into account so that the maximum permitted cumulative contents is not exceeded.

3.1.2 Constituent materials specifications

3.1.2.1 Generally , materials to be used shall comply with the relevant standards in this code. Where necessary, certificates of compliance shall be provided.

3.1.2.2 Materials not covered in the list of standards for testing may be used to make concrete, provided full account is taken of their effects on design requirements and that there are satisfactory data on their suitability and assurance of quality control. previous experiences along with the results of such materials should be considered and incorporated in the project with respect to specific characteristics, and the comments of the inspection engineer.

Where necessary, records concerning the details and performance of such materials, together with the construction drawings should be maintained by the client.

❒3.2 Test of materials

3.2.1 Inspecting engineers shall have the right to order further tests than those anticipated in the specifications of any material used in concrete construction to determine if materials are of the quality specified.

3.2.2 Tests shall be made in accordance with codes and criteria of the Technical Criteria and Standards Bureau of the Management and Planning Organization of Iran (T.C.S.B.). And observance of the issues in chapter five of this code.

3.2.3 A complete record of materials tests shall be kept by the inspection system until the end of guarantee period and for at least one year after completion of the project and shall be delivered to the project owner. Computerized storage of this information is required for major buildings.

❒3.3 Cement

3.3.1 The cement used in the construction of load–bearing elements shall conform to one of the following specifications or any other standards approved by the consulting engineers:

a)“Specification of Portland Cement” (T.C.S.B.101)

b)“Specifications of the blended slag Portland Cement” (T.C.S.B. 102)

c)“Specifications of the blended pozzolanic Portland Cement” (T.C.S.B. 103)

d)“Specifications of the blended calcareous Portland Cement” (T.C.S.B. 129)

❐3.4 Aggregates

3.4.1 Concrete aggregates shall be of such quality as to produce a strong and durable concrete.

3-4.2 Concrete aggregates shall conform to "Specification for concrete Aggregates" (T..S.B. 201)

3.4.3 Aggregates failing to meet the specifications in clause 3.4.2, but which have shown by special tests or by their actual performance the ability to produce concrete with characteristics of the fresh concrete and with adequate strength and durability, may be used in making concrete; with authorization of the inspection system.

3.4.4 Nominal maximum size of coarse aggregate shall not be larger than:

a) $\frac{1}{5}$ the smallest internal dimension of the concrete forms

b) $\frac{1}{3}$ of the depth of slabs

c) $\frac{3}{4}$ the minimum clear spacing between individual reinforcing bars

d) $\frac{3}{4}$ the thickness of the bar cover

Note 1 The size of the smallest sieve on which, maximum ten percent by weight of the aggregates remains is called the nominal size of aggregates.

Note 2 Application of coarse aggregates larger than 38 mm is not recommended in making reinforced concrete, but the size of coarse aggregates shall not exceed 63 mm.

3.4.5 Harmful materials in aggregates

3.4.5.1 General

Aggregates used in concrete shall be hard and durable and the amount deleterious material content shall not exceed allowable limits in 3.4.5.2 and 3.4.5.3.

3.4.5.2 Deleterious materials in fine aggregates used in concrete shall not exceed the maximum limits prescribed in table 3.4.5.2, and the method of tests for any deleterious materials shall be in accordance with the same table. Furthermore, the sand value of fine aggregates with respect to T.C.S.B. 232 shall not be less than 75 percent.

Table 3.4.5.2 Limits of the maximum allowable amount of deleterious materials in fine aggregates and test methods

Maximum allowable content expressed as a percentage of weight of sample	Test method	Deleterious material
3	T.C.S.B. 221	Clay packs and weak particles
3 5	T.C.S.B. 218	Particles passing sieve No:200(0.075mm) - Concrete subject to abrasion - Other concretes
0.5 1	T.C.S.B. 219	Coal, lignite , or other lightweight materials: - When concrete appearance is important - Other concretes
1	-	Mica
0.4**	T.C.S.B. 230	Sulphates expressed as $(So3^{--})$
0.04+	T.C.S.B. 231	Chloride expressed as (Cl^{-})

* For crushed sand, if the particles passing through sieve No. 200 are comprised of stone dust and are free of clay, the quantities may be increased to 5 and 7 percent respectively. These percentages are computed based on the grading of the sand passing through 4.75mm sieve (No.4).

** The total soluble sulphate in the concrete mix expressed in terms of $So3^{--}$ in cement shall not exceed 5 percent of the weight of cement.

+ Chloride content soluble in the concrete mixing water , expressed in terms of percentage of cement weight shall not exceed the maximum limit allowed in table 6.3.3.6.

3.4.5.3 Deleterious materials in coarse aggregates shall not exceed the limits given in table 3.4.5.3 and method of testing shall comply with the requirements of the same table.

Table 3.4.5.3 limits of deleterious material content of coarse aggregates and test methods

Maximum allowable content expressed as a percentage of weight of sample	Test method	Deleterious material
0.25	T.C.S.B. 221	Clay packs
5	T.C.S.B. 223	* Soft particles
 1 3 5		** Chert as an impurity - In severe exposure conditions - In moderate exposure conditions - In mild exposure conditions
1^{+}	T.C.S.B. 218	Aggregates passing sieve No. 200(0.075mm)
 3 5 7	-	Frible particles including clay lums, Soft particles, Weathered chert , Shales and weathered laminated schist. - Exposed concrete - Concrete subject to abrasion - Other concretes
0.4^{++}	T.C.S.B. 230	Sulphates ($So3^{--}$)
$0.02^{\neq}$	T.C.S.B. 231	Chlorides (Cl^{-})

* This limitation applies only in cases where the softness of each aggregate particle is critical with respect to the required concrete services, such as in heavily trafficked surface is of special importance.

** This type of chert disintegrates after 5 cycles of soundness test or 50 cycles of alternate freezing test, or when its specific gravity in the saturated state with dry surface is below 2.35 Disintegration to fracture or breaking is based on actual tests. This limitation is only applicable to aggregates in which chert is considered an impurity and is not applicable to aggregates that are mostly composed of chert.
Soundness limitation of aggregates must be based on their service records in the exposure conditions under consideration. For categories of exposure conditions see clauses 8.2.9.1.

+ In the case to crushed aggregates, if the particles passing sieve No.200 are comprised of stone dust and are free from clay or shale, the percentage can be increased to 1.5%.

++ The total quantity of soluble sulphate expressed as $So3^{--}$ in the concrete mix, including $So3^{--}$ content of cement shall not exceed 4 percent , and in any case the total sulphate content shall not exceed 5 percent of weight of cement content.

+++ The total amount of soluble chloride in the concrete mix, expressed as percentage of weight of cement content , shall not exceed the limit specified in clause 6.3.3.6.

3.4.5.4 Flaky and prismatic aggregates

Percentages of flaky or prismatic coarse aggregates shall not exceed the values given in the table 3.4.5.4.

Table 3.4.5.4 Maximum allowable amounts of flaky and prismatic particles in coarse aggregates

Type of aggregates	Test method	Maximum allowable percent by weight
Flaky particles:		
Aggregates left on 6.3mm sieve	T.C.S.B. 220	30
Prismatic particles:		
Aggregates with maximum size 63 or 50mm	T.C.S.B. 220	35
Aggregates with maximum size 38 or 25 or 19mm	T.C.S.B. 220	40
Aggregates with maximum size 12.5 or 9.5mm	T.C.S.B. 220	45

❒3.5 Water

3.5.1 Water used to wash aggregates or to make and cure concrete shall be clean, clear and free from injurious amounts of oils, acids, alkalis, salts, sakaroids, organic materials, or other substances that may be deleterious to concrete or reinforcement. Generally, drinking water is considered satisfactory for making and curing concrete. Water not fit for drinking may only be used when in compliance with clause 3.5.2. The amount of deleterious materials in concrete mixing water shall not exceed the limits specified in table 3.5.1 method of test for any deleterious materials shall comply with the requirements of the same table.

3.5.2 Non-drinking water shall not be used in concrete unless it complies with requirements of clauses 3.5.2.1, 3.5.2.2 and table 3.5.1

3.5.2.1 Selection of concrete proportions shall be based on concretes mixes using water that is used on site.

Table 3.5.1 Maximum allowable amounts of deleterious materials in concrete mixing water and test methods

Deleterious materials	Description	Test method	Maximum allowable viscosity (Parts per million)
Suspended solid particles	- Reinforced concrete exposed to severe exposure conditions and prestressed concrete - Reinforced concrete in mild exposure conditions and plain concrete	**T.C.S.B. 305**	**1000** **2000**
Dissolved materials	- Reinforced concrete exposed to severe exposure conditions and prestressed concrete - Reinforced concrete in mild exposure conditions - plain concrete, and concrete without any embedded metal items	**T.C.S.B. 305**	**1000** **2000** **3500**
Chlorides (Cl^{-})	- Reinforced concrete in severe exposure conditions , prestressed concrete and bridge deck concrete - Other reinforced concrete cases in damp conditions or with aluminum or unsoiled metals, or with galvanized permanent shuttering - Plain concrete without embedded metal items	**T.C.S.B. 306**	**500*** **1000*** **10000**
Sulphates (So_4)	- Reinforced concrete and prestressed concrete - Plain concrete without embedded metal items	**T.C.S.B. 307**	**1000*** **3000****
Alkalis	$(Na_2o + 0.658K_2o)$	**T.C.S.B. 304**	**600**

* The total amount of chloride ion soluble in concrete mixing water , expressed as a percentage of the cement weight, shall not exceed the maximum limits specified in table 6.3.3.6

** The total concentration of soluble sulphates in concrete mix including $So3^{--}$ in cement , shall not exceed 4 percent. In any event the total concentration of sulphates shall not exceed 5 percent of the weight of cement. Also see table 6.3.3.3 for concrete exposed to sulphates in various exposure conditions.

3.5.2.2 Test of mortars made with non-drinking water, according to T.C.S.B. 119 shall have 7 day and 28 day strengths equal to at least 90 percent the strengths of similar tests made with distilled water.

3.5.2.3 Determining the set time of the cement with non-drinking water shall be done according to T.C.S.B. 113. The test result shall not be more than one hour earlier nor 1.5 hours later than results given by the distilled water.

3.5.2.4 Test of cement soundness with non-drinking water shall be done according to T.C.S.B. 123 and the results shall not exceed allowable limits for distilled water.

3.5.2.5 Tests for these comparisons shall be made on identical condition except for the water used.

3.5.3 The level of pH. in mixing water shall not be less than 5 or greater than 8.5. Otherwise necessary test shall be made in accordance with the requirements of 3.5.2.2 through 3.5.2.4. pH test shall be in accordance with T.C.S.B. 303.

❒3.6 Admixtures

3.6.1 Definition

Admixtures are materials other than Portland cement , aggregates or water, in the form of powder or liquid, used as concrete constituent materials , in order to improve certain properties of concrete , and are added just before or during mixing. Admixtures are divided into two chemical and mineral groups.

3.6.2 General

3.6.2.1 Admixtures to be used in concrete shall be subject to prior approval by the engineer.

3.6.2.2 The suitability and effectiveness of any admixtures shall be verified by trial mixes of the concrete to be used in the works , prior to its use.

3.6.2.3 Admixtures shall be in compliance with characteristics in T.C.S.B. 401 through T.C.S.B.404, and T.C.S.B. 408 through T.C.S.B. 410.

3.6.2.4 Admixtures shall be compatible with the cement used. If more than one type of admixtures are to be used simultaneously in the same concrete mix, data shall be provided by the inspection system to assess their interaction and compatibility.

3.6.2.5 Measurement of dosage of admixtures should be precise. If two or more admixtures are to be used , their measurement shall be carried out individually.

3.6.2.6 All admixtures shall be capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions in accordance with clause 6.4.

3.6.2.7 Calcium chloride shall not be used when making reinforced concrete, also see 6.3.3.6.

3.6.3 Chemical admixtures

3.6.3.1 Air-entraining admixtures

a) Definition and application

Air–entraining admixtures produce tiny air bubbles which are spread evenly throughout the volume of concrete or mortar. These bubbles should remain in the concrete or mortar after hardening.

Air bubbles enhance durability of concrete against moisture and frequent freezing and thawing, and increase the resistance of hardened concrete to surface scaling caused by de–icing salts.
Moreover, use of air–entraining admixtures considerably increases the workability of fresh concrete and impermeability of hardened concrete, while at the same time reducing segregation of aggregates and bleeding.
b) Specifications:
Specifications of air–entraining admixtures shall conform to T.C.S.B. 402.

3.6.3.2 Water – reducing admixtures
a) Definition and application
Water-reducing admixtures are used for the purpose of reducing the water content for even conditions of flowing concrete or improved workability for the same water content. There are two types of water reducing materials, ordinary and strong.

b) Specifications:
Specifications of water-reducing admixtures shall conform to T.C.S.B. 401.

3.6.3.3 Retarding admixtures

a) Definition and application
   Retarding admixtures are used for the purpose of slowing down the setting rate of concrete and are divided in three types: retarding, retarding with ordinary water reducing, and retarding with strong water reducing.
b) Specifications
   Speeification of retarding admixtures shall conform to (T.C.S.B. 401).

3.6.3.4 Accelerating admixtures

a) Definition and application

Accelerating admixtures are used for the purpose of increasing the rate at which concrete sets, or for accelerating strength development at an earlier age, or both. There are two types of regular accelerators, and accelerators with ordinary water reducing agents.

b) Specifications

Specifications of Accelerating admixtures shall conform to T.C.S.B. 401

c) Calcium chloride

The use of calcium chloride as an accelerating admixture is permitted for plain concrete only, and its specifications shall conform to T.C.S.B. 401. The quality of calcium chloride shall not exceed the dosage necessary for achieving the required result and in any case the concentration shall not exceed 2 percent of the weight of cement.

3.6.3.5 Plasticizers

a) Definition and application

Plasticizers are used for the purpose of improving workability of fresh concrete. Hence, plasticizers are used for improving harsh mixtures, making trowel finishable concrete, concreting heavily reinforced members, pumping concrete, and concreting with tremie. This category of admixtures includes air–entrainers, pozzolans, plasticizers and superplasticizers. Superplasticizing admixtures produce highly workable concrete.

b) Specifications

Specifications for plasticizers and superplasticizers should be in accordance with 3.6.3.1. b, 3.6.4.2. d and the specifications of T.C.S.B. 408.

3.6.4 Mineral admixtures

These very fine mineral particles improve some of the properties or provide special characteristics in concrete. Mineral admixtures can improve workability and consistency of fresh concrete, strength and permeability of hardened concrete and also change its color. There are three types of mineral admixtures.

3.6.4.1 Neutral mineral admixtures and pigments

These materials, like grinded quartz usually tend not to increase the concrete strength by chemical reactions. Usage of these admixtures leads to improved workability and adhesion of concrete with shortage of fine materials. These materials usually fulfill concrete aggregate requirements and are used as aggregates in concrete.

Pigments too are used as neutral admixtures for producing colored concrete. These mineral mixtures like (Ferric oxide) which is used for red, brown, and yellow colors, and (Cromic oxide) which is used for the green color shall be resistant to light and alkalis and shall not interfere with hydration reactions of cements.

3.6.4.2 Pozzolans

a) Definition

   Pozzolans comprise of silicoius or silicoius and alumine which have little or no bonding value if used alone, but in the form of very fine particles in a damp media and at normal temperature, they chemically react with calcium - hydroxide to produce cementitius materials.

b) Applications

Use of pozzolanic materials in concrete is to provide for one or more of the following characteristics:

- Reduce cement content
- Reduce rate and quantity of heat produced from the cement hydration process
- Improve concrete workability
- Increase concrete strength

-Increase concrete durability by reducing permeability
Application of pozzolans for every one of the above properties shall be verified before usage

c) Types

There are two types of pozzolans:
-Raw natural or calcined pozzolans which mainly include volcanic ashes
-industrial pozzolans which mainly include fly ash and silica fume

d) Specifications

Pozzolans used as mineral admixtures in mortar or concrete shall conform to one of the following specifications or any other standards which is approved by the inspection system before usage
-All kinds of raw natural or calcined pozzolans and fly ash (T.C.S.B. 403)
-Silica fume (T.C.S.B. 409)

3.6.4.3 Cement-like admixtures

These materials have hidden hydraulic characteristics and they gain cement properties when become properly active. These admixtures react with water in a similar manner to the portland cement only in alkaline environment and their chemical compositions are more similar to ordinary cement in comparison with fly ash and other pozzolanic materials. The most common type of such materials is the blast-furnace slag which shall be in compliance with specifications in T.C.S.B. 410.

3.6.5 Miscellaneous admixtures

Some of the miscellaneous admixtures include: damp proofing materials, permeability-reducing materials, grout or slurry producing materials, gas producing materials, and de-icers. Miscellaneous admixtures and their use shall conform to the relevant specification and the general requirement for admixtures and with approval of the inspection system.
If de-icer is used in concrete, in addition to the above points, specifications of cold weather concreting shall be observed (Refer to 7.6.3).

❐3.7 Storage of constituent materials of concrete

3.7.1 Portland cement shall not be stored in contact with moisture, but it shall be kept dry to prevent its deterioration. In dry areas a maximum of 12 bags may be stacked on top of each other if their height does not exceed 1.8m.
Based on their types, cements shall be kept separately in waterproof and airproof silos, and shall not be piled stagnated in the silo.

3.7.2 Aggregates shall be stored so as to avoid segregation in pile and contamination by deleterious materials; they shall be kept separately based on their type, size and grading. If the maximum size of aggregates is less than 38 mm, they shall be divided into at least two groups of fine and coarse aggregates. Aggregates with maximum size of 63 mm. shall be divided at least into three separate groups. Aggregates shall be properly stored so as to prevent from freezing and cumulation of snow and ice among particles.
For hot weather concreting conditions, aggregates shall be properly stored so as to be safe from direct sun radiation and that their temperature does not increase.

3.7.3 Admixtures shall be kept in proper conditions based on the requirements prescribed by the producer. Some admixtures degrade if stored for a long period. If the state of the admixture is in doubt, test shall be made in accordance with standard specifications.

3.7.4 Any material that has deteriorated or has been contaminated shall not be used for making concrete.

❒3.8 Control and inspection

To ensure conformance of the materials specifications with requirements and standards of this code, minimum inspections and tests shall be carried out in accordance with table 3-8.

Table 3-8: Inspection on quality control of concrete materials

No.	Materials	Inspection-test	Objective	Inspection and test period
1	**Cement** (1)	**Inspecting the consignment documents of cement delivered to the site** (2)	**Conformity of cement consignment to purchase order and control the specifications given by the cement factory**	**For each consignment delivered to the site**
2	**Aggregates** (3)	**Inspecting the documents of aggregates delivered to the site**	**Conformity of cement consignment to purchase order and control the specifications given by the cement factory**	**For each consignment delivered to the site**
3	**Aggregates** (3)	**Inspecting the external conditions of aggregates**	**Evaluation of external condition regards to its size, shape of aggregates and impority**	**For each consignment delivered to the site**
4	**Aggregates** (3)	**Grading test**	**Comparison of test results with specifications standards and requirements for aggregates**	**1- first consignments from source and for new mine 2- If there is double. After any observation. 3- one every day**
5	**Aggregates** (3)	**Test for evaluation of sand content**	**Comparison of test results with requirements and standards**	**Same as No.1 and 2 mentioned in this column and at least once a week**
6	**Aggregates** (3)	**Test for deleterious material content**	**Finding content of deleterious material in aggregates and its specification**	**Same as No.1 and 2 mentioned in this column**

Table 3-8: Inspection on quality control of concrete materials

No	Materials	Inspection-test	Objective	Inspection and test period
7	**Aggregates** (3)	**Test for moisture content of concrete**	**Water content in concrete mixture**	**Once a day and in necessary conditions** (4)
8	**Aggregates** (3)	**Unit weight test**	**Unit volume weight measurement**	**Same as No.1 and 2 mentioned in this column**
9	**Chemical** (5) **admixtures**	**Inspecting the documents of delivered material to the site and the label on their pakages and performing standard tests**	**Assurance of conformation of materials with the purchase order and control signs and labels, and comparison of the test results with written specifications in corresponding standards**	**For each delivered consignment to the site**
10	**Chemical** (5) **admixtures**	**Inspecting and observation of external conditions**	**Comparison with external conditions**	**For each consignment and when necessary, according to the views of the inspection system**
11	**Chemical** (5) **admixtures**	**Weight of unit volume test and PH number**	**Comparison with the specification given by producer**	**At least once a week and when necessary, according to the views of the inspection system**
12	**Emulsion admixtures**	**Inspecting the documents of delivered material to the site**	**Assurance of conformation of material with purchase order and the name of producer factory according to written proforma**	**For each delivered consignment to the site**
13	**Emulsion admixtures**	**Weight of unit volume lest**	**Assurance on homogenity of material**	**For each delivered consignment to the site**

Table 3-8: Inspection on quality control of concrete materials

No	Materials	Inspection-test	Objective	Inspecting and test period
14	**Admixtures** (in powder form) [5]	Inspecting the documents of material delivered to the site	Assurance on conformation of material with purchase order with regards to specification, quality and the name of producer factory according to the written proforma	For each delivered consignment to the site
15	**Water**	Chemical analysis test based on relevant standards	Assurance on lack of deleterious impurity materials in water	Only when drinking water is not drawn from potable sources or when water is taken from new and doubtful sources
16		Mortar test based on relevant standards	To compare the results of standard tests with those of consumed water and distilled water	Only when drinking water is not drawn from potable sources or when water is taken from new and doubtful sources

1- At least once a month or for each 100 tons of cement, whichever happens earlier. Sampling and testing should be done for each of the cement types. In addition to that for each consignment delivered to the site, it is necessary to keep three kilos of labeled sample for four months for future testes when necessary.

2- For each consignment delivered to the site, specification of factory and cement type should be written in delivery receipt.

3- For any change of the mine or source of aggregate delivered to the site some informations regarding to maximum chloride (Cl^-) and sulphate content (So_3) soluble in the internal water of the materials and also probable capability and sensivity of aggregates to alkali reactivity shall be specified.

4- This test shall be done on aggregates stored in the location of concrete production and at the time of production.

5- Sampling shall be done for each delivered consignment to the site. Samples shall be kept for necessary future tests.

CHAPTER FOUR

METAL REINFORCEMENT

❒4.0 Notation

E_s =Modulus of elasticity of steel bars , MPa (N/mm^2)

f_{su} = Tensile strength of steel bars, MPa (N/mm^2)

f_y =Yield stress of steel bars , MPa (N/mm^2)

f_{yk} =Characteristic strength of steel bars , MPa (N/mm^2)

$f_{y,obs}$ =Elasticity limit of steel bars obtained from tensile tests , MPa

f_{ym} = Average yield stress of ten test samples, see equation 4.3.

s_{10} = Standard deviation for yield stress of ten test samples, see equation 4.4.

❒4.1 General

Any type of steel, either as bars or wires, to be used in concrete as reinforcement shall be manufactured to specific standards and shall have the manufacturer's identification certificate.

❒4.2 Types of steel

Types of steel used in reinforced concrete with respect to production technique, surface form, weldability and ductility are as follows:

4.2.1 Production technique

a) Hot rolled steel (Hot rolled)
b) Cold-worked steel by mechanical operations such as twisting, stretching or rolling (Cold-worked)
c) Special steel, work hardened by quenching and heating (tempered)

4.2.2 Surface form

a) Deformed bar
b) Plain bar

4.2.3 Weldability

In terms of weldability, steel is classified into three grades:

a) "Readily weldable Steel", which can be welded by common methods and equipment”
b) “Weldable steel", which can be welded under controlled conditions with the use of special methods and techniques
c) “Unweldable steel", which cannot be welded by ordinary methods

4.2.4 Ductility

In terms of ductility, steel is classified into three grades:

a) Mild steel with stress-strain curve showing a very clear yielding step.
b) Semi-hardened steel with stress-strain curve showing a very limited yielding step.
c) Hardened steel with stress-strain curve not showing a yielding step.

❒4.3 Nominal diameter

4.3.1 Steel is manufactured and marketed for use as strands, bars and welded mesh fabric, and are separated according to the nominal diameter.

4.3.2 The nominal diameter of a bar is defined as the diameter that is given in the identification certificate and is equivalent to the diameter of a circle of the same area as the theoretical cross-sectional area of the bar, in millimeters.

4.3.3 The nominal diameter , nominal surface area and nominal cross-sectional area of a deformed bar, area equal to the diameter, surface area and cross-sectional area of a plain bar of the same weight.

4.3.4 Calculation of weight, surface area and cross-sectional area of bars shall be based on their nominal diameters and unit mass of 7850 Kg/m^3.

4.3.5 The nominal diameters of bars range from 5 mm to 50 mm at different increments and those of welded mesh fabrics range from 4mm to 12mm, at 0.5mm increments.

4.3.6 Preferred nominal diameters for use in reinforced concrete are : 5,6,8,10,12,16,20,25,32,40,50

❒4.4 Mechanical properties

4.4.1 f_y =Steel yield stress

f_{yk} = Steel characteristic strength

The characteristic strength of steel shall be based on yield stress and is defined as the strength below which 5 percent of all possible yield strength test results would be expected to fall. In cases where the yield stress of steel is not clearly known, it shall be taken as the stress equivalent to 2 percent of the permanent deformation.

Tensile test results of every sample shall satisfy the following equations:

$$f_{su} \geq 1.25\ f_y \qquad f_{su} > 1.18\ f_{y,obs} \tag{4-1}$$

4.4.2 Classification of bars

Steel bars are classified according to characteristic strength. The classification of bars used in reinforced concrete, based on the type of steel are as follows :

S220 , S300 , S350 , S400 , S500

The figures after s indicate the minimum characteristic strength of bars, in MPa.

4.4.3 Tests

Testing of steel bar samples, in consideration of clauses 4.4.3 and 4.4.4, shall conform to the Technical Criteria and Standards Bureau of the Management and Planning Organization of Iran (T.C.S.B.) standards listed below:

- Tensile test on reinforcement bars (T.C.S.B. 701)
- 180^0 Folded test(T.C.S.B. 703)
- Rebend test on reinforcement steel(T.C.S.B. 703)
- Tensite test after rebending of bars and wire less than 9mm in diameter (T.C.S.B. 702)
- Bond test for reinforcement steel (pull-out test)(T.C.S.B. 705)
- Requirements for the control of weld joints in reinforcement (T.C.S.B. 706)
- Fatigue test on concrete-reinforcing steel (T.C.S.B. 707)

Note: Tensile testing of all steel bars and rebend testing of all cold-worked steel bars used in reinforced concrete are mandatory.

4.4.4 Sampling

Strength and other characteristic properties of steel are based on testing of samples cut from them. In every sampling there shall be cut a 1 meter long piece, from which test samples shall be taken.

4.4.5 Frequency of sampling

The number and frequency of samples shall be such as to enable assessment of the quality of reinforcement to be used. For this purpose for up to and including every 50 tonnes of every diameter of every type of steel used, at least 3 samples shall be taken. Upon agreement with the engineer, from every 3 bundles of 5 tonnes of similar steel, one sample shall be selected.

4.4.6 Acceptance requirements of steel bars.

Characteristic strength of steel shall be considered to be the required classification and acceptable when in addition to the requirements of clauses 4.4.1 and 4.6, one of the following conditions is satisfied:

4.4.6.1 Tensile test results of five consecutive samples shall all be greater than the required characteristic strength.

4.4.6.2 If the requirements of clause 4.4.6.1 were not satisfied, another group of five samples shall be tested. The total ten results shall satisfy the following equation:

$$f_{ym} \geq f_y + 0.6S_{10} \qquad (4\text{-}2)$$

Where:

$$f_{ym} = \frac{f_{y1} + f_{y2} + ... + f_{y10}}{10} = \frac{\sum f_{yi}}{10} \quad , \text{i=1 to 10} \tag{4-3}$$

$$S_{10} = \sqrt{\frac{(f_{ym} - f_{yi})^2}{9}} \quad , \text{i=1 to 10} \tag{4-4}$$

If the requirements of clauses 4.4.6.1 and 4.4.6.2 are not met, the characteristic strength of the steel shall not be considered to be to the required standard.

❒4.5 Deformations

4.5.1 Stress-strain diagram

For simplification of calculation, the actual stress-strain diagram of steel may be replaced with a twin-lined diagram. Steel stress-strain diagram is considered to be the same for both tension and compression.

4.5.2 Modulus of elasticity

The modulus of elasticity for all steel bars used in reinforced concrete shall be taken as 200000 MPa.

4.5.3 Coefficient of thermal expansion

The coefficient of thermal expansion for all steel bars used in reinforced concrete may be taken as 1×10^{-5} per C°.

❒4.6 Ductility

Ductility of steel bars is based on"Rebend test" or 180^0 fold test" results. Ductility of steel bars shall be considered acceptable when partial lengthening of steel at the point of failure in the tensile test does not fall below 8 percent of ten times the bar diameter or 12 percent of five times the bar diameter.

❒4.7 Weldability

Steel weldability depends on its production method and chemical composition. All hot-rolled steel of normal chemical composition have good weldability properties. Cold-worked steel and tempered steel are sensitive to the heat generated by welding and cannot be welded by common techniques.Weld splicing of such steels can only be carried out under certain conditions which are different for different steel.

Welding of steel bars shall conform to sub clause 8.2.5.3.

❒4.8 Storage, preservation and control of steel

4.8.1 Steel bars shall be stored and kept in clean and damp-free areas to prevent corrosion and contamination.

4.8.2 steel bars which have scaled in depth due to corrosion, cannot be used in reinforced concrete unless they are tested and verified for safety and for the required characteristics and by taking into consideration the probability of reduced cross-sectional area.

CHAPTER FIVE

STANDARDS FOR SPECIFICATION AND TESTS

❒5.1 General

The serial numbers of the standards cited in this code have been designated by the Management and Planning Organization, Technical Criteria and Standards Bureau and have been classified under such headings as “specifications and tests for cement and mortar”, “Aggregates”, “Water”, “Admixtures”, “Fresh concrete”, “Hardened concrete” and “Steel”; and are contained herein. Until such time that the above standards are prepared by the Bureau, the official standards published by the Institute of Standards and Industrial Research of Iran (ISIRI), or the documents of the International Standards Organization (ISO), or the standards of the American Society for Testing and Materials (ASTM) may be used.

❒5.2 Standards connected with this code

These standards which are numbered by “Management and Planning Organization, Technical Criteria and Standards Bureau” are listed briefly in table 5.2. More comprehensive details are given in the commentaries of chapter five in the form of separate tables.

Table 5.2 Standards for specifications and test methods connected with this code

No.	Title	Standard	Technical Criteria and Standards Bureau (T.C.S.B)
1	Cement and mortar	Specifications	101 to 104 and 128,129
		Test	105 to 127
2	Aggregates	Specifications	201 to203
		Test	204 to 232
3	Water	Specifications	301
		Test	302 to 308
4	Admixtures	Specifications	401 to 404 and 408 to 410
		Test	405 to 407
5	Fresh Concrete	Specifications	501, 518, 519, 521
		Test	502 to 517, 520
6	Hardened Concrete	Test	601 to 707
7	Steel	Test	701 to 707

CHAPTER SIX

CONCRETE QUALITY

❒6.0 Notation

f_c = Characteristic compressive strength of concrete , MPa (N/mm^2) .

f_{cm} = Average compressive strength of concrete, MPa(N/mm^2) .

$f_{1,2,3}$ = Compressive strength of test samples number 1, 2, and 3 .

s = Standard deviation of the compressive strength of samples.

x_3 = Average compressive strength of 3 samples.

x_{min} = Smallest compressive strength of test samples.

❒6.1 General

6.1.1 Quality of concrete with regards to strength, durability and other exposure condition consideration shall conform to the provisions of this chapter.

6.1.2 In the laboratory, concrete shall be proportioned to produce an average compressive strength as prescribed in clause 6.4.1. Concrete shall be made to minimize the number of samples showing strengths below the required average compressive strength as specified in clause 6.4.4 .

6.1.3 Requirements of characteristic strength of concrete shall be based on tests of cylindrical samples with dimensions of 150 $\times$300 mm. Strength of cubic test samples shall be converted to equivalent strength of cylinders.

6.1.4 Preparation and testing of cylindrical samples shall conform to the following standards:

6.1.4.1 For sampling : “Method of sampling fresh concrete” (T.C.S.B. 502).

6.1.4.2 For making test samples: “Making and curing concrete test specimens in the field” (T.C.S.B. 504).

6.1.4.3 “Compressive strength of concrete (cylinders)test”(T.C.S.B. 602).

6.1.5 Requirements of characteristic strength of concrete shall be based on 28-day tests.

6.1.6 Concrete tensile strength tests shall not be used as the basis for field acceptance of concrete unless the tensile strength test is specified as the criteria for acceptance of concrete in the technical job specification of the project

6.1.7 The inspection system shall keep a complete record of the test results on the concrete used, till the end of the protection period and for at least one year after the completion date and shall then deliver it to the owner. Computerized storage of the information is mandatory for important buildings.

❐6.2 Selection of concrete proportions

6.2.1 proportion of constituent materials of concrete shall conform to the following requirements:

6.2.1.1 Adequate workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed, without segregation or excessive bleeding.

6.2.1.2 Resistance of concrete as regards to durability and special exposure conditions shall conform to clause 6.3 .

6.2.2 Mix proportions for the constituent materials of concrete shall be established on the basis of field experiences and on trial mixes made with materials to be used.

❒6.3 Durability of concrete

6.3.1 General

Durability of concrete made with Portland cement is determined by its resistance to weather elements, chemical attacks, abrasion, erosion and other deteriorations.
Durable concrete will maintain its initial form, quality and serviceability in the specified exposure conditions.

6.3.2 Factors influencing durability

6.3.2.1 Frequent freezing

Frequent freezing and thawing leads to the deterioration of concrete in cold regions. This is intensified by the use of de-icer chemicals. In these cases concrete shall be made with the use of air–entraining admixtures, good aggregates, low water per cement ratios and low permeability.

6.3.2.2 Corrosive chemical agents

Use of suitable cement, correct mix proportions and low permeability of concrete will add up to its high resistance to salts and deleterious materials present in soil and ground water.
Good quality concrete is resistant to mild acids, but resistance to strong corrosive acids requires special protective provisions.

6.3.2.3 Abrasion and erosion

In some cases, especially for industrial floors, concrete surfaces may suffer abrasion. In marine structures, sand and grit in the water current may cause surface abrasion. Use of fine quality concrete and in more severe cases, use of very hard aggregates may safeguard adequate durability to resist such actions.

6.3.2.4 Reactive aggregates

Certain aggregates chemically reactive with alkali materials present in portland cement can lead to the expansion and disintegration of concrete. Care in the selection of source of aggregates, use of cement of low alkalinity and pozzolanic materials in appropriate cases, can prevent such problems(Also see clause 3.4.5).

6.3.2.5 Corrosion of reinforcement

Corrosion of reinforcement may lead to the spelling of concrete surfaces. This phenomenon can cause difficulties , particularly in bridge decks in cold regions where de-icing chemicals may be used, and in concrete structures in hot and moist regions. Specifying a thick concrete cover, use of chemical and mineral admixtures, use of low permeability concrete and the application of other special protective measures can greatly increase durability.

6.3.3 Special requirements for increasing durability in different exposure conditions.

6.3.3.1 Use of air – entraining admixtures

Concrete that is likely to be exposed to frequent freezing and thawing or de-icer chemicals shall be made with air-entraining admixtures. Percentage of bubbles in fresh concrete shall be in accordance with T.C.S.B 510 and table 6.3.3.1. When characteristic concrete strength is greater than 35 MPa, the figures in the table may be reduced by 1 percent.

Table 6.3.3.1 Total quantity affair bubbles for concrete resistant to freezing and thawing

Maximum nominal size of aggregates (mm)	Percentage of air content at exposure conditions*	
	Severe exposure conditions +	Moderate exposure conditions ++
9.5	7.5	6
12.5	7	5.5
19.0	6	5
25	6	4.5
38	5.5	4.5

* Air content tolerance is $\pm$ 1.5 percent at the location of use
+ Severe exposure conditions: before freezing, concrete is constantly exposed to humidity or de–icing chemicals e.g. concrete pavements, bridge decks, sidewalks, and water tanks.
++ Moderate exposure conditions: before freezing in cold weather, concrete is only sometimes exposed to humidity, or is not exposed to de–icing chemicals, like some beams or external walls and also slabs not in direct contact with soil.

6.3.3.2 Water per cement ratio limitation

Concrete exposed to the exposure conditions stated in table 6.3.3.2 shall provide for requirements of maximum water per cement ratio and minimum characteristic strength.

Table 6.3.3.2: Requirements of special exposure conditions

Exposure conditions	Maximum water per cement ratio	minimum characteristic strength (MPa)
Waterproof concrete: a) Concrete exposed to fresh water b) Concrete exposed to brackish water or sea water	0.50 0.45	25
Concrete exposed to freezing and thawing in moist conditions, frequent wetting and drying or de–icing chemicals	0.45	30
For protection against corrosion in reinforced concrete structures exposed to chlorides or de–icing chemicals, salt, brackish water, sea water or leaching.	0.4	35

6.3.3.3 Precautionary provisions for sulphate-containing solutions

Concrete to be exposed to sulphate–containing solutions shall conform to requirements of table 6.3.3.3. Classification of sulfates contained in soil in different exposure conditions and also precautionary measures recommended for different kinds of concrete members are presented in the tables.

These types of concrete shall have suitable strength and low permeability and possibly without hazardous materials for meeting these needs:

-Use portland cement or properly blended portland cement such as slag cement, portland cements blended with natural or artificial pozzolans.

-Water per cement ratio shall be reduced using suitable admixtures such as plasticizers or super plasticizers.

-By using active fine grained siliceous materials like silica fume (T.C.S.B. 409), and fly ash (T.C.S.B. 403), the hydroxide from hydration of cement will transform to calcium silicate as much as possible.
-In areas where chlorides are present in addition to sulfates, more care shall be taken in choosing the type of cement for reinforced concrete elements and members. Especially, usage of type 5 portland cement alone, which has little protection against penetration of chlorine ions into concrete and which prevents corrosion of the reinforcement bars shall be avoided.

Table 6.3.3.3 (a)-Classification of sulphates in soils and recommended precautionary provisions for precast concrete elements

Recommended precautionary provisions	**Classification of sulphates in different exposure conditions**			
Precast concrete elements (Piles, cylindrical elements)	**So_3 in soil** ** in extract 2 to 1(g/l)	**So_3 in soil** Total content %	**So_3 in Ground water (PPm)**	**Exposure conditions[1]**
No special precautionary provisions necessary	-	<0.2	<300	Mild
a) When the whole structure is above ground water table, type 1 cement shall be used (Minimum cement content shall not be less than 310 Kg/m^3 and water-cement ratio shall not exceed 0.5)	-	0.2 To 0.5	300 To 1200	Moderate
b) when the whole structure is in contact with a variable water table [+], type 1 cement shall be used (Minimum cement content shall not be less than 330 Kg/m^3, and water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (minimum cement content shall not be less than 240 Kg/m^3. and water-cement ratio shall not exceed 0.5)				
a) when the whole structure is above water table, type 1 cement shall be used (minimum cement content shall not be less than 330 Kg/m^3. and water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (cement content shall not be less than 290 Kg/m^3, and the water-cement ratio shall not exceed 0.5)	1.9 To 3.1	0.5 To 1.0	1200 To 2500	*** Severe
b) when the structure is in contact with a variable water table, type 5 cement shall be used (minimum cement content shall not be less than 330 Kg/m^3 and the water-cement ratio shall not exceed 0.5)				
a) When the whole structure is above water table. Type 5 cement shall be used (Minimum cement content shall not be less than 390 Kg/m^3. and the water-cement ratio shall not exceed 0.45)	3.1 To 5.6	1 To 2	2500 To 5000	Very severe
b) In case of contact with variable ground water table, steel or plastic overlay shall be used, and concrete shall be made with type 5 cement (minimum cement content shall not be less than 370 Kg/m^3 and the water-cement ratio shall or exceed 0.4)				
a) Above ground water table, type 5 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 and the water cement ratio shall not exceed 0.4)	>5.6	>2	>5000	Extremely severe
b) If in contact with variable ground water table, steel or plastic overlay shall be used, and concrete shall be made with type 5 cement (minimum cement content shall not be less than 370 Kg/m^3 and the water-cement ratio shall not exceed 0.4)				

[1] For a definition of environmental conditios refer to 8-2-9-2

* The cement contents recommended in this table are appropriate for low workability concrete (10mm-25mm slump)

** The expression “2 to 1 extract” means the weight ratio of soil to water is 2.

*** Portland blast-fumace slag cement conforming to T.C.S.B 102 specification, or pozzolanic Portland cements with less than 25% pozzolan content, may be substituted for type 5 cement provided that the So_3 content does not exceed 1200 parts per million in water (or 0.5 % in soil), pozzolanic portland cements containing more than 25% pozzolan can be considered as a substitute for type 5 cement when the So_3 dos not exceed 2500 parts per million in water (or 1% in soil)

+ If the structure is exposed to penetrating water, it shall be considered similar to structures exposed to variable water tables.

Table 6.3.3.3 (b)-Classification of sulphates in soils and recommended precautionary provisions for large concrete foundations

Recommended precautionary provisions	Classification of sulphates in different exposure conditions			
Large concrete foundations (Includes pile caps)	**So_3 in soil**		**So_3 in Ground water (PPm)**	**Exposure conditions**
	**** in extract 2 to 1(g/l)**	**Total content %**		
a) Where the whole foundation is above ground water table, no special precautionary provisions are necessary.	-	<0.2	<300	Mild
b) Where the foundation is in contact with a variable water table, type 1 cement shall be used (minimum cement content shall not be less than 310 Kg/m^3 and the water-cement ratio shall not exceed 0.55)				
a) Where the whole foundation is above ground water table, type 1 cement shall be used (Minimum cement content shall not be less than 330 Kg/m^3 and the water-cement ratio shall not exceed 0.5)	-	0.2 To 0.5	300 To 1200	Moderate
b) If the foundation is in contact with a variable ground water table type 1 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3, and the water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (minimum cement content shall not be less than 310 Kg/m^3 and water-cement ratio shall not exceed 0.5)				
a) If the whole foundation is above ground water table, type 1 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3 and water-cement ratio shall not exceed 0.5), or use type 5 cement (minimum cement content shall not be less than 340 Kg/m^3, and the water-cement ratio shall not exceed 0.5)	1.9 To 3.1	0.5 To 1.0	1200 To 2500	*** Severe
b) If the foundation is in contact with a variable ground water table, type 5 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3 and the water-cement ratio shall not exceed 0.5)				

Table 6.3.3.3 (b)-continued

a) If the foundation is above water table. Type 1 cement shall be used (minimum cement content shall not be less than 400 Kg/m^3 , and the water-cement ratio shall not exceed 0.45) or type 5 (minimum cement content shall not be less than 350 Kg/m^3 and the water-cement ratio shall not exceed 0.45)	3.1 To 5.6	1 To 2	2500 To 5000	Very severe
b) If the foundation is in contact with a variable ground water table, determination of cautions is necessary to enable decision on use of type 5 cement [+] , or supersulphate cement [++] or overlay.				
a) If the whole foundation is above ground water table, type 1 cement shall be used (minimum cement content shall not be less than 400 Kg/m^3 and the water-cement ratio shall not exceed 0.4)	>5.6	>2.0	>5000	Extremely severe
b) If the foundation is in contact with ground water table, type 5 cement shall be used (minimum cement content shall not be less than 390 Kg/m^3 and the water-cement ratio shall not exceed 0.4). Adequate protection shall be provided with the use of asphalt or bituminous felt or plastic overlay.				

*, **, *** Notes are similar to those in table 6.3.3.3.(a)

+ In acidic soil conditions, type 5 cement (sulphate resistant) can only be used when the pH is greater than 6. in such cases the use of admixtures is not permitted.

++ In acidic soil conditions, super sulphate cement can only be used when the pH is greater than 3.5. in such cases the use of admixtures is not permitted.

Table 6.3.3.3 (c)-Classification of sulphates in soils and recommended precautionary provisions for thin concrete sections

Recommended precautionary provisions	Classification of sulphates in different exposure conditions			
Thin concrete elements in basements, pipes, culverts and access channels	So_3 in soil		So_3 in Ground water (PPm)	Exposure conditions
	** in extract 2 to 1(g/l)	Total content %		
a) Where the whole structure is above ground water level type 1 cement shall be used (minimum cement content shall not be less than 310 Kg/m^3 , and water-cement ratio shall not exceed 0.55)	-	<0.2	<300	Mild
b) Where the structure is exposed to water pressure from outside, type 1 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3 and the water-cement ratio shall not exceed 0.55), or membrane protection such as asphalt or bituminous felt shall be employed				
a) Where the whole structure is above ground water table, type 1 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not be greater than 0.5)	-	0.2 To 0.5	300 To 1200	Moderate
b) Where the structure is exposed to water pressure from outside, type 1 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 , and water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5) or membrane protection such as asphalt or bituminous felt shall be used.				
a) Where the whole structure is above water table, type 1 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 and the water-cement ratio shall not exceed 0.5)	1.9 To 3.1	0.5 To 1.0	1200 To 2500	*** Severe
b) Where the structure is exposed to water pressure from outside, type 5 cement shall be used (minimum cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5) or use other methods such as asphalt or bituminous felt or other impervious membranes.				
a) Where the whole structure is above water table and the soil always remains dry type 2 cement shall be used (minimum cement content shall not be less than 400 Kg/m^3 , and the water-cement ratio shall not exceed 0.45) or type 5 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3 and water-cement ratio shall not exceed 0.45)	3.1 To 5.6	1 To 2	2500 To 5000	Very severe
b) Where the structure is in contact with a varying ground water table, determination of cautions is necessary to enable decision on the use of type 5 cement [+] , or supersulphate cement [++] or asphalt, bituminous felt or other impervious membranes.				

Table 6.3.3.3 (c)- continued

a) Where the whole structure is above ground water table and the soil always remains dry, type 1 cement shall be used (minimum cement content shall not be less than 400 Kg/m^3 and water-cement ratio shall not exceed 0.45). or type 5 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3 and water-cement ratio shall not exceed 0.45) b) Where the structure is in contact with a variable ground water table, type 5 cement shall be used (minimum cement content shall not be less than 390 Kg/m^3 and water-cement ratio shall not exceed 0.4) or protection shall be provided by the use of asphalt, bituminous felt or adhesive plastic lovers.	>5.6	>2.0	>5000	Extremely severe

* The recommended cement contents ion table 6.3.3.3 (c) are suitable for concretes with moderate workability (50mm to 75mm slump).

, *, +, ++ Notes are similar to those in table 6.3.3.3 (b)

Table 6.3.3.3 (d)-Classification of sulphates in soils and recommended precautionary provisions for cast-in situ concrete piles

*Recommended precautionary provisions	Classification of sulphates in different exposure conditions			
In situ concrete piles	So_3 in soil		So_3 in Ground water (PPm)	Exposure conditions
	** in extract 2 to 1(g/l)	Total content %		
a) Where piles are totally above water table, type 1 cement shall be used (cement content shall not be less than 330 Kg/m^3 , and water-cement ratio shall not exceed 0.55)	-	<0.2	<300	Mild
b) Where piles are in contact with a variable water table, type 1 cement shall be used (cement content shall not be less than 340 Kg/m^3 and water-cement ratio shall not exceed 0.55)				
a) Where piles are totally above ground water table, type 1 cement shall be used (cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5)	-	0.2 To 0.5	1200 To 1300	Moderate
b) Where piles are in contact with a variable water table, type 1 cement shall be used (cement content shall not be less than 370 Kg/m^3 , and water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5)				
a) Where pile are above water table, type 1 cement shall be used (cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5), or type 5 cement shall be used (cement content shall not be less than 370 Kg/m^3 and water-cement ratio shall not exceed 0.5)	1.9 To 3.1	0.5 To 1.0	1200 To 2500	*** Severe
b) Where piles are in contact with ground water, type 5 cement shall be used (cement content shall not be less than 370 Kg/m^3 , and water-cement ratio shall not exceed 0.5), only applicable to end-bearing piles[+] .				

Table 6.3.3.3 (d)- continued

a) Where piles are totally above ground water table and the soil is always free from seeping water, type 1 cement shall be used (cement content shall not be less than 400 Kg/m^3 , and water-cement ratio shall not exceed 0.45), or type 5 cement shall be used (cement content shall not be less than 350 Kg/m^3 , and water-cement ratio shall not exceed 0.45) b) Sulphates are considered very aggressive when present in ground water at more than 3000 parts per million. Concentrations special provisions shall be required such as using super sulphate cement or protection of end-bearing piles with membrane covers. Type of cement to be used depends on the cations.	3.1 To 5.6	1 To 2	2500 To 5000	Very severe
a) Where piles are totally above water table, and the soil is free from seeping water, type 1 cement shall be used (minimum cement content shall not be less than 400 Kg/m^3 , and water-cement ratio shall not exceed 0.45) or type 5 cement shall be used (minimum cement content shall not be less than 350 Kg/m^3 and water-cement ratio shall not exceed 0.45) b) Sulphates are considered to be very aggressive when present in ground water at concentrations greater than 3000 parts per million and special precautionary provisions shall e necessary such as using supersulphate cement or production of end-bearing piles with membrane covers. Type of cement to be used depends on the cautions.	>5.6	>2.0	>5000	Extremely severe

* The recommended cement contents in table 6.3.3.3 (d) are suitable for concretes with relatively high workability (slump about 100mm)

, * See notes for table 6.3.3.3 (a)

+ Sulphate attack might lead to the creation of a thin shell on the surface of the pile and hence cause a reduction in shaft friction. Therefore, the precautionary measures referred to in this code are only applicable to endbearing piles.

6.3.3.4 Corrosive exposure conditions

When concrete is likely to be exposed to seawater, brackish water or de – icing salts, in addition to the requirements of tables 6.3.3.3 for water per cement ratio, the requirements of subclause 8.2.9.1 for minimum concrete cover shall also be complied with.

6.3.3.5 Sulphates in concrete

The total concentration of soluble sulphates in concrete mixing water shall not exceed 4 percent and the total concentration of sulphates present shall not exceed 5 precent of the cement content in the mix. The total sulphate content in concrete shall be determined on the basis of summation of all sulphate contents present in the constituent materials of concrete. Also see tables 3.4.5.2, 3.4.5.3 and 3.5.1 .

6.3.3.6 Chlorides in concrete

For corrosion protection, maximum water soluble chloride ion concentrations in hardened concrete at the age of 28 days contributed from the constituent material including water, aggregates, cementitious materials and admixtures, shall not exceed the limits of table 6.3.3.6 .

Table 6.3.3.6: Maximum chloride ion content for corrosion protection.

Type of member	Maximum water soluble chloride ion (Cl^{-}) in concrete. Percent by weight of cement
- prestressed concrete	0.06
- Reinforced concrete exposed to chlorides and moisture in service	0.15
- Reinforced concrete that will be dry of protected from moisture in service	1.00
- Other reinforced concrete construction	0.30

❒6.4 Proportioning on the basis of field observation and/or train mixtures.

6.4.1 Characteristic strength of concrete

The characteristic strength of concrete is the strength below which no more than 5 percent of all measured strengths for the specified concrete grade are expected to fall. In practice, the concrete will be considered to be to the required class and acceptable when it is in compliance with the requirements of clause 6.5.

6.4.2 Classification of concrete

Classification of concrete based on characteristic strength is as follows.

C6 C8 C10 C12 C16 C20 C25 C30 C35 C40 C45 C50

The figures after C represent the characteristic compressive strength of concrete in MPa.

Concrete Grades C16 and above are used in reinforced concrete construction and use of C12 concrete shall only be allowed subject to adequate justification and in full consideration of the necessary conditions.

For concrete grades above C50, special provisions other than instructions mentioned in this chapter shall be observed.

6.4.2.1 Methods of selection of concrete proportions

a) Concrete proportions for concrete up to grade C12 shall be established on the basis of past experience without laboratory studies.
b) For concretes up to grade C25 , concrete proportions may be based on the "standard mix proportions" in accordance with the general technical specification document provided that standard materials are used.
c) For concrete grades C30 and above, the optimum mix proportions shall be based on laboratory studies. These studies may be carried out by the designer prior to the commencement of construction works, and the results may be entered in the specification as the " Required mixing proportions" or the studies may be carried out by the contractor and the results may be used as " The specified mixing proportions".

6.4.3 Standard deviation

6.4.3.1 When a concrete production facility has test records, a standard deviation shall be established. Test records from which a standard deviation is calculated shall conform to the following conditions:

a) Shall represent materials, quality control procedures, and conditions similar to those expected, and changes in materials and proportions within the test records shall not have been more restricted than those for the proposed work.
b) Shall represent concrete produced to meet the required average compressive strength in accordance with clause 6.4.4 .
c) Shall consist of at least 30 consecutive samples or two groups of consecutive samples totaling at least 30 tests except as provided in clause 6.4.3.2 “Consecutive samples” are defined as samples where the time span between the samplings does not exceed 3 calendar days.

6.4.3.2 When a concrete production facility does not have test records meeting the requirements of clause 6.4.3.1, but does have a record based on 15 to 24consecutive tests, a standard may be established as the product of the calculated standard deviation and the relevant modification factor of table 6.4.3.2 to be acceptable, test records must meet requirement a) and b) of the subclause 6.4.3.1, and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

Table 6.4.3.2. Modification factor for standard deviation when less than 30 tests are available.

NO. OF TESTS *	MODIFICATION FACTOR FOR STANDARD DEVIATION
15	1.16
20	1.08
25	1.03
30 or more	1.00

* Interpolate for intermediate numbers of tests.

6.4.4 Required average strength

6.4.4.1 Required average compressive strength used as the basis for selection of concrete proportions shall be the larger of the equations using a standard deviation calculated in accordance with clause 6.4.3:

$$f_{cm} = f_c + 1.34S + 1.5 \quad \text{(MPa)} \tag{6-1}$$

$$f_{cm} = f_c + 2.33S - 4.0 \quad \text{(MPa)} \tag{6-2}$$

6.4.4.2 When a concrete production facility does not have test record for calculation of standard deviation meeting the requirements of subclause 6.4.3.1 or 6.4.3.2, the required average strength shall be determined from table 6.4.4.2 and documentation of average strength shall be in accordance with reqirements of clause 6.4.5.

Table 6.4.4.2- Required average compressive strength when data are not available to establish a standard deviation.

Concrete grade	REQUIRED AVERAGE COMPRESSIVE STRENGTH (MPa)
C12 and below	$f_{cm} = f_c + 6.0$
C16	$f_{cm} = f_c + 7.5$
C20	$f_{cm} = f_c + 8.5$
C25	$f_{cm} = f_c + 4.5$
C30 and C35	$f_{cm} = f_c + 10.5$
C40 and above	$f_{cm} = f_c + 11.0$

6.4.5 Documentation of average compressive strength

Documentation that the proposed concrete proportions will produce a required average compressive strength may consist of a field strength test record, several strength test records , or trial mixtures.

6.4.5.1 When test records are used to demonstrate that the proposed concrete proportions will produce the required average compressive strength, such records shall present materials and condition similar to those expected. Changes in materials, conditions and proportions within the test records shall not have been more restricted than those for the proposed work. For the purpose of documenting average strength potential, test records consisting of 10 consecutive tests or 30 dispersed tests may be used, provided test records encompass a period of time not less than 45 days. Required concrete proportions may be established by interpolation between the strength and proportions of two or more test records each of which meets other requirements of this clause.

6.4.5.2 When an acceptable record of field test results in not available, concrete proportions may be established based on trial mixtures meeting the following restrictions:

a) Mix proportions shall be the same as those actually used in project
b) Trial mixtures having proportions and consistencies required for the proposed work shall be made using at least 3 different water-cement ratios or cement contents that will produce a range of strengths encompassing the or cement contents that will produce a range of strengths encompassing the required average compressive strength.
c) Trial mixtures shall be designed to produce a slump within ± 20 mm of maximum permitted, and for air-entrained concrete, within ± 0.5 percent of maximum allowable air content.
d) For each water-cement ratio or cement content, at least three test samples for each test age shall be made and cured in accordance with " Method of making and curing concrete test specimens in the laboratory" (T.C.S.B. 503). Samples shall be tested at 28 days or at test age designated for determination of characteristic compressive strength of concrete.

e) From results of tests, a curve shall be plotted showing the relationship between water-cement ratio or cement content and compressive strength at designated test age.
f) Maximum water-cement ratio or minimum cement content for concrete to be used in the proposed word shall be that shown by the curve to produce the average strength required by clause 6.4.4 unless a lower water-cement ratio or higher cement content is required by clause 6.3 .

6.4.6 Lowering or increasing the average compressive strength
6.4.6.1 As data become available during construction, the required compressive strength may be reduced provided:

a) 30 or more results are available and average test results exceeds that required by clause 6.4.4.
b) Special exposure requirements of clause 6.3 are met.

6.4.6.2 If 30 or more test results are available and average test results falls below that required by clause 6.4.4, steps shall be taken to increase the average strength results of subsequent tests.

❒6.5 Evaluation and acceptance of concrete

6.5.1 Acceptance of concrete, frequency of sampling and strength tests
Field acceptance of concrete shall be based on compressive strength test results of samples taken from the concrete to be placed. Frequency of sampling shall be evenly spread over period covering the making and placing of concrete. Samples shall be taken from where the concrete is to be poured.

6.5.1.1 The purpose of sampling is the making of two test specimens on which compressive strength tests are made 28 days after making or at any other designated age, and the average compressive strength results shall be considered as the final test result. For assessment of concrete quality earlier that the designed age, another test specimen may be made for the purpose of compressive strength test.

6.5.1.2 When the volume of each class of concrete is greater than $1m^3$, frequency of sampling shall be as follows:

a) For walls and slabs, one sample for each $30m^3$ of concrete or per $150m^2$ of surface area.
b) For beams and ties, if poured separately from other members, one sample for each 100m length.
c) For columns, one sample for each 50m length.

6.5.1.3 When the volume of each class of concrete is less than 1 m^3, frequency of sampling prescribed in sub clause6.5.1.2 may be reduced proportionally.

6.5.1.4 At least one sample shall be taken from each class of concrete placed each day.

6.5.1.5 At least six samples shall be taken for the whole structure.

6.5.1.6 When the total quantity of any given class of concrete is less than 20 m^3, strength tests maybe waived by the inspecting engineer, if in his judgment evidence of satisfactory strength is provided.

6.5.2 Requirements for acceptance of concrete for laboratory cured specimens

6.5.2.1 Strength level of an individual class of concrete shall be considered satisfactory when one of the following requirements is met.

a) In compressive strength testing of three consecutive samples, no strength test result falls below the characteristic compressive strength.

$$X_{1,2,3} \geqslant f_c \tag{6-3}$$

b) Average strength of samples shall be at least 1.5MPa greater than the characteristic strength, and the minimum strength of the samples shall not be less than the characteristic strength minus 4 MPa.

$$\bar{X}_3 \geqslant f_c + 1.5 \tag{6-4}$$

$$X \geqslant f_c - 4.0 \tag{6-5}$$

6.5.2.2 Concrete will not be considered acceptable when average strength of samples falls below characteristic strength or when the minimum sample strength falls below the characteristic strength minus 4

$$X_{\min} < f_c - 4.0 \quad \text{or} \quad X_3 < f_c \tag{6-6}$$

6.5.2.3 Concrete which is acceptable according to subclause 6.5.2.2 but is not acceptable under subclause 6.5.2.1 (b), may be considered structurally acceptable by the designer without further assessment.

When concrete is not acceptable under clause 6.5.2.2 , steps shall be taken in accordance with clause 6.6 .

6.5.2.4 In checking whether concrete is up to the required class, no test results shall be ignored unless evidence can be provided to prove that a major fault has occurred in casting, maintenance, transportation, curing or testing.

6.5.3 Requirements for controlling the method of curing and protecting concrete.

6.5.3.1 The inspecting engineer may require strength tests of specimens cured under field conditions to check adequacy of curing and protection of concrete in the structure.

6.5.3.2 Field-cured specimens shall be cured under field conditions in accordance with " Method of making and curing concrete test specimens in the field" (T.C.S.B. 504)

6.5.3.3 Field-cured test samples shall be molded at the same time and from the same samples as laboratory cured test specimens.

6.5.3.4 Procedures for protecting and curing concrete shall be considered acceptable when the compressive strength of field-cured samples tested at the designated age for characteristic strength , is at least 85% of that of companion laboratory-cured specimens, or when it is greater than characteristic strength by 4 MPa. Otherwise, steps shall be taken to improve procedures.

6.5.4 Additional tests on concrete for special purposes

Additional samples may be required for various purposes such as the time at which to remove formwork. These samples are additional to test specimens taken for assessing strength, curing and protecting concrete (clauses 6.5.1 and 6.5.3). These samples are referred to as " Special purpose samples".

❐6.6 Investigation of low-strength test results

If any strength test of laboratory-cured specimens is rendered unacceptable and not up to the required class of concrete by clause 6.5.2, the following steps shall be taken to assure that the load-carrying capacity of the structure is not jeoperdized:

6.6.1 If by using the same structural analysis utilized for the structure and an reappraisal of design, it can be proved that the load-carrying capacity of the structure, with a lower strength than that previously anticipated, is acceptable, the class of concrete as regards to structural strength may be declared acceptable.

6.6.2 If the requirements of clause 6.6.1 is not met, but with renewed analysis and design it can be proved that the load-carrying capacity of the whole structure on the assumption of lower strength concrete in probable parts, is adequate, the class of concrete as regards to strength of the structure may be considered acceptable.

6.6.3 If the requirements of clauses 6.6.1 and 6.6.2 are not met, cores drilled from the area in question shall be tested in accordance with " Method of obtaining and testing drilled cores and sawed beams of concrete" (T.C.S.B625) for parts of the structure where test results do not meet the acceptance requirements of clause 6.5.2 . Three cores shall be taken for strength test.

6.6.4 If concrete in the structure will be dry under service condition, cores shall first be air-dried for 7 days at temperatures between 16^0C to 27^0C and relative humidity of less than 60% , and shall be tested dry. If concrete in the structure will be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hours and be tested wet.

6.6.5 Concrete in an area represented by core tests shall be considered structurally adequate if the average of 3 cores is equal to at least 80 percent of characteristic strength and if no single core is less than 75 percent of characteristic strength. To check testing accuracy, locations represented by erratic core strengths may be retested.

6.6.6 If criteria of clause 6.6.5 are not met, and if structural adequacy remains in doubt, load tests as outlined in clause 19.1.3 for the questionable portion of the structure must be done or other appropriate action shall be taken.

❐6.7 Control and inspection

To ensure conformance of quality and properties of concrete with the provisions and standards of the code, the least frequent control and inspection shall be made according to the table 6.7 .

Table 6.7: Control and inspection of concrete characteristics

No	Type of test	Type of inspection	Objective	Repetition time
1	Proportioning for mix design	Test at the beginning	Providing reasons that designed specifications are in safety margin	Before using each new mixture if data from long term experiences is not available
2	Chloride content in mixture	Calculation based on existing chloride in constituent materials of concrete	To assure that chloride content dose not exceed allowable value	At the beginning and in the cases when chloride content changes
3	Moisture content in coarse aggregates	Dry test or its equivalent	Modification of required water	None of required tests may be reduced or increased in the case of different weather conditions, and if it is not constantly done on a daily basis
4	Moisture content in fine aggregates	Measuring constantly, Dry test or its equivalent	Modification of required content	Same as above
5	Consistency of concrete	Observation inspection	To compare to the required appearance condition of concrete	Every batching
6		Consistency test	evaluation of conformity for consistency level required to control probable changes of water content	1- when sampling for hardened concrete 2- when testing for air content of concrete 3- when there is doubt based on observation
7	Unit weight of fresh concrete	Unit weight test	Inspection of batching and control unit weight of light or heavy concrete	Same number of times as compressive strength tests
8	Compressive strength test of prepared sample	According to standard test	Evaluation of compressive strength characteristic	According to clause 6-5 of the code

Table 6.7- continued

9	Appearance unit weight of light or heavy weight concrete	According to standard test	Evaluation of unit weight	Same number of times as compressive strength tests
10	The amount of water added to mixture	Keep record of the amount of added water	Test for real water per cement ratio	For each batching
11	The amount of cement in fresh concrete	Write down the amount of cement used	To control the amount of cement and real water per cement ratio	For each batching
12	The amount of admixture in fresh concrete	Write down the amount of admixture added	To control the amount of admixture used	For each batching
13	Water per cement ratio of fresh concrete	To divide the summation of rows No:3,4 and 9 by row No:10 or any accepted standard method.	Evaluation of the water per cement ratio	Daily or more frequently if it is necessary
14	Air content in fresh concrete for air entrained concrete	According to standard test	Evaluation of conformity of air content with specified one	For mixture with air bubble 1- first batch and at least once a day 2- Many times according to production and exposure conditions
15	Homogenity	Test by comparison of specifications of the samples taken from different parts of mixture	Evaluation of homogeneity of the mixture	In the case of doubt
16	Permeability	According to standard test	Evaluation of strength to permeability	At the beginning and based on agreement for more times
17	Other specifications	According to revelant codes or based on agreement	Evaluation comformity with required specifications	Based on agreement

CHAPTER SEVEN

MIXING AND PLACING CONCRETE

❒7.0 Notation

T= Average temperature on concrete surface, degree celcius (centigrade)

❒7.1 Manpower, equipment and preparation of the concreting location

7.1.1 Manpower

Preparation, application and control of concrete operations shall be done by well trained workers who have sufficient experience and knowledge in this field.

7.1.2 Equipment and devices

a) All equipment used for mixing and transporting shall be clean.

b) Batching of constituent materials required of concrete shall be done by weighing as much as possible.

c) Weight tolerance of each of the constituent materials of concrete is ± 3 percent.

d) Accuracy and sensitivity tolerance for balances and other weighing devices shall be ± 0.4 percent of the total capacity of the machine.

e) Use of other methods for batching materials is permitted, provided that accuracy of the amount of materials obtained by this method is comparable with that of the weighing method.

f) For uniform distribution of chemical admixtures in concrete volume, while using appropriate equipment, required attention shall be rendered and the instructions recommended by the factory shall be followed.

7.1.3 Preparation of the concreting location

a) All extra stuff such as ice shall be removed from spaces to be used for concreting.
b) Forms shall be properly cleaned and coated.
c) Masonry materials that will be in contact with concrete shall be well drenched.
d) All reinforcement bars shall be completely cleaned free from polluting coats before depositing.
e) Extra water shall be removed from the place before concrete is placed unless special under-water concreting pipe and hopper (tremie) is used, or it is authorized by the inspector engineer.
f) All probable laitance layers and unsound materials shall be removed before additional concrete is placed against hardened concrete surface.

❐7.2 Concrete mixing

7.2.1 Concrete shall be conveyed from mixer to place of final deposit by methods that will be capable of providing a supply of concrete at site of placement without separation of ingredients and without interruptions sufficient to permit loss of plasticity between successive increments.

7.2.2 When conveying concrete by pumping, the ratio of maximum size of aggregate to minimum internal diameter of the pipe shall not exceed the following.

7.2.3 Site-mixed concrete shall be in accordance with the following provisions:

7.2.3.1 Mixing shall be done in a batch mixer approved by the inspector engineer.

7.2.3.2 Mixer shall be rotated at a speed recommended by the manufacturer.

7.2.3.3 Entering order of concrete materials into the mixer shall be in accordance with the type of mixer and concrete.

7.2.3.4 Mixing shall continue for at least 1½ minutes after all materials are in the drum, unless a shorter time is shown to be satisfactory by the tests carried out on the basis of "specifications for ready-mixed concrete" (T.C.S.B 501).

7.2.3.5 Materials handling, batching, and mixing shall conform to standard provisions on "Specifications for ready–mixed concrete" (T.C.S.B 501) or the "Specifications of the concrete made by volume batching and bond mixing" (T.C.S.B. 517)

7.2.3.6 A detailed daily record for all concrete mixes including concrete specifications and the following items shall be kept:

a) Proportions of materials used.
b) The results of tests on fresh concrete.
c) The concrete and the environment temperature when placing.
d) Final location and approximate volume of the concrete placed.
e) Time and date of mixing and placing.

7.2.4 Concrete remixing with water at the end of mixing, while transporting, or at the concreting location is not permitted unless for exceptional cases and with permission of the inspector engineer.

❒7.3 Conveying

7.3.1 Concrete shall be poured as nearly as practicable in its final position to avoid segregation due to rehandling or flowing.

7.3.2 Retempered concrete or concrete that has been remixed after initial set shall not be deposited in the structure.

7.3.3 When conveying concrete by pumping, the ratio of maximum size of aggregates to minimum internal diameter of the pipe shall not exceed the followings:

a) 0.33 for angular aggregates
b) 0.4 for round aggregates

❒7.4 Concrete placing

7.4.1 Concrete shall be poured as nearly as practicable in its final position to avoid segregation due to rehandling or flowing.

7.4.2 Concreting shall be carried out at such a rate that concrete maintains its plastic state during pouring and placing and flows readily into spaces between reinforcement.

7.4.3 If the concrete slump while delivered for use is less than the specified value, the concrete is not allowed for use. Nevertheless, it is allowed with permission of inspector engineer to increase the slump of concrete before removal from mixer, by adding cement grout with or without plasticizer, provided the water per cement ratio does not exceed its maximum permissible value.

7.4.4 Retempered concrete or concrete that has been remixed after initial set shall not be deposited in the structure.

7.4.5 After concreting is started, it shall be continued as a continuous operation until concreting of a panel or section, as defined by its boundaries or predetermined joints, is completed. Predetermined joints shall be made in accordance with clause 9.8.

7.4.6 Concrete deposited in horizontal layers shall have a level surface.

7.4.7 When construction joints are required, joints shall be made in accordance with clause 9.8.

7.4.8 All concrete shall be thoroughly consolidated by suitable means during placement and shall be thoroughly worded around reinforcement and embedded features and into comers of forms.

7.4.9 The vibrator shall be inserted in concrete systematically and in specific intervals so that the two adjacent shaken parts overlap each other. Part of vibrator shall be inserted in the lower layer that is still plastic.

7.4.10 Vibrator shall be inserted as nearly as possible vertically and taken out slowly in order not to let the air bubbles remain inside the concrete.

❒7.5 Curing

7.5.1 General

Curing is the process of preventing moisture loss from the concrete whilst maintaining a satisfactory temperature regime. Curing enhances hardened

concrete properties such as degree of impermeability and resistance to freezing and thawing. Curing of concrete shall commence immediately after compaction in order to protect it from damaging actions.
Curing of concrete is comprised of caring, protection and development in accordance with the following clauses:

7.5.1.1 Caring refers to measures taken to ensure that cement in concrete remains moist for a sufficient period to enable maximum hydration.

7.5.1.2 Protection refers to measures taken to protect the concrete from:
- leaching out by rain and flowing water.
- Rapid cooling or frost.
- Vibration.
- Impact

7.5.1.3 Development means accelerating strength gain by heat.

7.5.2 Curing methods

For preventing moisture loss from concrete whilst maintaining a satisfactory temperature regime, the following methods may be used.

7.5.2.1 Any method that keeps the mixing water in the concrete in the early hardening stages such as spraying with water or covering the surface with a damp absorbent material.

7.5.2.2 Any method that prevents moisture loss from the surface of the fresh concrete such as covering or plastering the surface with an impermeable material like nylon or water proof paper, or spraying the surface with an efficient curing membrane.

7.5.2.3 Any method that may accelerate strength gain of concrete by heat or moisture such as using heated forms or steam, provided it does not adversely affect the characteristics and durability of concrete.

Table 7.5.3 Minimum periods of curing and protection

<table>
<tr><th rowspan="2">Type of cement</th><th rowspan="2">Ambient conditions After placing*</th><th colspan="3">** Average surface temperature of concrete</th></tr>
<tr><th>5°C to 10°C</th><th>Above 10°C</th><th>T(Any temperature between 5°C to 25°C)</th></tr>
<tr><td rowspan="2">Type 1,2,3,5</td><td>Average</td><td>4 days</td><td>3 days</td><td>$\frac{60}{T+10}$ Days</td></tr>
<tr><td>Poor</td><td rowspan="2">6 days</td><td rowspan="2">4 days</td><td rowspan="2">$\frac{80}{T+10}$ Days</td></tr>
<tr><td rowspan="2">All cement except type 1,2,3,5and all cements with pozzolanic or blast-fumace slag</td><td>Average</td></tr>
<tr><td>Poor</td><td>10 days</td><td>7 days</td><td>$\frac{140}{T+10}$ Days</td></tr>
<tr><td>All cements</td><td>Good</td><td colspan="3">No special requirements</td></tr>
</table>

* Ambient conditions contained in this column are defined as follows:

Good, damp and protected (relative humidly greater than 80%, protected from the sun and the wind).

Poor: dry or unprotected (relative humility less than 50%, not protected from the sun and the wind).

Average: Intermediate between good and poor.

** when surface temperature of concrete is not measured. It may be taken to be the same as ambient temperature.

7.5.3 Minimum periods of curing and protection

Surfaces should normally be cured for a period not less than that specified in table 7.5.3. Curing period depends on the type of cement, the ambient conditions and the temperature of the concrete. During this period, no part of the surface temperature should fall below 5˚C.

7.5.4 Curing efficiency control

efficiency control of concrete curing shall comply with clause 6.5.3.

❐7.6. Concrete placing in special conditions

7.6.1 Hot weather concreting

7.6.1.1 Hot weather reduces the quality of freshly hardened concrete. Hot weather is referred to as high temperature with or without wind and low moisture. These factors will cause rapid water evaporation, increase in cement hydration speed, decrease in workability of fresh concrete and accelerate setting rate leading to a decrease in the final resistance of concrete. Hot weather can also cause some difficulties in concrete placement, compaction and enhancement of plastic shrinkage, which will result cracks in fresh concrete.

7.6.1.2 Maximum water absorption for the aggregates used in concrete is 2.5 percent for coarse aggregates and 3 percent for fine aggregates according to with T.C.S.B. 210 and 211.

7.6.1.3 Concrete temperature while paving shall not be more than 32 deg celcius (centigrade) for normal concrete and $15^{\circ}C$ for mass concrete. Hot weather concreting shall be done by providing proper conditions, adopting necessary measures, and approval of inspector engineer.

7.6.1.4 Temperature difference in various points of concrete, due to air temperature and heat of hydration shall be incorporated in the calculations, as it produces stress in the concrete.

7.6.1.5 Depending on the case, application of the following measures is required to reduce concrete temperature:

a) Appropriate and careful planning for the starting time in processes regarding making and placing concrete.
b) Fixing the time for concreting to be during cool air.
c) Application of proper and low-heat-inducing cements, or replacing part of the cement with pozzolanic materials, or use of pozzolanic or slag portland cements and utilization of appropriate mix design in order to avoid excess usage of cement.
d) Non-utilization of cement with a higher temperature than 75 degree celcius (centigrade).
e) Keeping cement temperature low by storing it in insulated or white painted silos.
f) Lowering aggregate temperature by storing them in the shade or spraying them with water and simultaneously blowing cold air.
g) Cooling service water and/or replacing part of it with crushed or flaky ice.
h) Insulation of water tanks and distribution pipes and/or white painting those parts exposed directly to sunshine.
i) Keeping in shade or water spraying tools and machinery for production and transportation of concrete.
j) Insulation of mixers or cold water spraying them or blowing cold air at them or painting them in white.

7.6.1.6 Bars, embedded elements, and moulds with higher temperature than 50 deg celcius (centigrade) shall be sprayed with water immediately before concreting, and the additional water shall be completely removed.

7.6.1.7 The following provision shall be applied to avoid moisture reduction and to increase concrete temperature after concreting, so as to prevent cracking:

- protection of concrete against wind and sunshine by providing windbreaker and shade.
- Prevention of water evaporation from concrete by spraying water on it and on the surrounding
- For structures where cracking is entirely unacceptable, special precautionary measures shall be taken

7.6.1.8 While water spraying is preferred, concrete curing in compliance with 7-5 is mandatory. Membrane curing compounds approved by inspection system may be used for horizontal surfaces

7.6.1.9 In addition to provision of time conditions of the table 7.5.3, curing time of concrete shall not be less than seven days.

7.6.2 Concreting in offshore areas of Persian gulf and the sea Oman. In coastal regions of the Persian gulf and the sea of Oman, while observing requirements of hot weather concreting (article 7.6.1) the following issues shall also be incorporated.

7.6.2.1 Appropriate materials are selected as described above and mix proportions are determined so as to avoid excess cement usage and reduce water per cement ratio and permeability.

7.6.2.2 Proper, low-heat-inducing cement, type II portland cement, type I pozzolanic cement, or blended pozzolanic or slag portland cements, or other pozzolanic cements shall be used. The amount of pozzolan depends on its type and on environmental location of the structure.

7.6.2.3 Minimum cement content is 350 Kg/m^3 of concrete and maximum cement content is 450 Kg/m^3 of concrete.

7.6.2.4 Minimum chloride content of service water shall be 500 parts per million (PPm). Other characteristics of service water shall comply with the table 3.5.1. Total water-soluble chloride content of the 28-day hardened concrete, contributed by all constituent components of the concrete shall not exceed the values given in the table 6.3.3.6.

7.6.2.5 Use of saline water, especially sea water for washing aggregates, production and curing concrete is not permitted.

7.6.2.6 Maximum ratio of water per cementitious materials (cement plus pazzolanic or slag materials) is 0.4 .

7.6.2.7 Aggregates used, especially fine aggregates shall be washed and cleaned so as to have compliance with the values of the tables 3.4.5.2 and 3.4.5.3 .

7.6.2.8 Maximum water absorption of the aggregates used in concrete is limited to 2.5 percent for coarse aggregates and 3 percent for fine aggregates, based on T.C.S.B. 210 and 211.

7.6.2.9 To reduce concrete permeability, fresh concrete mix shall have adequate compaction, and for this purpose strong water-reducing additives shall be used.

7.6.2.10 When chemical additives, pozzolans, and semicementitious materials are used, these substances shall comply with specifications of 3.6. Use of additives shall have approval of the inspection system.

7.6.2.11 Maintenance and storage of reinforcement shall be such that they are not contaminated with harmful materials.

7.6.2.12 Use of reinforcement contaminated with detrimental materials (such as chlorine or sulfate) and those that are rosted up to flaking is not permitted.

7.6.2.13 Stirrups used for tying reinforcement or keeping them in place shall be bent towards inside the form so that the concrete cover thickness is not reduced.

7.6.2.14 Concrete cover over reinforcement shall comply to the table 8-2-9-1 (extra severe environmental conditions).

7.6.2.15 Requirements of curing fresh concrete shall be applied in accordance with 7.6.1.7 through 7.6.1.9 .

7.6.3 Cold weather concreting

7.6.3.1 Cold air is referred to the situation where the following conditions persists for three consecutive days:

a) Average air temperature over a day and night is less than 5˚C (average daily temperature is the maximum and minimum average air temperature in the time period from midnight to the next midday).
b) Air temperature dose not exceed 10˚ C for greater half of the day.

7.6.3.2 Precautionary measures

- Cold weather concreting requires care in materials selection, concrete mix design, mix conditions, handling, placing and curing so as to ensure that freshly placed concrete does not freeze and that hardened concrete has the required quality.

- Concrete temperature shall be recorded during placing and curing to ensure that the range recommended in this code is preserved.
- Concrete temperature at different points in the structure shall be recorded at least twice in a day and night so that concrete maintenance is adequately ensured.
- As corners and edges of concrete are vulnerable to freezing, temperature at these points shall therefore be controlled closely.

7.6.3.3 Materials used

a) Rapid setting (type III Portland) cement may be used instead of normal cement to made sure that more rapid strength gain is obtained.
b) Use of slag cement or blended cements is not recommended for cold weather concreting.
c) Hot water may be used to increase concrete temperature to the desired level; in that, direct contact between cement and hot water is avoided. This issue is taken into consideration when placing materials inside mixer.
d) Aggregates shall not be associated with ice and snow. Sand is usually wetter than gravel and there is probably more ice in it Heating up sand may therefore be necessary.
e) Maximum water absorption of aggregates used in concrete is limited to 2.5 percent for coarse aggregates and 3 percent for fine aggregates, based on T.C.S.B. 210 and 211 .
f) Use of air entraining materials to make air entrained concrete is required for cases where concrete is exposed to moisture and frequent freezing and thawing.
g) Rapid setting materials or concrete anti-freez agents may be used when there is danger of freezing, provided they comply with standard requirements.

7.6.3.4 Concrete mix design requirements

a) water per cement ratio shall be based on the trend of strength gain of concrete in the ambient temperature; the ratio shall not exceed 0.5. It is therefore required to take necessary measures before concreting so that strength gain of concrete is ensured.
b) To reduce the amount of freezable water in concrete and also the amount of bleeding in fresh concrete, water content of the mix shall be the minimum possible. Hence, plasticizer and water-reducing additives may be used to maintain required workability.
c) If plasticizers are not used, concrete slump shall not be taken greater than 50mm.
d) Percentage of the required air bubbles in the mix design shall be in compliance with the values of the table 6.3.3.1 .

7.6.3.5 Minimum concrete temperature

- Minimum permitted concrete temperature during mixing, placing and maintenance, and the maximum permitted gradual loss of temperature during the first 24 hours after the end of concrete protection period shall be in accordance with the table 7.6.3 .
- Concrete temperature during mixing shall not exceed the values of the table 7.6.3 by more than 8 deg celcius (centigrade) as it causes greater energy dissipation, severe loss of slump and finally, reduction in concrete quality.
- When placing, concrete temperature shall not exceed the values in the table by more than 11˚C, otherwise it leads to reduced concrete quality.

7.6.3.6 Issues on handling and placing concrete

- Handling and placing concrete shall be such that fresh concrete does not lose its temperature; concrete shall be transported in covered and insulated vessels.

- Reinforcement, form, hardened surface of the previous concrete and of the ground shall be cleared of any frosting.

Table (7.6.3): concrete temperature in terms of degree celcius at different stages of work considering environmental temperature and the size of elements and members

No.	Description	Environmental temperature	Size of elements and members (mm)			
			Less than 300	300 to 900	900 to 1800	greater than 1800
1	Minimum concrete temperature during mixing	greater than –1	16	13	10	7
2		-18 to –1	18	16	13	10
3		less than –18*	21	18	16	13
4	Minimum concrete temperature during placing and maintenance	any quantity	13	10	7	5
5	Maximum permitted gradual loss of concrete temperature after the end of protection	any quantity	28	22	17	11

* If special provisions are not taken for mixing and concrete placement, concreting at- 30 deg celcius (centigrade) and less is prohibited

7.6.3.7 Curing fresh concrete

a) Curing fresh concrete shall continue for at least 24 hours and until concrete is reached the strength of 5 MPa.
b) The following methods may be used for curing fresh concrete and protecting it from freezing:
- Using insulating covers
- Heating concrete and its surrounding environment
- Other methods approved by the inspection system

c) Fresh concrete shall be protected against wind, especially after removing covers. Care shall be taken to prevent excess evaporation of water leading to carbonation of concrete surface due to combustion of fuel compounds for heating.

7.6.3.8 Protection of hardened concrete

It is required that freezing of the saturated concrete that has not yet gained a strength of 24 MPa be prevented. Standard methods or site sampling may be used to distinguish that concrete has gained adequate strength. Compressive strength of concrete may also be determined by standardized nondestructive methods

❒7.7 Special methods of concrete application

7.7.1 Shotcrete

Shotcrete is a process in which concrete or mortar is jetted over the surface until a compact, self sustaining and load-bearing layer is produced. Where the work configuration is complicated or forming is difficult and costly, especially for rehabilitation of buildings and bridges, this type of concrete is used. Application of shotcrete requires experience, provision of proper measures and technically expert labor, particularly for shotcreting. The main advantage of this type of concrete as compared to the usual concrete is the mere need to an internal form or a surface available for shotcreting. For this reason shotcreting has found ever-increasing application for round arch surfaces such as tunnels and chimneys. Based on the time to add water to the mixture of aggregates and cement, shotcrete is divided into two groups: “dry shotcrete” and “wet shotcrete”.

Regarding maximum size of aggregates used, shotcrete is classified into three groups.

7.7.1.1 Dry shotcrete

a) The bonding material, aggregates and if necessary, powder additives are mixed.
b) The prepared mix is fed into the special mechanical feeding instrument (or concrete jet).
c) Using mechanical instruments equipped with measurement units, under air pressure, the mix is guided through the special hose to the nozzle tip. Pressurized water is guided, through a rim carefully installed in the hose, to the nozzle where it is thoroughly mixed with other constituent ingredients of concrete being jetted rapidly out of the gun to the surface considered.

7.7.1.2 Wet shotcrete

a) Constituent materials and mix water (other than accelerating agents) are mixed thoroughly.
b) Mortar or concrete is transferred to the instrument chamber.
c) Through measurement units, the mixture is guided in the conveying hose and to the gun, under air pressure.
d) Accelerating agent is added to the matrix at the nozzle tip.
e) To increase speed and to improve shotcreting trend, more air is transmitted to the gun.
f) The mixture is jetted rapidly through the gun to the surface considered.

7.7.1.3 General specifications

Preparation of shotcrete mix design requires special attention to ensure proper bond, adequate compaction and desired physical properties. Water per cement ratio for this type of concrete is usually in the range of 0.35 to 0.50 . Maximum aggregate size in the mix is 20 mm, and gradation is in accordance with T.C.S.B. 521. specific gravity of shotcrete in similar to that of regular compacted concrete. Addition of silica fume to this type of concrete usually causes an appreciable reduction in buckling and rebound of the shotcrete aggregates. When determining shotcrete mix proportions it must be noted that part of the mixture is lost during shotcreting, due to rebound of aggregates. Therefore, composition of the sprayed shotcrete differs from its original composition. In that, early mixture or the mixture being jetted out of the gun, is different from that sprayed over the surface. Because of this difference, close control and laboratory tests at various stages of shotcreting is necessary. Due to the unusual speed of aggregates as they are jetted out of the gun, observance of safety factors is of special importance to the operating personnel. Reinforced shotcrete has recently found many applications, especially in tunnels.

Details of shotcrete shall be covered in job specifications.

7.7.2 Underwater concreting

7.7.2.1 General specifications

When underwater concreting is intended, hopper and pipe (or tremie), or pump may be used for concreting.

a) concreting by tremie

For this method, care must be taken so that cementicious materials are not washed out by water current. Underwater placed concrete shall have cementitious material content at least 350 Kg per cubic meter of concrete for high workability. Water per cement ratio shall not exceed 0.45 . Hopper and pipe system shall be thoroughly watertight so that concrete can easily flow in. The pipe shall be full of concrete all through concreting period.

Tremie pipe diameter shall be at least eight times that of maximum size of aggregate used. Concrete slump shall be taken between 170 to 250 mm.

b) concreting by pump

For concreting by pump, concrete mix design shall be selected such that water per cement ratio has the maximum possible value but it does not exceed 0.6. Cement content shall be relatively high in the range of 200 to 350 Kg per cubic meter of concrete so that adequate bond is provided and the danger of cement being washed out is omitted. To increase concrete workability, rounded aggregates with plane surface may be used. Use of uniform gradation with maximum aggregate size of 38 mm, and also adequate amount of fine aggregates is required. If aggregates do not contain sufficient quantity of fine grains, required bond may be induced in the concrete by adding fine aggregates.

Concrete being pumped shall be somewhat more viscous than usual, provided pipes are prevented from being clugged. Flowing mixes may be pumped if appropriate additives such as superplasticizers or water-retaining agents are used.

Except when special additives are used, free fall of concrete into water shall be prevented so that segregation dose not occur.

7.7.2.2 Implementation method

a) Hydraulic compression difference inside the form and out of it shall be nullified during concreting and the water level shall be uniform there.
b) When concreting with hopper and pipe, the lower end of pipe shall be inside concrete for a length of at least 1 to 1.5m. so that water can not enter the pipe from the bottom. For this, the pipe shall be gradually pulled out as it gets filled with concrete.
c) Horizontal lift joints separating various concrete lifts shall be avoided.
d) When concrete surface reaches the upper limit considered, that portion of the concrete blended with external materials; where, aggregates and

the matrix are segregated shall be continuously removed until a well consistent concrete is exposed.

e) Use of other underwater concreting methods is permitted only with recommendation and approval of the inspection body.

Details of underwater concreting shall be covered in the job specification.

7.7.3 Prepacked concrete

For concreting special members like some confinements in nuclear cells or spaces where pouring, placing and compaction of fresh concrete is not easily done, and for making heavy concrete, prepacked concreting method is used. In this method, coarse aggregates are properly placed inside the form or at concreting location and then distributed and compacted cement paste, usually containing plasticizer and expansive materials are continuously and uniformly injected, under a compression force between 5 and 8 atmospheres, over empty spaces among aggregates.

Smallest aggregate size used shall be at least eight to ten times the largest size of sand grains in the slurry. Form and the elements confining concreting location shall be water sealed to withstand cement paste pressure. A pipe shall be installed at the top of form or at the highest point of the space subject to concreting so that air can exit during paste injection, which shall continue until the paste is ejected from air pipe.

The paste overflowing from air pipe has plenty of air bubbles at the beginning. Hence, injection shall continue until no air bubbles are available in the outgoing paste.

To maintain workability of the slurry, coarse aggregates shall be selected to be of low water-absorption type. When prepacked concrete is considered to be used, necessary tests shall be done with the same type of aggregates and cement chosen for the concrete to acquire the most suitable grading with respect to the cement and the injection pressure.

7.7.4 Vacuum concrete

7.7.4.1 General

This method is used for concreting the slabs and some thin walled members. The main objective is to improve quality and increase concrete durability by water vacuum process.

7.7.4.2 Implementation method

a) To improve quality of the upper layer in reinforced concrete slabs, after pouring, placing and leveling according to conventional methods, water on some areas close to the concrete surface is vacuumed out with the help of devices connected to a pump and the surface is then finished. Hence, part of the concrete mix water and the bubbles trapped in upper layer is forced out so that the concrete close to the top surface is more dense and its abrasion durability is especially increased.

b) For making thin walled hollow members, tiny boles are drilled on the inner form surface, which is then covered with cloth or porous sheets. Easy-flowing concrete is then poured between inner form, wrapped in porous cover, and the outer form. The inner form is connected to a pump.

Excess water only, but not cement is pumped out of the concrete so as to make it denser and more viscous. Vacuum pump is then turned off and the inner form easily pulled out, and the outer form is opened.

❒7.8 Control and inspection

To ensure the conformance of manufacturing equipment and production method with standards and requirements of the code, minimum control and inspection frequency shall conform with the table 7.8.

Table 7.8 : Control and inspection of equipment for making concrete

No	Description of equipment	Type of inspection test	Objective	Reptition duration
1	Materials depot, silo and other	Optical inspection	Assurance for conformance with required issues	Once a week
2	Equipment related to measurement	Optical inspection of the work flow	Assurance for well performance of weight measurement equipment	Daily
3		Weight measurement test	Assurance of the precision considered	1-At the inspection period 2-Alternatively, as recognized by the inspection body
4	Measurement and weighing machine for additives	Optical inspection of the work flow	Ensure that weighing and measurement machines are clean and work accurately	For the first batch of each additive in a day
5		Accuracy test	Non-uniform weighing shall be avoided	1-At the installation period 2-monthly, after installation 3-In cases of doubt and when recognized by the inspection body

Table 7.8 (continued)

<table>
<tr><td>6</td><td>Hydrometer</td><td>Comparison of the real value with the value read from measurement unit</td><td>Ensure the accuracy considered</td><td>As described above in 1 and 2</td></tr>
<tr><td>7</td><td>Equipment for continuous measurement of vapor content in fine aggregates</td><td>Comparison of the real value with the value read from measurement unit</td><td>Ensure the accuracy considered</td><td>As described above in 1 and 2</td></tr>
<tr><td>8</td><td rowspan="2">Batching and mixing system</td><td>Optical inspection</td><td>Ensure accuracy of batching</td><td>As described above in 1 and 2</td></tr>
<tr><td>9</td><td>Comparion of actual mass of constiutent components of mix with the considered mass, based on a proper method</td><td>Ensure that batching and mixing system operates properly</td><td>Daily</td></tr>
<tr><td>10</td><td>Test equipment</td><td>Necessary tests according to standards and other regulations</td><td>Conformance control</td><td>Based on type of test instrument, regularly but at least once a year</td></tr>
<tr><td>11</td><td>Mixer (concrete mixing and handling trucks)</td><td>Optical inspection</td><td>Mixer equipment erosion control</td><td>Monthly</td></tr>
</table>

CHAPTER EIGHT

REINFORCEMENT DETAILS

❐8.0 Notation

A_b = Cross-sectional area of a skin reinforcement bar for mass concrete, mm^2

$A_{S\min}$ = Minimum of shrinkage and temperature reinforcement, mm^2

d_b = Nominal diameter of bar or wire ,mm

d_c = Distance of the center of skin reinforcement bar to concrete surface in mass concrete, mm

h = Slab thickness, mm

s = Center to center spacing of the bars in mass concrete, mm

α = Adjusting factor for temperature and shrinkage reinforcement

❐8.1 Specification and construction details

8.1.1 Mechanical properties of bars

Mechanical properties of reinforcement bars shall comply with requirements of clause 4.4.

8.1.2 Cutting the reinforcement bars

cutting the reinforcement bars shall be done mechanically, usage of other methods needs to be approved by the inspection system. When the full length of coldworked twisted bars are to be used or when bars are to be spliced by butt welding, the untwisted ends of bars shall be cut.

8.1.3 Bending of bars

8.1.3.1 All reinforcement shall be bent cold, unless otherwise permitted by the engineer.

8.1.3.2 As far as possible, bending of bars shall be carried out mechanically by machines equipped with a bending wheel. Bending shall be completed in one pall through the machine at high speed, so that the bent section shall have a constant curvature.

8.1.3.3 Wheels used for bending of bars shall be of a diameter appropriate for the type of steel to be bent. Also see clause 8.2.4.4 for minimum diameter of bend.

8.1.3.4 Speed of bending of bars shall be appropriate to the type of steel and the ambient temperature. Speed of bending of cold-worked steel bars shall be determined by experiment.

8.1.3.5 Where the temperature of the steel is below 5˚c , bending of steel shall not be permitted.

8.1.3.6 Generally, reshaping of steel previously bent is not permitted unless with the engineers approval. Each bar should be inspected for signs of fracture.

8.1.3.7 Reinforcement partially embedded in concrete shall not be field bent, except as shown on the design drawings or permitted by the engineer.

8.1.4 Transportation and storage of reinforcement

During the period between delivery and fixing of reinforcement in the structure, the following requirements shall be complied with:

8.1.4.1 Mechanical damage or plastic deformation such as cuts, shock loading and the dropping of reinforcement from a height shall be avoided.

8.1.4.2 Failure of welds in welded mesh fabrics shall be prevented.

8.1.4.3 Symbols and signs identifying type of reinforcement shall be protected.

8.1.4.4 Reinforcement shall not be exposed to substances that can be shown to reduce the bond, such as mud, grease or other nonmetallic deleterious materials.

8.1.4.5 Reinforcement shall not be exposed to corrosion to such an extent that might lead to a reduction in the cross-sectional area.

8.1.5 Surface conditions of reinforcement

Reinforcement shall be free from mud, oil, paint, retarders, loose rust, loose mill scale, snow, ice and grease, prior to placing. Steel reinforcement with rust, mill scale or both shall be considered satisfactory, provided their standard specification is maintained in accordance with 4.4.1 and 4.8.2 , after cleaning. In any case, reinforcement can only be used after cleaning of any rust or mill scale present.

8.1.6 Placing and fixing of reinforcement

8.1.6.1 Prior to placing of concrete, reinforcement shall be accurately placed as shown on construction drawings and adequately supported and shall be secured against displacement within tolerances permitted in subclause 8.1.6.2 .

8.1.6.2 Unless otherwise specified by the engineer, reinforcement shall be placed within the following tolerances:

a) Maximum tolerance for thickness of the protecting concrete cover over the reinforcement 8 mm.
b) Tolerance for location of reinforcement in relation to sectional depth of flexural member, thickness of walls and/or smallest column dimension:
   - Up to and including 200 mm ± 8mm
   - Between 200 mm to 600 mm ± 12 mm
   - 600 mm or more ± 20mm
c) Tolerance for lateral spacing of bars ± 30 mm
d) Tolerance for longitudinal location of bends and ends of bars:
   - Ends of discontinuous members ± 20 mm
   - Other cases ± 50 mm

8.1.6.3 Welded wire fabric with wire size not greater than 6 mm used in slabs not exceeding 3m in span may be curved from a point near the top of slab over the support to a point near the bottom of slab at midspan , provided such reinforcement is either continuous over, or securely anchored at support.

8.1.6.4 Maximum permitted tolerances specified in subclause 8.1.6.2 (a) for concrete covers to reinforcement apply to cases where the concrete covers to reinforcement apply to cases where the concrete cover is not less than 2/3 of the specified cover.

Concrete covers to main reinforcement and ties shall be shown on design drawings.

8.1.6.5 Material, size and spacing of spacer blocks and chairs used to maintain reinforcement in its correct place shall be such that, in addition to safeguarding of subclause 8.1.6.2 , they shall not hinder concrete placing, strength and durability.

8.1.6.6 Non- structural connections for positioning of reinforcement shall be made with steel wire or lying devices. Care should be taken to ensure that projecting ends of ties or clips do not encroach into the concrete cover.

8.1.6.7 Welding of crossing bars with arc electricity shall not be permitted to assemble reinforcement unless for weldable steel and after inspector engineer's approval. In this case welding shall not cause reduction in the cross sectional area or damage to the bars.

❐8.2 Reinforcement detailing

8.2.1 Types of reinforcement used

Types of reinforcement used shall comply with clause 4.4.

8.2.2 Nominal diameters

Nominal diameters shall comply with clause 4.3.

8.2.3 Different type of steel shall not be used together in one member, unless,

a) Their different mechanical characteristics are allowed for in the design.
b) There is no risk of manufacturing mistake in the use of one type of steel for longitudinal reinforcement and another type of steel for lateral reinforcement when in compliance with (a).

8.2.4 Anchorage of reinforcement

8.2.4.1 General

Common methods for anchoring reinforcement bars include:

a) Straight anchorages,
b) Curved anchorages (Such as hooks and loops),
c) Straight anchorages with a minimum of one lateral reinforcement welded to them in the anchorage zone,
d) Mechanical anchoracies,
e) A combination of the above methods.

Reinforcement anchorage shall comply with the following general requirements:

a) Straight anchorages shall not be considered effective in developing plain bars in tension.
b) Hooks shall not be considered effective in developing bars in compression.
c) When using straight anchorages (embedment length) with a minimum of one lateral bar welded to them, or mechanical devices or when a combination of different types of anchorage methods are to be used, they shall comply with the requirements of chapter eighteen.

8.2.4.2 Standard hooks

In this code the following bend details are considered as standard hook:

a) Main reinforcement
- Half a circle (180 $^{\circ}$end hook) plus a minimum of 4 d_b straight, but not less than 60 mm in the free end of the bar.
- 90° bend plus straight length equal to at least 12 d_b in the free end of the bar.
- 135°bend plus at least 8 d_b straight length in the free end of the bar

b) Distribution reinforcement and ties
- 90° bend plus at least 6 d_b straight length but not less than 60 mm in the free end of the bar, for up to and including 10mm diameter bars
- 90°bend plus at least 12 d_b straight length in the free end of the bar, for 16mm to 25 mm diameter bars.
- 135° bend plus at least 6 d_b straight length but not less than 60mm in the free end of the bar.

When using non-standard hooks, full details shall be shown on construction drawings.

8.2.4.3 Permitted diameter of bends

Minimum permitted diameter of bends for reinforcement is based on the following considerations:

a) Avoidance of breaking or bursting of concrete caused by concentrated pressures inside the bend.
b) Compliance with the minimum wheel diameter that would lead to a satisfactory rebind test result.

8.2.4.4 Minimum diameter of bends

a) internal diameter of bends not including ties shall not be less than the values given in table 8.2.4.4 (a).

Table 8.2.4.4: (a)-Minimum bend diameters for main reinforcement

Bar diameter	Minimum bend diameter for main reinforcement		
	S 220	S 300, S 350	S 400 and S 500
< 28 mm	$5\ d_b$	$5\ d_b$	$6\ d_b$
28mm-34mm	$5\ d_b$	$6\ d_b$	$8\ d_b$
36mm-55mm*	$7\ d_b$	$10\ d_b$	$10\ d_b$

* For bending of bars greater than 36 mm in diameter , with an angle greater than 90˚, special technics are required.

b) Internal diameter of bends for ties greater than 16mm in diameter shall not be less than the values given in table 8.2.4.4 (a), and for ties less than 16mm in diameter shall not be less than the values given in table 8.2.4.4(b).

Table 8.2.4.4(b):-Minimum bend diameters for ties

Bar diameter	Minimum bend diameter for ties		
	S 220	S 300, S 350	S 400 and S500
16 mm or less	$2.5\,d_b$	$4\,d_b$	$4\,d_b$

c) Internal diameter of bends in welded mesh fabrics, plain or deformed , when used as distribution reinforcement shall not be less than $4\,d_b$ for deformed bars greater than 7mm in diameter and less than $2\,d_b$ for other wires. Bends with than 8 d_b internal diameter shall not be closer than 4 d_b to the nearest welded joint.

8.2.5 Splices of reinforcement

Common methods for splicing reinforcement are as follows:

- Lap splices (contact or noncontact)
- Welded splices
- Mechanical connections
- Composite splices
- End- bearing splices

8.2.5.1 Lap splices shall comply with the following requirement

a) In lap splices, required ends of the two bars to be lapped shall be placed along side each other as shown on the construction drawings.

In contact lap splices, the two bars to be lapped shall be in full contact with each other. In non-contact lap splices, the two bars to be lapped shall be placed at the following maximum distances:

- In flexural members, the distance between centers of the two bars to be lapped shall not exceed 1/5 of the required lap length or 150mm.
- In other members, the distance between centers of the two bars to be lapped shall not exceed 5 times the smaller bar diameter.

b) In lap splices of plain bars in tension, the ends of the bars shall be bent as one of the standard hooks with an angle greater than 135°.
c) Noncontact lap splices shall be bounded by lateral bars placed at right angles to the direction of the spliced bars.
Lap splices shall comply with requirement of chapter eighteen.

8.2.5.2 End-Bearing splices shall comply with the following requirements:
a) In end-bearing splices, bars shall be cut square and held in concentric contact by a suitable device.
b) End-bearing splices may only be used for bars in compression and larger than 25mm in diameter.
c) Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars.
d) End bearing splices shall be used only in members containing closed ties, closed stirrups or spirals.
End- bearing splices shall comply with requirements of chapter eighteen.

8.2.5.3 Common methods for welded splices are as follows:
a) Plastic butt welded joint splice (contact electric weld)
b) Fusion weld joint with electrodes (electric arch weld)

Plastic butt welded joints shall be used only under factory conditions and only for not-rolled steel bars larger than 10mm in diameter or for cold-worked steel bars larger than 14mm in diameter. The ratio of the cross sectional areas of the bars to be spliced shall not exceed 1.5 .

Fusion welding with electrodes shall be permitted only when appropriate electrodes and welding techniques are used for the type of steel to be welded.

Common methods of fusion welding with electrodes are as follows:
- Side–by–side weld jointing welded from one or both sides: is permitted only for hot-rolled bars from 6mm to 36mm in diameter. In this

method, the length of weld in one-sided welded lap jointing shall not be less than 10 times the diameter of the smaller bar to be jointed and for two-sided welded lap jointing, the length of weld shall not be less than 5 times the smaller bar diameter.

- Weld jointing with additional splice or splices welded from on or both sides is permitted for hot-rolled steel bars only. The required weld length are the same as those for side-by-side weld jointing.
- Butt weld and buttress with backing with or without prepared bar ends, the length of backing shall not be less than three times the bar diameter for hot-rolled steel bars and eight times bar diameter for cold–worked steel bars. The distance between bar ends to be spliced shall be 3mm for prepared bars and equivalent to half their diameter for unprepared bars. For splicing of cold-worked steel bars, ends of both bars to be jointed must be prepared. For vertical or near vertical bars, the end of the upper bar must be prepared and the lower bar shall be cut square.

For welded splice requirements, see chapter eighteen.

8.2.5.4 In mechanical splices, mechanical devices shall be used so as to enable jointing of bars without damaging concrete.

For requirements of mechanical splices see chapter eighteen.

8.2.5.5 In composite splices, a combination of different splicing techniques discussed above are used together. For requirements of composite splices, see chapter eighteen.

8.2.6 Spacing limits for reinforcement

8.2.6.1 Clear distance between parallel bars in a layer shall be not less than:

a) The largest bar diameter.
b) 25mm

c) 1.33 times the largest nominal size of aggregates.

8.2.6.2 Where parallel reinforcement is placed in two or more layers. Bars in the upper layers shall be placed directly above bars in the bottom layer with a clear distance between layers not less than largest bar diameter, nor 25 mm.

8.2.6.3 In spirally reinforced or tied reinforced compression members, the clear distance between longitudinal bars shall be not less than 1.5 times the largest bar diameter nor 40 mm.

8.2.6.4 for spacing limits of reinforcement at lap splice locations see subclause 8.2.5.1.

8.2.6.5 Clear distance limitations between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

8.2.6.6 In walls and slabs with exception of thin slabs having concrete joint construction, primary flexural reinforcement shall be spaced not further apart that two times the thickness of slabs or three times the walls thickness and 350 mm.

8.2.7 Bundled bars

8.2.7.1 Groups of parallel reinforcing bars bundled in contact to act as a unit shall comply with the following requirements:

a) The number of bars in any one bundle shall be limited to 4 vertical bundles in compression and 3 in other cases.
b) In all cases, the number of bars at splice locations shall not exceed 4 in any one bundle.

c) In bundles comprising of 3 bars or more, The axes of the bars shall not all be in one plane. No more than 2 bars shall lay in one plane except at splice locations.
d) Bars larger than 36 mm shall not be bundled in beams.
e) Bundled bars shall be enclosed within closed stirrups or ties.
f) Where spacing limitations and minimum concrete cover on based on bar diameter, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.
g) Grouping of bars to be used as bundles in contact shall be permitted only when shown on construction drawings.
h) For anchorage, splicing and curtailment of bundles bars see clause 18.4.

8.2.8 Concrete cover for reinforcement

8.2.8.1 Concrete cover to reinforcement is equal to the minimum distance between the surface of the bar, lateral or longitudinal, to the nearest exposed concrete surface.

8.2.8.2 Concrete cover to reinforcement, specified in clause 8.2.9 is not required to the end of straight bar in a floor or roof unit where its end is not exposed to weather or condensation.

8.2.9 Concrete cover depth to reinforcement

The following minimum concrete cover shall be provided for reinforcement:

a) Diameter of bars. For bundied bars see subclause 8.2.7.1 (g).
b) The largest nominal size of aggregates up to 32mm, or 5mm more than nominal size of aggregates larger than 32mm.

8.2.9.2 Depth on concrete cover to reinforcement, appropriate to type of member and exposure conditions shall not be less than the values given in table 8.2.9.1.Exposure conditions are categorized as follows:

a) Mild exposure conditions are referred to those conditions in which, concrete surfaces are protected against weathering or aggressive conditions such as moisture, condensation, frequent wetting and drying, freezing and thawing, frequent heating and cooling, contact with aggressive or nonaggressive soils, corrosive substances, severe abrasion, heavy vehicular traffic or impact, or when the element is properly protected against such aggressive actions.

b) Moderate exposure conditions: concrete subject to moisture or occasional condensation, concrete in contact with non-aggressive soils or water, concrete surfaces continuously under water carrying pH > 4.5.

c) Severe exposure conditions are referred to those in which, concrete surface is exposed to severe moisture or condensation, frequent wetting and drying, frequent freezing and thawing and repeated heating and cooling.

d) Very severe exposure conditions are referred to those in which, concrete surface is exposed to gases, still water and wastewater with maximum PH value equal to 5, corrosive materials or moisture along with severe freezing and thawing.

e) Extremel severe exposure conditions are referred to those in which, concrete surface is exposed to severe abrasion, vehicular traffic or flowing water and wastewater with maximum value of PH equal to 5.

Extreme exposure conditions of islands and areas around the Persian gulf and Oman sea is included in this exposure condition.

Thickness of concrete layer to protect reinforcement, in accordance with exposure conditions, concrete quality and the member considered shall not be less than the values given in table 8.2.9.1

Table 8.2.9.1- Minimum concrete cover,mm.

Member type	EXPOSURE CONDITIONS				
	Extreme	**Very severe**	**Severe**	**Moderate**	**Mild**
Beams and columns	75	65	50	45	35
Slabs,walls and joists	60	50	35	30	20
Shells and folded plate members	55	45	30	25	15
Footings	40	50	60	75	90

*The cover values given in the table may be reduced by 5 mm for concrete grades C35 and C40, or by 10 mm for higher grades concrete, except for (c) and (d) exposure conditions, provided that concrete cover does not fall below 20 mm.

The concrete covers given in table 8.2.9.1 shall be increased by 10mm where reinforcement larger than 36 mm in diameter is to be used.

8.2.9.3 where concrete is cast nearby strong earthen walls and is permanently in contact with it, the concrete cover shall not be taken less than 75 mm.

8.2.9.4 Where there are recesses or projections in the concrete surface, concrete cover shall be measured in the recesses.

8.2.9.5 Concrete cover as fire protection to reinforcement

Where there is a requirement for the concrete member to have a certain fire rating, the minimum depth of concrete cover shall be specified by the relevant authorities.

8.2.9.6 Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

❐8.3 Special reinforcing details for columns

8.3.1 Bent dowel

Bent dowel bars of the column, at the section change shall conform to the following:

8.3.1.1 Slope of inclined portion of a dowel bar with respect to the column axis shall not exceed 1 to 6. Portions of bar above and below the dowel shall be parallel to axis of column.

Horizontal support at dowel bends shall be provided by lateral ties, spirals, or parts of the floor construction. Horizontal support provided shall be designed to resist 1.5 times the horizontal component of the computed force in the inclined portion of a dowel bar. Lateral ties or spirals, if used shall be places not more than 50mm from points of bend.

8.3.1.2 Dowel bars shall be bent before placement in the forms.

8.3.1.3 Where a column or wall face has a projection or recess by 74mm or more, longitudinal bars shall not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column shall be provided. Requirements for development and splices in the section change rogion shall be observed in all cases.

❐8.4 Lateral reinforcement for compression members

8.4.1 Lateral reinforcement for compression members shall conform to the provisions of clauses 8.4.2 and 8.4.3 and, where shear or torsion reinforcement is required, shall also conform to provisions of chapter towelve.

8.4.2 Spirals

Spiral reinforcement for compression members shall conform to chapter eighteen and to the following:

8.4.2.1 Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled as to permit handing and placing without distortion from designed dimensions.

8.4.2.2 Size of bar or wire used as spirals shall not be less than 6mm.

8.4.2.3 Clear spacing between spirals shall not exceed 75mm not exceed 75mm nor be less than 25mm.

8.4.2.4 Clear spacing between spirals shall not exceed 1/6 of the diameter of the concrete core inside the spirals.

8.4.2.5 Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

8.4.2.6 Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

8.4.2.7 In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is two times that of the column.

8.4.2.8 Spirals shall be held firmly in place by proper spacers.

8.4.2.9 For spiral bar or wire smaller than 16mm diameter a minimum of two spacers shall be used for spirals less than 500mm in diameter, three spacers for spirals 500mm to 750mm in diameter, and four spacers for spirals greater than 750mm in diameter.

8.4.2.10 For spiral bar or wire 16mm in diameter or larger , a minimum of three spacers shall be used for spirals 600mm or less in diameter , and four spacers for spirals greater than 600mm in diameter.

8.4.2.11 Anchorage of spiral reinforcement shall be provided by 1.5 extra Tums of spiral bar or wire at each end of a spiral unit.

8.4.2.12 Splices in spiral reinforcement shall conform to requirements of chapter eighteen.

8.4.3 Ties

8.4.3.1 All bars for compression members shall be enclosed by lateral ties.

8.4.3.2 Diameter of tie reinforcement shall not be les than the following:

a) 1/3 of the largest longitudinal bars with maximum diameter of 30mm.
b) 10mm in size for longitudinal reinforcement larger than 30mm in diameter and bundled longitudinal bars.

8.4.3.3 Diameter of ties shall not in any case be less than 6mm.

8.4.3.4 Spacing between two consecutive ties should not exceed the followings:

a) 12 times the longitudinal bar diameter whether as singular or as an element of reinforcement group
b) 36 times the tie bar diameter
c) The least dimension of the compression member
d) Or 250mm

8.4.3.5 Ties shall be arranged in a way that every corner and alternate longitudinal bar will have lateral support provided by the comer of a tie with an included angle of not more than 135^0, and no bar shall be further than 150mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of circle, a complete circular tie may be used provided the end of the ties are bent to a standard 135^0 hook or properly anchored in the concrete inside the tie.

8.4.3.6 Ties shall be located not more than ½ a tie spacing above the top of footing or slab in any story , and shall be spaced as provided herein to not more than ½ a tie spacing below the lowest horizontal reinforcement in slab or drop panel above determined by 8.3.4.3.

8.4.3.7 Where beams or brackets frame from our directions into a column, ties may be terminated not more than 75mm below lowest reinforcement in the shallowest of such beams or brackets.

8.4.3.8 Splices and laps of ties shall conform to requirements of chapter eighteen.

❒8.5 Lateral reinforcement for flexural members

8.5.1 Lateral reinforcement shall conform to chapter twelve requirements.

8.5.2 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in clause 8.4 or by welded wire fabric of equivalent area. Such ties or stirrups shall be provided throughout the distance where compression reinforcement in required.

8.5.2.1 Lateral reinforcement for flexural farming members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

8.5.2.2 Plain bars larger than 16mm in diameter shall not be used as ties.

8.5.2.3 Closed ties or stirrups may be formed in one place by overlapping standard stirrup or tie and hooks around a longitudinal bar, or formed in one or two pieces lap spliced or anchored in accordance with chapter eighteen.

❐8.6 Lateral reinforcement in connections

8.6.1 At connections of principal framing elements such as beams or columns, enclosure shall be provided for splices of continuing reinforcement and for anchorage of reinforcement terminating in such connections.

8.6.2 Enclosure at connections may consist of external concrete or internal closed ties, spirals , or stirrups.

❐8.7 Shrinkage and temperature reinforcement

8.7.1 Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided in structural slabs where the flexural reinforcement extends in one direction only, in accordance with clause 8.7.3.

8.7.2 In one-way and two-way spanning slabs, the total cross-sectional area of reinforcement in the direction of flexural reinforcement including top and bottom bars shall not be less than the values given in clause 8.7.3.

8.7.3 Amount of shrinkage and temperature reinforcement

8.7.3.1 Area of shrinkage and temperature reinforcement for slabs up to 1000mm thick, shall provide at least the following ratios of reinforcement area to gross concrete area:

- Slabs where grade S220 and S300 and S350 deformed bars are used 0.002.
- Slabs where grade S400 deformed bars or welded wire fabric (smooth or deformed) are used 0.018.
- Slabs where grade S500 or greater S500 or greater deformed bars are used 0.0015.

8.7.3.2 For slabs between 100mm to 2000mm thickness the minimum ratio of shrinkage and temperature reinforcement area to gross concrete area shall not be less than α times the ratios given in clause 8.7.3. where α is given in the following equation :

$$\alpha = 1.30 - 0.3h \qquad (8\text{-}1)$$

8.7.3.3 The minimum area of shrinkage and temperature reinforcement AS_{min}, for slabs thicker than 2m shall be taken to be equal to that required for 2m thich slabs and as follows:

- For grade S200 and S300 and S350 deformed Bars $AS_{min} = 2800\, mm^2/m$
- For grade S400 deformed bars and for welded wire fabric (smooth or deformed) $AS_{min} = 2500\, mm^2/m$
- For grade S500 or greater $AS_{min} = 2100\, mm^2/m$

8.7.3.4 In slabs and footings with variable thickness, for calculating the amount the temperature and shrinkage reinforcement, it's possible to use the thickness equal to the thickness of the imaginary slab and footing having the same volume.

8.7.4 Distribution of temperature and shrinkage reinforcement

8.7.4.1 Temperature and shrinkage reinforcement for slabs and footings of 1000mm thickness or less may be placed on one side of the slab only.

8.7.4.2 Temperature and shrinkage reinforcement for slabs and footings of greater than 1000mm thickness must be distributed on both sides such that the predicted amount of reinforcement of either side is no less than 1/3 of the total reinforcement needed.

8.7.5 Skin reinforcement

The amount of skin reinforcement in mass concrete can be defined by:

$$A_b = \frac{1.6\,\mathrm{d_c}\,s}{100} \tag{8-2}$$

This value shall never be less than bar with 10mm diameter in every 200 mm.

CHAPTER NINE

FORMWORK,EMBEDDED PIPES AND CONSTRUCTION JOINTS

❒9.1 General

9.1.1 Definitions

- Form :A temporary structure that supports fresh concrete until such time that concrete gains sufficient strength to support its own weight.
- Formwork :The whole system that retains fresh concrete in the required final shape , including forms , side forms , braces , ties etc.
- Falsework :A temporary structure that supports formwork, working platforms, and construction loads, including props, scaffolding, braces, transoms, standards, base plates etc.

9.1.2 Function of forms

9.1.2.1 Form shall result in a final structure that conforms to the permitted tolerances and the required concrete finish. Form shall support concrete untilsuch time that it gains sufficient strength to support its own weight.

9.1.2.2 Forms shall protect concrete from mechanical damage and prevent moisture loss and leakage of mortar, protect concrete from damage caused by cold or hot weather , maintain reinforcement and other fitting and parts that are placed within the concrete in the required position , resist forces that are caused by vibrating concrete, and allow removal without damage to concrete.

9.1.3 Drawings and specifications

Formwork and falsework drawings and specifications for special and complex structures or any other necessary cases shall comply with design requirements of clause 9.3 and tolerance requirements of clause 9.1.4 and 9.4.2.

9.1.4 Tolerances

Tolerances shall be as large as possible without undermining anticipated purposes of the structure or parts there of.

The basis for measurement of probable errors (tolerances) are the points and lines formed at the beginning of the work and are properly maintained to the end. If tolerances are not specified by the designer, deviation in dimensions and location of forms shall not exceed a specified range. Scope of the tolerances of forms in buildings and conventional reinforced concrete members are given in table 9.1.4.

For special structures, tolerances shall be given in the job specification.

Table 9.1.4 Tolerances in conventional concrete structures

<table>
<tr><th>No</th><th colspan="2">Description</th><th>Tolerances</th></tr>
<tr><td rowspan="4">1</td><td rowspan="4">Vertical extension deviation</td><td rowspan="2">a) At the edge and surface of columns, piers, walls, angles and corners</td><td>6mm in each 3m length</td></tr>
<tr><td>maximum 25 mm in total length</td></tr>
<tr><td rowspan="2">b) For exposed corner of columns, control joints, gutters and other exposed and important elevated lines</td><td>6mm in each 6m length</td></tr>
<tr><td>Maximum 12mm in total length</td></tr>
<tr><td rowspan="5">2</td><td rowspan="5">Diviation from surfaces or levels defined in the plans</td><td rowspan="3">a) At lower face of slabs, roofs, lower face of beams, angles and corners before removing bulkheads</td><td>6mm in each 3m length</td></tr>
<tr><td>9mm in each panel or any 6m length</td></tr>
<tr><td>maximum 12 mm in total length or any 6m length</td></tr>
<tr><td rowspan="2">b) At lintels, bearings, exposed parapets, horizontal gutters and other exposed and important elevated lines</td><td>6mm in any 6m length</td></tr>
<tr><td>Maximum 12mm in total length</td></tr>
<tr><td rowspan="3">3</td><td rowspan="3">Diviation of columns, walls and dividers from the specified location in the plan</td><td>In any panel</td><td>12mm</td></tr>
<tr><td>In any 6m length</td><td>12mm</td></tr>
<tr><td>Maximum at the entire length</td><td>25mm</td></tr>
<tr><td>4</td><td colspan="2">Size and location diviation for openings located on the floors, walls and sleeves</td><td>± 6mm</td></tr>
</table>

Table 9.1.4 - continued

<table>
<tr><td rowspan="2">5</td><td rowspan="2">Difference in dimensions of columns, transversal section of columns, beams and thickness of slabs and walls</td><td colspan="2">a) In the descending direction</td><td colspan="2">6mm</td></tr>
<tr><td colspan="2">b) In the ascending direction</td><td colspan="2">12mm</td></tr>
<tr><td rowspan="5">6</td><td rowspan="5">Footings</td><td colspan="2" rowspan="2">a) Dimension differences in the plan</td><td>Descending</td><td>12mm</td></tr>
<tr><td>Ascending</td><td>50mm</td></tr>
<tr><td colspan="2">b) Displacement or eccentricity</td><td colspan="2">Two percent of footing width in the direction of the length considered, provided it is not greater than 50mm</td></tr>
<tr><td rowspan="2">c) Thickness</td><td>Thickness reduction relative to the specified value</td><td colspan="2">5 percent</td></tr>
<tr><td>Thickness increase relative to specified value</td><td colspan="2">No limitation</td></tr>
<tr><td rowspan="4">7</td><td rowspan="4">Stairs</td><td rowspan="2">a) In a limited number of stairs</td><td>Stair height</td><td colspan="2">± 3mm</td></tr>
<tr><td>Stair floor</td><td colspan="2">± 6mm</td></tr>
<tr><td rowspan="2">b) In sequential stairs</td><td>Stair height</td><td colspan="2">± 1.5mm</td></tr>
<tr><td>Stair floor</td><td colspan="2">± 3mm</td></tr>
</table>

❐9.2 Materials

Materials for forms shall be selected in full consideration of economy, safety and the required surface finish. Physical and mechanical characteristics ofmaterials for construction of different parts of formwork such as sides, surfaces, attachments and supports, shall be fully considered.

❐9.3 Formwork design

9.3.1 Formwork design

Forms and their supports shall be designed so as to safely support construction loads , including vertical , horizontal and impact loads , until concrete gains sufficient strength.

9.3.2 Loadings

9.3.2.1 Vertical loads

The most important dead and live vertical loads are as follows:

a) Weight of forms and backstay,
b) Weight of fresh concrete,
c) Weight of reinforcement and fittings,
d) Weight of persons, equipment and work platforms,
e) Temporary loads caused by storing of materials,
f) Upward wind pressure,
g) Support reactions in prestressed concrete.

9.3.2.2 Lateral loads

The most important lateral loads imposed on forms are as follows:

a) Concrete pressure(thrust),
b) Wind pressure and suction,
c) Loads caused by temperature variations.

9.3.2.3 Special loads

The most important special loads to be considered are as follows:

a) Loads caused by asymmetrical concreting,
b) Impact loads due to machinery and concrete pump,
c) Upward forces on forms and embedded fittings,
d) Dynamic effects such as concrete discharge from hoppers,
e) Loads caused by differential settlement of form supports,
g) Injected grout pressure in prepacked concrete

❒9.4 Construction

9.4.1 General

9.4.1.1 It is recommended that top surfaces with greater than 2:3 (2vertical , 3horizontal) batter should be formed . However, forming of top surfaces steeper than 1:1 is mandatory.

9.4.1.2 Forms shall be fixed an release agent shall be applied to surfaces prior to placing of reinforcement.

9.4.1.3 Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

9.4.1.4 Forms shall be free from any contamination, mortar and foreign substances. Forms shall be covered with release agent before each use. Release agent shall be applied to form surfaces so as to give a uniform and thin film, without contaminating reinforcement.

9.4.1.5 When access to bottom of forms in difficult or impossible, inspection holes and drain holes shall be incorporated to enable cleaning prior to concrete placement.

9.4.1.6 When concrete surface finish is of special importance, forms damaged in previous uses shall not be used.

9.4.1.7 After removal of forms, bottom surfaces of reinforced concrete members shall be shored in accordance with clause 9.4.1.8 so as to prevent time-dependent deformations.

9.4.1.8 Provision for safety props for beams with spans greater than 5m, cantilever beams with lengths greater than 2.5m, slabs with spans greater than 3m , and cantilever slabs with lengths greater than 1.5m, is mandatory. Number of safety props shall be such that their spacing shall not exceed 3m.

9.4.2 Setting of formwork

Setting of formwork before, during and after placement of concrete shall be under careful inspection , so as to maintain the whole system within the limits of the dimensional tolerances specified.

9.4.3 Removal of forms and shores

9.4.3.1 General

a) Forms shall remain in place until such time that concrete can safely support effective stresses and its deformation shall not exceed the anticipated deformation.
b) Load-bearing forms and supports shall not be removed until such time that concrete elements have gained sufficient strength to support their own weight and loads placed thereon.
c) Forms and shores shall be removed in such manner as not to impair safety and serviceability of the structure. All concrete to be exposed by form removal shall have sufficient strength not to be damaged thereby.

d) When forms are removed before curing period expires, the provision of relevant curing methods shall immediately follow the removal of formwork as such early ages.

9.4.3.2 Striking period

a) When striking period is not specified in the design , the recommended periods before striking formwork given in table 9.4.3.2 shall be used.

b) Shorter periods before striding formwork are allowed only I famished by tests.

If field-cured test samples demonstrate at least 70% of the specified 28-day strength, soffit forms may be removed. However, removal of safety props shall only be allowed, when, in addition to all other considerations. The concrete can be demonstrated to have reached the required 28-day strength.

9.4.3.3 Removal of safety props

a) For beams up to7m spans, removal of the whole formwork and falsework may be permitted . However, for spans larger than 7m, the arrangement of formwork and falsework shall be such as to allow the the removal of forms without disturbing safety props.

b) For structures consisting of walls and reinforced concrete slabs, such as structures constructed with tunnel forms or with large size forms, removal and re-erction of safety props may be allowed for spans up to 10m, provided that safety props are erected immediately after form removal , and it can be demonstrated that no undue cracks or deformations will be caused.

c) In general, when a member forms part of the continuous system, safety props to that member may only be removed when the adjacent members have all been cast.

When the beam or slab under consideration has been designed as a continuous member, no safety props from any spans shall be removed until adjacent spans have been concreted and gained sufficient strength.

Table 9.4.3.2*-Minimum period before striking formwork.

Type of formwork		minimum period before striking surface temperature of concrete			
		24°c & above	16°c	8°c	0°c
Vertical formwork (hours)		9	12	18	30
Slabs	Soffit formwork (days)	3	4	6	10
	Safety props (days)	7	10	15	25
Beams	Soffit formwork (days)	7	10	15	25
	Safety props 9days)	10	14	21	36

*The given time periods are only valid with observance of the following issues:

-Concrete is made with type I or II usual portland cement or other cements with similar strength gain.

-If ambient temperature reduces to lower than zero degree celcius while concrete is hardening, the time durations given shall be modified according to the conditions in 7-6-3.

-If type III portland cement or accelerator agents are used, the given durations may be reduced.

-If retarding materials, typeV portland cement, or cements with similar strength gain are used, the given periods shall be increased.

-If special provisions are intended for prevention of cracks (especially in members and elements with different thicknesses or exposed to different temperatures), or for reduction of creep deformations, the given periods may be increased.

-If accelerated curing or special formwork is considered, reduction of the given periods is possible.

d) When formwork arrangement for an upper floor rests on a lower floor , safety props of the lower floor may be removed only after the upper floor concrete has gained the required strength. It is recommended that safety props are always provided in two successive floors, and as far as possible , similar props in the upper and lower floors shall be in line and in the same axis.

e) Removal or safety props shall be carried out without shock and impact so as to lead to a gradual load relief. (in large spans working from mid span towards supports and in cantilever spans from the end towards support).

   Dismantling of safety props in spans and in elements playing a critical structural role shall be carried out using adjustable means so as to ensure the ability to stop the dismantling process if necessary.

❒9.5 Shuttering and form-removal in special structures

Special provisions shall be taken to meet this purpose. The related methods shall be described in the job specification.

❒9.6 Shuttering and form–removal for special construction methods

In shuttering and form–removal for special construction methods, in addition to observance of general requirements, special criteria described in detail in the job specification shall also be applied.

9.6.1 Prepacked concrete forms

Forms, in this method, shall be positioned such that they sustain injected grout pressure and other exerted forces, prevent loss of grout and provide for air exit as well, since the grout in prepacked concrete must force out the air around aggregates and replace it.

Extra lateral pressure makes it inevitable to use more skilled labor force, more accurate operational details, and more qualified materials in comparison with the usual concrete formwork.

9.6.2 Sliding forms

Sliding forms are mainly divided into two vertical and horizontal types. Vertical sliding forms are used for vertical structures like silos, installations for storage of materials and substances, resisting cores and shear walls in buildings, piers, chimneys, communication, control and watch towers, protective walls in nuclear and atomic installations, and similar structures.
Horizontal sliding forms are used in projects like tunnel lining, water pipes, sanitary canals, precast elements and members, canal pavement and similar cases.

Form-sliding operations shall be carried out under direct supervision of the personnel experienced especially in this type of formwork.
The tolerances required for execution of the job with the use of sliding forms shall be described in the job specification.

9.6.3 Permanent forms

Permanent forms are those that stay in place permanently, and may even be counted as part of the structure of a building. These forms may be of rigid type like metal decks (polygonal or deformed plates), precast concrete, wood, various types of plastics, and different versions of boards made of fiber, or they may be of flexible type like the deformed, reinforced, water repellant paper or the paper-backed water resistant wire-mesh.

Sufficient care must be taken so that these forms are not subjected to out of range warping, creep, and deformations, under in-construction loads. When rigid forms are used, concentrated live and dead loads exerted on members and elements located between support elements shall be considered in structural calculations.

9.6.4 Formwork for precast members

9.6.4.1 Introduction

This type of forms are used to produce precast load-carrying and non-load- bearing members and elements.

9.6.4.2 Appliances

Use of appliances and fixtures that have good rigidity and quality and can keep reinforcement firmly in place, are of utmost importance in these forms. All openings, bends, embedded parts, special transport hooks and required appliances to connect concrete members that are to be embedded in concrete shall be accurately placed in their specified position and assuredly connected to the form.

Quality and strength of the considered appliances shall be in accordance with plans and specifications annexed to the contract.

9.6.4.3 Tolerances

Tolerances required for precast concrete members and elements are given in table 9-1-4.

9.6.4.4 Removal of forms

Precast concrete members and elements shall be separated from the form only when concrete has gained the specified strength. The time to open the forms is determined from the strength measurement of the test specimen cured in site conditions.

Transport and demoulding methods for precast concrete members and elements shall be approved by the inspection system.

9.6.4.5 Use of precast concrete as a form

Precast concrete plates and members are used as formwork for in-situ placing of concrete to make precast concrete members and elements; they are also used as permanent forms, or as forms which are part of the concrete mass. Precast concrete members as forms may be made of reinforcement free concrete, reinforced concrete or the concrete prestressed in factory or at the site. The most common precast concrete forms are the concrete slabs that function as composite sections after being located in place as a form and being embedded by complement concrete.

9.6.4.6 Design considerations

Where precast concrete form is to function as a composite section together with the covering structural concrete, form plates shall be calculated and designed in accordance with details. For permanent forms that are to be utilized to obtain desired façade, characteristics of surfaces and minimum thickness of desired façade materials shall be specified.

Details of connections shall be such that precast members are prevented from being connected to each other, and also to other in-situ placed elements.

Effective bond between precast form piece and structural concrete is required.

Special transport hooks for concrete forms may be designed and implemented such that it can function as anchor or shear hook. Precast concrete forms, which are to function as combined with in-situ concrete shall be designed and calculated according to the requirements of the second part of this code.

9.6.5 Formwork for underwater concreting

The formwork for underwater concreting shall be designed and calculated with regard to considerations for other types of forms. The difference is that concrete mass is reduced under water by an amount equal to the mass of displaced water, due to archimidos force.

In tidal regions forms shall be designed and calculated for the lowest water level. Changes in operational programs might expose the concrete, planned for buoyancy, to condition changes that force water pressure out of control. Underwater forms shall be built above water surface and in as great as possible pieces they shall then be placed in their specified locations under water.

Application of internal ties in the form, which can lead to distortions in the concreting process shall be avoided as much as possible.

Forms shall be carefully connected to each other and shall be set by the side of materials and previously made elements so that pressured grout and mortar do not penetrate out of the joints. If the form inevitably stands on the path of water current, it must be assured that small cavities inside the form are avoided, as they provide for fine particles of fresh concrete to be washed out.

❒9.7 Conduits and pipes embedded in concrete

9.7.1 Embedding of pipes carrying water, waste water, gas or vapor along the axes of beam and column concrete shall not be allowed except those permitted under clause 9.7.2.

Passing of the pipes mentioned above at right angles to the said axes shall axes shall be avoided as far as possible. In any the area around the pipes and conduits shall be appropriately strengthened.

9.7.2 In dry regions, in building up to 3 stories high, drain pipes may be embedded within columns provided the displaced area of cross-section is considered as empty in the design calculations.

9.7.3 Passing of pipes and conduits through beams and columns of hollow cross-sections may be permitted provided that they can be inspected or replaced with ease.

9.7.4 Embedding of electrical and mechanical services pipes and conduits may be permitted unless prohibited by the provisions of clause 9.7.1 see also clause 9.7.

9.7.5 Conduits and pipes of aluminum shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminum- concrete reaction or electrolytic reactions between aluminum and steel.

9.7.6 Provisions shall be made for holes through formwork for passing of the required mechanical and electrical pipes and conduits through load-bearing walls and slabs so as to avoid breaking of concrete after casting. If absolutely necessary, cutting of holes through concrete may only be permitted using appropriate and approved equipment.

9.7.7 Plastic pipes for passing of from anchors may be embedded in columns, beams and walls provided they are filled with sand and cement mortar after from removal. If the diameter and number of the pipes are such that no concrete section is reduced by more than 3 percent, the holes may be left unfilled.

9.7.8 Conduits and pipes, with their fittings, embedded within a column shall not displace more than 3 percent of the area of cross-section on which strength is calculated or which is required for fire protection. In addition, these pipes and conduits shall be positioned near the longitudinal axis of the column. However, capacity of the concrete member shall not be significantly impaired. If the above conditions are met, effects of the conduits must be included in the column strength calculations.

9.7.9 Except when plans for conduits and pipes are approved by the structural engineer, conduits and pipes embedded within a slab, wall a beam shall satisfy the following:

9.7.9.1 In their outer dimension they shall not be larger than 1/3 the overall thickness lf the member under consideration.

9.7.9.2 They shall not be spaced center to center closer than 3 theirs diameters or widths.

❒9.8 Construction joins

9.8.1 The number of construction joints shall be the minimum number required for carrying out the works.

9.8.2 Location of construction joints shall be selected carefully. Details of typical joints and their location, dependent on importance of the words shall be shown on drawings or shall be determined by the inspecting engineer.

9.8.3 Surface of concrete construction joints shall be cleaned and laitance removed.

9.8.4 Construction joints shall be located where forces, particularly shear forces, are at their lowest value. Provision shall be made for transfer of shear and other forces through construction joints. See clause 12.14.2.5 .

9.8.5 To provide for concrete bond at construction joints, previous concrete surface shall be roughened before the next lift is placed.

9.8.6 All construction joint surfaces shall be surface saturated dried prior to new concreting.

9.8.7 Construction joints shall not be shapeless, but they shall be in the extension perpendicular to the vertical stress extensions. Large construction joints shall be avoided, and required joints shall be considered as stepped or broken surfaces.

9.8.8 Vertical construction joints shall be made using appropriate forms.

9.8.9 Construction joints in floors shall be located within the middle third of spans of slabs, beams and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams. However, requirements of clause 9.8.4 shall take priority.

9.8.10 Beams, girders or slabs supported by columns or walls shall not be cast or erected until concrete in the vertical support members is no longer plastic.

9.8.11 Beams, girders, haunches, drop panels and capitals shall be placed monolithically as part of a slab system, unless otherwise shown in design drawings or specifications.

CHAPTER TEN

PRINCIPLES OF ANALYSIS AND DESIGN

❒ 10.0 Notation

D = dead load.
E = earthquake load.
E_c = modulus of elasticity of concrete, MPa (N/mm^2).
f_c = specified compressive strength of concrete, MPa (N/mm^2).
F = weight and pressure of fluids.
G_c = modulus of rupture of concrete, MPa (N/mm^2).
H = lateral earth pressure and weight of soil.
K = effective length factor for compressive members.
ℓ_n = length of clear span measured face–to–face of support in the direction for which moments are being determined.
ℓ_u = unsupported length (without bearing) in a compression member, mm.
L = live load.
R = redistribution percentage for negative flexural moments.
r = radius of gyration for the compressive member section.
S = load-effect.
S_r = ultimate strength.
S_u = ultimate load-effect.
T = cumulative effect of temperature, creep and shrinkage of concrete and differential settlement of supports.
w_u = ultimate load per unit length or unit area.
W = wind load.

γ_f = partial safety factor of forces.
γ_n = load modification factor.
ρ = tensile reinforcement ratio.
ρ' = compressive reinforcement ratio.
ρ_b = tensile reinforcement ratio at balanced section.
ϕ_c = partial safety factor for concrete strength.
ϕ_m = partial safety factor for materials strength.
ϕ_n = strength modification factor.
ϕ_s = partial safety factor for reinforcement strength.

❒ 10.1 Scope

10.1.1 The provisions of this chapter are associated with general principles that shall be observed in analysis and design of structures. The principles include: design method, loading and loading combinations, recommended safety factors, assumptions related to different methods of structural analysis and general principles used in design of certain members.

❒ 10.2 Design fundamentals

10.2.1 Design objective

10.2.1.1 Design of a structure is meant to specify its configuration, dimensions and specification of components such that the three main objectives stated in 10.2.1.2 through 10.2.1.4 are satisfied.

10.2.1.2 Safety

Safety is meant to organize the entire structure including its components and connections in such a manner that the structure can withhold its integrity and stability and:

One) It does not get damaged under ordinary loads and overloads.
Two) It does not fail or collapse under unusual loads and overloads.

10.2.1.3 Proper performance

Proper performance is meant that structure provides no interventions for the predefined serviceability of the building and:

One) No unusual cracking or deformation leading to damages in non-structural.

Two) members such as finishing or panels are produced under usual loads and overloads.

Three) Settlers do not feel unsafe in case of tremors.

10.2.1.4 Durability

The meaning behind durability is that construction materials preserve their quality characteristic throughout their estimated service life such that safety and serviceability of the structure does not fall under minimum threshold due to age, erosion and corrosion, etc.

10.2.2 Design method

Design method stipulated in this code refers to “design for limit state”. According to this method, the structure is designed in a manner such that under none of the adverse loading conditions and under a specific safety level, it never approaches the special state referred to as ”limit state”.

“Limit states” are states where the structure or parts of it continue their full functioning before approaching any of the above special conditions, in which case the structure is no longer capable of performing its due functions and is therefore not serviceable any more. Limit states are divided into different groups, in accordance with 10.2.2.2 and 10.2.2.3.

10.2.2.2 Ultimate limit states

Ultimate limit states are referred to those related with maximum load-carrying capacity of the structure or part of it. These states may occur as one of the following forms:

One) Loss of balance in structure or in part of it as a rigid body.

Two) Deformation or displacement of the structure or part of it, to the extent where geometric form and the resulting structural behavior loss of balance in structure or in part of it as a rigid is totally changed.

Three) The structure has reached its maximum load-carrying capacity in one of the following forms:

- Collapse of sections, members or their connections due to failure, over-limit deformations or fatigue of the materials used. This state is called ”limit state of strength”
- The structure or part of it approaches a failure mechanism
- Loss of general stability of the structure or part of it

10.2.2.3 Serviceability limit states

Serviceability limit states are referred to those related with structural serviceability or durability requirements. These states may occur in one of the following forms:

One) Over deformation of structure or parts of it such that it adversely affects the profile or proper function of the structure, and incurs damage on the structure, finishing or on non-structural parts.

Two) Local damages like cracking, flaking or over-decomposition of concrete so that extra maintenance would become imminent, leading to increased danger of reinforcement corrosion and consequent adverse effect on the profile and function.

Three) Extra vibration under the effect of service loads, wind force or machinery is such that creates anxiety among settlers concerning structural safety and/or it disturbs structural function of the building.

Four) When the structure is designed for a special unusual function, other limit states appropriate to the special case may appear, which corrupt the structural function. These states are determined according to knowledge and judgement of the Design Engineer.

10.2.2.4 In addition to investigation of limit states in the design of structures, the following considerations shall be observed:

One) Different components of structure and their connections shall be organized so as to provide assurance to its overall stability and integrity. In this regard, it shall especially be assured that the structure does not go through overall failure or through chain failure due to accidental local damages.

Two) Resistance of a structure to fire shall be provided through appropriate special plan. Accordingly, Application of special code requirements is mandatory.

Three) Durability of structure shall be maintained through appropriate special plan. In this regard, operational technical specifications including: quality and minimum content of cement, water quality, water per cement ratio, type and quality of aggregates, maximum content of harmful materials as concrete constituents, mix proportions, concrete casting and placing conditions, curing, lift joint, construction joints and similar aspects shall be met.

Four) Structural design is part of the overall design-construction-maintenance process and it shall be assured that each of the three components is carried out properly. In this respect, type of materials used and their production procedures shall obey specific standards, construction quality shall be assured by proper inspection and service-maintenance regulations shall be observed and controlled by authorities.

10.2.2.5 In this code, control in limit states is observed only for “ultimate limit state of strength” and “serviceability limit state of strength”, in accordance with 10.2.2.6 and 10.2.2.7 and with observance of partial safety factors as described in 10.2.3. No special control is foreseen for other limit states, but general static balance control of the structure as a rigid body, and control towards non-formation of mechanism in the structure is the primary and inherent part of design. No special control is foreseen for unstable limit state, but to incorporate slenderness effects of members under pressure and tension so as to control these members in ultimate limit state of strength, certain special requirements are provided that shall be closely observed.

10.2.2.6 Design for ultimate limit state of strength

Design of different members of the structure, in this limit state, is undertaken such that the ultimate strength or maximum load-carrying capacity of the member in each section is greater than or equal to load-effects in the section subject to ultimate loads acting on the structure. In determining ultimate strength of the section and ultimate loads, safety factors under consideration shall be incorporated in the design. Details of design for this limit state are given in 10.5.

10.2.2.7 Control in limit state of serviceability

These states include limit states of deformation and cracking. Control in this limit stated is performed to ensure that deformation and cracking induced in each member under the effect of service loads acting on the structure is less than the values specified in the design under consideration. Details of control in this limit state are given in 10.6.

10.2.3 Safety factors

10.2.3.1 Safety factors employed in this code are of partial safety factor type, such that for loads, on one side and for concrete and reinforcement strengths on the other, they are considered as described in 10.2.3.2 and 10.2.3.3 respectively. Additionally, in special cases where there is need for more safety in a member, another partial safety factor is incorporated in member strength or in loads and forces, as described in 10.2.3.4.

10.2.3.2 Load magnification factors.

These factors are multiplied by the specified values of loads and forces. Depending on the level of uncertainty to each estimated load quantity, the values of factors differ for each load. Details of these factors are given in 10.5.3. These are referred to as "partial safety factors for forces" and are denoted by γ_f .

10.2.3.3 Strength reduction factors

These factors are multiplied by characteristic strengths of concrete and reinforcement. The factors reflect the level of uncertainty to materials quality, construction process, correctness of dimensions and member sizes. Details of the factors are given in 10.5.2.2. These are referred to as "partial safety factors for materials strength" and are denoted by ϕ_m.

10.2.3.4 Partial safety factor for modification

This factor is employed for cases where the importance of member and its failure consequences, namely significance of failure mechanism such as its ductility or brittleness is to be considered. Depending on the case, the factor is either multiplied by the strength of member to reduce it or by loads and forces to increase them. This is referred to as "modification factor", denoted by ϕ_n or γ_n respectively. Details are given in 10.5.4.

❐ 10.3 Design principles

10.3.1 General

10.3.1.1 Structural analysis is meant to determine load-effects at different sections of the structure, and displacement in different points of the section subjected to forces acting on it, considering its geometric and mechanical specifications. Structural analysis shall account for the most critical cases regarding combined action of possible combination of forces, in accordance with 10.5.3.

10.3.1.2 According to this code, the following methods are permitted for structural analysis:

One) Linear analysis

Two) Linear analysis with limited redistribution

Three) Nonlinear analysis

Four) Plastic analysis

Description of the above methods, their requirements and limitations regarding different structures are given in 10.3.5 through 10.3.8.

10.3.2 Proper models for structures

10.3.2.1 In order to analyze structures, they may be converted into simplified models composed of prismatic elements like beams, columns and arches, planar members like slabs, walls and shells, and three-dimensional members like mass foundations.

10.3.2.2 Prismatic elements

Prismatic elements are referred to those structural members in which, one of the dimensions is considerably larger than the other two, which do not significantly differ from each other. Spacing between two sections with zero flexural moments shall be at least twice the member depth.

10.3.2.3 Planar members

Planar members in the structure are referred to those in which, one of the dimensions (thickness) is considerably smaller than the other two. These members including slabs, walls and deep beams are described as follows:

One) Slabs

are planar members that are mostly under the effect of loads acting orthogonal to their middle plates. Forces acting on the middle plates are negligible in comparison with flexure and tension. Planar members are considered as slabs only when spacing between sections with zero curvature subject to distributed loads is at least four times their thickness.

Two) Walls

are planar members that are mostly subject to loads acting on their middle plates. Walls usually have continuous supports.

Three) Deep beams

are wall-shaped members with discontinuous supports. In addition to forces acting on middle plates, these members are under the effect of flexure and shear. Refer to 11.7 for more information.

Four) Shells

are planar members that are mostly subject to forces acting orthogonal to their middle plates. Due to geometric shape, the forces acting on middle plates are significant in comparison with flexure and shear.

10.3.2.4 Three-dimensional members

Three-dimensional structural members are those in which none of the dimensions differ significantly with the other two.

10.3.3.1 Materials specifications

10.3.3.1 Modulus of elasticity of concrete in linear analysis is calculated by Eq. (10.1):

$$E_c = 5000\sqrt{f_c} \qquad (10\text{-}1)$$

For nonlinear analysis in which favorable shapes of stress-strain curve are used, early modulus of elasticity of concrete is taken equal to the above value.

10.3.3.2 Modulus of elasticity of reinforcement in linear analysis is assumed to be 200,000 MPa (N/mm^2). For nonlinear analysis in which favorable stress-strain curve characterizing reinforcement is used, early modulus of elasticity for reinforcement is taken equal to the above value.

10.3.3.3 Heat expansion factor for concrete is taken equal to 10×10^{-6} per degree Celsius.

10.3.4 Geometric specifications

10.3.4.1 Effective span length in different structural members are determined according to the following:

One) Effective span length of members not built integrally with supports shall be considered the clear span plus depth of the members or distance between centers of supports, whichever is smaller.

Two) For members built integrally with supports, effective span length shall be taken as the distance center-to-center of supports. In members having support length to effective depth ratio greater than 2, the extra length of the member, relative to its effective depth, that lays on the support may be assumed rigid.

Three) For fully restrained cantilever members, effective span length may be taken equal to clear span length.

Four) Solid and ribbed slabs with clear span less than or equal to 3 meters, cast integrally with their supports may be considered as continuous slabs on simple supports having effective span length equal to their clear spans.

10.3.4.2 Flexural and torsion stiffness of structural members, in structural analysis, may be calculated based on the uncracked section regardless the reinforcement, or based on the cracked section considering reinforcement.

In order to incorporate the effect of cracking on flexural and torsional stiffness, beam and column stiffness may be taken as 0.35 and 0.70 of that of their uncracked section stiffness, respectively. Wall stiffness may be taken as 0.35 and 0.70 depending on their cracked and uncracked sections stiffness, respectively. The assumptions applied shall be consistent throughout analysis. Geometric changes of sections corresponding to the effects of haunches shall also be incorporated.

10.3.5 Linear analysis

10.3.5.1 In this analysis method, all load-effects at different sections of the structure are determined based on the assumption that materials behavior is linear and that deformations induced are small and also based on the theory of elasticity.

10.3.5.2 This analysis method may be used in various structures in ultimate and serviceability limit states in structures composed of prismatic elements not braced against side-sway, this method is applied only if column slenderness factor, $\mathbf{K}\frac{\ell_u}{\mathbf{r}}$, is not greater than 100.

10.3.5.3 in flexural frames in which special constraints are observed, structural analysis may be carried out using approximate methods. The methods and constraints are described in 10.3.9.

10.3.6 Linear analysis associated with limited redistribution

10.3.6.1 Assumptions adopted in this analysis method are exactly as those in linear analysis method. In addition, it shall be permitted to increase or decrease load-effects at sections to a limited extent, depending on their mechanical specifications. Effects of these load-effect changes shall also be applied for other sections.

10.3.6.2 This analysis method may be applied, in ultimate limit states, to structures composed of prismatic elements and planar elements, in accordance with 10.3.6.3 and 10.3.6.4.

10.3.6.3 In flexural frames with the following conditions, redistribution of flexural moments may be carried out to the extent of pre-defined value:

a) Negative flexural moments values, calculated at supports shall be permitted to increase or decrease by not more than:

$$R = 20(1 - 0.7\frac{\rho - \rho'}{\rho_b})\% \qquad (10\text{-}2)$$

Provided that moment values at other sections are changed with regard to load balance conditions. Redistribution of moments shall be made only when the ρ or ρ - ρ' at the considered section is less than $0.7\rho_b$.

b) Redistribution of moments in flexural members not braced against sidesway is not permitted if column slenderness factor is greater than 25. When the factor is less than 25, maximum redistribution shall be limited to:

$$R = 10(1 - 0.7\frac{\rho - \rho'}{\rho_b})\% \qquad (10\text{-}3)$$

c) When approximate methods stated in 10.3.9 are used to determine load-effects, redistribution of moments is not permitted.

d) Redistribution of moments due to wind and earthquake forces is not permitted.

10.3.6.4 For two-way continuous slabs, calculated flexural moments at supports, in each strip, may be increased or decreased by maximum 25 percent, provided that values of flexural moments at other sections of that strip are varied based on the equilibrium conditions of loads. Redistribution of moments in slab systems where their moments are determined using direct method stated in 7.15, is not permitted.

10.3.7 Nonlinear analysis

10.3.7.1 According to this analysis method, all load-effects are determined based on nonlinear behavior of materials and the effect of large deformations induced in the structure, known as “geometric nonlinear behavior”.

10.3.7.2 This analysis method may be used for structures composed of prismatic elements and plate elements, in ultimate or serviceability limit states.

10.3.7.3 In this analysis method, the elasto-plastic bilinear diagram reflecting cracked state of concrete and state of plastic hinge formation, or trilinear diagram reflecting uncracked and cracked states of concrete and state of plastic hinge formation, or any other diagram confirmed by tests may be used to represent the flexural-curvature moment diagram of the member.

10.3.8 Plastic analysis

10.3.8.1 In this analysis method, all load-effects are determined assuming plastic-rigid behavior for member and using theory of plasticity.

10.3.8.2 This analysis method may be applied to structures composed of planar members, in ultimate limit state only.

10.3.8.3 For slabs, this method may be applied as static method (like strip method), and as cinematic method (like failure hinges method). Bar Placement inside slabs, for each of the methods, shall be made such that complete assurance relative to rotation capacity of hinges is achieved. For static method, distribution function for selected flexural moments shall be as close as possible to that obtained from linear analysis method.

10.3.8.4 Static method is the only method permitted to be applied for shells. In static method, distribution function for selected flexural members shall be as close as possible to that obtained from linear analysis method. Previous experience and findings of tests carried out shall be taken as the basis to select distribution function.

10.3.9 Approximate methods for linear analysis

10.3.9.1 For continuous beams and one-way slabs, flexural moments and shear load-effects at different sections may be determined from table 10.3.9.1, provided that the following conditions exist:

a) There are at least two spans in beam or slab.

b) The larger of two adjacent spans is not greater than the shorter by more than 20 percent.
c) Loads are almost uniformly distributed along beam or slab
d) Live load intensity does not exceed three times that of dead load.
e) All members have fixed sections.

10.3.9.2 For analysis of multistory frames in usual structures under gravity loads, frames may be model into smaller and separate sub frames to be investigated individually, if lateral displacements are not significant.
Each of the sub-frames includes all beams of one story plus all the columns of the upper and the lower story. Far ends of columns are to be considered as fixed at the locus of connection to adjacent stories. In cases where column connection to adjacent story is specifically hinged, the connection is assumed to be of hinge type. For design of columns in each of the sub frames, axial loads transmitted from upper stories shall be incorporated.

10.3.9.3 For analysis of multistory frames in usual structures under gravity loads, the method stated in 10.3.9.2 may be applied if lateral displacements are significant and provided that column ends at the locus of connection to are adjacent story is fixed but at connection to the other adjacent story is considered restrained but able to sway.

10.3.9.4 In analysis of multistory frames for lateral loads, approximate (or simplified) methods such as "the portal method" may only be permitted when the position of inflection points can be identified with good approximation and that the effect of column axial deformations are incorporated.

10.3.9.5 In usual buildings with maximum eight stories, the method described in 10.3.9.4 may be applied and the effect of column axial deformation is neglected.

Table 10.3.9.1 Approximate values of flexural moments and shears in continuous beams and one-way slabs.

Positive moment: End spans	
With discontinued end as simple (unrestrained)	$w_u \frac{\ell_n^2}{11}$
With discontinued end as integral with support	$w_u \frac{\ell_n^2}{14}$
Interior spans	$w_u \frac{\ell_n^2}{16}$
Negative moments at exterior face of first interior support:	
Two spans	$w_u \frac{\ell_n^2}{9}$
More than two spans	$w_u \frac{\ell_n^2}{10}$
Negative moment at other faces of internal supports:	$w_u \frac{\ell_n^2}{11}$
Negative moment at other faces of internal supports for: Slabs with spans not exceeding 3m; and beams where ratio of sum of column stiffness to beam stiffness exceeds eight at each end of the span	$w_u \frac{\ell_n^2}{12}$
Negative moment at interior face of all exterior supports for members built integrally with supports:	
Where support is spandrel beam	$w_u \frac{\ell_n^2}{24}$
Where support is a column	$w_u \frac{\ell_n^2}{16}$
Shear in end members at face of first interior support	$1.15 w_u \frac{\ell_n}{2}$
Shear at face of all supports	$w_u \frac{\ell_n}{2}$

❒ 10.4 Loading

10.4.1 Loads that shall be considered in the design include the following loads or forces:

a) Direct forces such as dead loads, live loads, earth and fluids pressure, crane pressure and wind effect.
b) Indirect force such as earthquake , vibrations, temperature changes, concrete shrinkage and support settlement.
c) Under-construction actions such as form weight and concrete placement in one story over lower floor (s).

10.4.2 All loads acting on structure, except for loads due to earthquake, shall be calculated according to the Iranian standards No.519 titled as "Minimum Design Load on Ordinary Buildings and Structures."

10.4.3 Earthquake loads shall be determined according to the Iranian standard No.2800 titled "Iranian Standard Code for Design of Buildings against Earthquake."

10.4.4 Different forces are incorporated in loading considering the possibility of their concomitance and their combination. In combining forces, the most critical probable loading case shall be applied in accordance with 10.5.3.

10.4.5 Loads and forces acting on the structure, when magnified by partial safety factor for strength in serviceability limit state, are called "Service Loads", but when magnified by partial safety factor for serviceability in ultimate limit state, they are called "Ultimate Loads". Details of these factors are given in 10.6.2 and 10.6.3.

10.4.6 In determining the maximum load-effect at different sections of continuous beams and frames, the action point of dead and live loads shall be considered in accordance with 10.4.6.1 and 10.4.6.2.

10.4.6.1 For determining maximum load-effect in beams, the following two conditions shall be considered:

a) Dead load on all spans and live load on two adjacent spans.
b) Dead load on all spans and live load on alternate spans.

10.4.6.2 In determining maximum load-effect in columns, the following two conditions shall be considered:

a) Dead load on all spans and live load on the larger span adjacent to column under consideration and only on the story under investigation.
b) Live loading condition giving the maximum ratio of flexural moment to axial load in column.

❐ 10.5 Design in ultimate limit state of strength

10.5.1 Design of different structural sections in ultimate limit state of strength, and for each special load-effect shall be done based on:

$$S_u \leq S_r \qquad (10\text{-}4)$$

Where S_r is the ultimate strength of member at section considered and S_u is the load-effect in the section subject to ultimate loads. Values of S_r and S_u shall be determined in accordance with 10.5.2 and 10.5.3.

10.5.2 Required strength of section, S_r

10.5.2.1 Required strength of section, S_r, for any special load-effect shall be determined in accordance with geometric specifications of section, mechanical behavior of the member subject to that type of load-effect and incorporating equilibrium conditions of forces at the section and compatibility of deformation in its various fibers.

10.5.2.2 In determining required strength of section, partial safety factor for the specified strengths of reinforcement and concrete shall be incorporated:

a) Partial safety factor for concrete strength.

$$\phi_c = 0.6 \qquad (10\text{-}5)$$

b) Partial safety factor for reinforcement strength.

$$\phi_s = 0.85 \qquad (10\text{-}6)$$

10.5.2.3 In determining required strength of section, special regulation given in different chapters of this code shall be observed as follows.

a) For members subject to flexure, flexure and compression, flexure and tension: chapter 11.
b) For members subject to shear and torsion: chapter 12.
c) For buckling and effects of slenderness in members under compression and flexure: chapter 13.
d) For bond and anchorage: chapter 18.

10.5.3 Ultimate load-effects, S_u

10.5.3.1 Ultimate load-effects, S_u at different sections of structure including flexural moments, axial forces, shear forces and torsional moments shall be determined based on analysis of structure subject to ultimate loads and their combinations. In this regard, the most critical loading conditions and their combinations shall be considered.

10.5.3.2 Partial safety factor for loads and other forces, $\gamma_{r,}$ that shall be applied to determine ultimate loads, and different combinations of these loads that shall be incorporated to determine ultimate load-effects, shall be considered in accordance with 10.5.3.3 trough 10.5.3.8.

10.5.3.3 Load-effects due to dead loads (D), live loads (L) shall be determined according to:

$$S_u = S\,(1.25D + 1.5L) \qquad (10\text{-}7)$$

When the effect of live loads reduces S_u, this effect is taken to be equal to zero.

10.5.3.4 Load-effects due to dead loads (D) live loads (L) and earthquake (E) in addition to equation No. 10.7, shall be determined according to the following formulas:

$$S_u = S\,(D + 1.2L + 1.2E) \qquad (10\text{-}8)$$

$$S_u = S(0.85D + 1.2E) \qquad (10\text{-}9)$$

In the Eq. (10-8), when the effects of live loads decrements S_u, it is taken equal to zero.

10.5.3.5 Load-effects due to dead loads (D) and live loads (L) and wind (W) is determined exactly according to the relation 10.5.3.4 in which wind load substitutes for earthquake load.

10.5.3.6 For determining load-effects due to dead loads (D), Live loads (L), earth pressure or underground water pressure (H), in addition to Eq. (10-9), the following relations are used:

$$S_u = S\,(1.25D + 1.5L + 1.5H) \qquad (10\text{-}10)$$

$$S_u = S\,(0.85D + 1.5H) \qquad (10\text{-}11)$$

In the Eq. (10-10), if the effect of live loads is unfavorable and reducing, this effect is taken equal to zero.

10.5.3.7 Load-effects due to dead loads (D), live loads (L) and weight or pressure of fluids with well-defined densities and controllable maximum heights shall be in accordance with 10.5.3.6 in which, load F is substituted for load H and its coefficient is taken to be equal to 1.25 instead of 1.5.

10.5.3.8 In determining load-effects due to dead loads (D), live loads (L) and cumulative effects of temperature, creep and shrinkage of concrete and settlement of supports (T), the following relations, in addition to relation no. 10.7, shall be used:

$$S_u = S\,(D + 1.2L + T) \qquad (10\text{-}12)$$

$$S_u = S\,(1.25D + 1.25T) \qquad (10\text{-}13)$$

10.5.3.9 Effect of impact and vibration in different combinations of loads shall be considered as similar to live load.

10.5.4 Application of modification factor

10.5.4.1 Where a member of structure, due to special reasons, shall have greater safety margin, and application of modification factor ϕ_n or γ_n deems necessary, control of different sections of member against any special load-effect shall be done in accordance with one of the following:

$$S_u \leq \phi_n S_r \qquad (10\text{-}14\text{-a})$$

$$\gamma_n S_u \leq S_r \qquad (10\text{-}14\text{-b})$$

In the above relations, parameters S_r and S_u are determined according to 10.5.1 and coefficient ϕ_n or γ_n is adopted in accordance with 10.5.4.2.

10.5.4.2 Modification factor ϕ_n or γ_n is equal to unity for all members, unless its value like ϕ in 13.8.2 is a specific value.

❐ 10.6 Control in limit state of serviceability

10.6.1 Control of different members of structure in two limit states of deformations and cracking shall be done based on limited deformations induced in the member or the extent of crack opening at the section, subject to service loads and according to the values specified in chapter 14.

10.6.2 In order to determine service loads, partial safety factors for loads, γ_f, shall be taken equal to unity.

10.6.3 In determining load-effects inside concrete and steel, partial safety factor for strength ϕ_n shall be taken equal to unity.

10.6.4 In determining the values of deformations induced in the member, and also the extent of crack opening at the section, requirements of chapter 14 shall be observed.

❐ 10.7 General principles for design of sections

10.7.1 For members cast integrally with their supports, flexural moments at the sections over supports may be taken equivalent to the value of this moment at section on face of support; if not, flexural moment at end section of effective span shall be taken as basis of design. Refer to 10.3.4.1 for definition of effective span.

10.7.2 In members with varying cross section, section variations shall be incorporated in the design.

10.7.3 In design for wind and earthquake lateral loads, the strength of integrated structural components shall be considered only.

10.7.4 T-shape beams

10.7.4.1 For T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

10.7.4.2 Width of slab effective as T-beam flange shall not exceed one quarter of clear span length of the beam for continuous beams, and two-fifth of clear span length of beam for simple beams. Effective overhanging flange width on each side of the web shall not exceed the following two values:

a) Eight times the slab thickness.
b) One–half the clear distance to the next web.

10.7.4.3 For beams with a slab on one side only, the effective overhanging flange width shall not exceed three times the following values:

a) One-twelfth the clear span length of the beam.

b) Six times the slab thickness.
c) One-half the clear distance to the next web.

10.7.4.4 For isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange think ness not less than one-half the width of web and on effective flange width not more than four times the width of web.

10.7.45 Where primary flexural reinforcement in a slab that is considered as a T-beam flange is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following requirements concrete joist system that obeys regulations of $10-7-5$ is excluded:

a) Transverse reinforcement perpendicular to the beam shall be designed to carry ultimate loads on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.
b) Transverse reinforcement shall be spaced not farther apart than five times the slab thickness, nor farther apart than 350 mm.

10.7.5 Criteria for concrete joist system

10.7.5.1 Concrete joist system which is composed of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions and in which the following limitations are observed, may be designed as whole, in accordance with requirements for slabs:

a) Ribs shall be not loss than 100 mm in width, and shall have a depth of not more than 3-1/2 times the minimum width of rib
b) Clear spacing between ribs shall not exceed 750 mm

10.7.5.2 Concrete joist construction not complying with regulations of 10.7.5.1 shall be designed as slabs and beams.

10.7.5.3 In systems where burned clay or concrete block permanent fillers, of materials having a unit compressive strength at least equal to that of the specified strength of concrete in the joist, are used in between ribs, shear and negative flexural moment strengths shall be computed using strength of vertical shells of fibers in contact with the ribs strength of other fillers shall be neglected in system strength computations, the following limitations shall be observed in these systems:

a) Slab thickness over permanent fillers shall be not less than one-twelfth the clear distance between ribs, nor less than 40 mm.
b) In one–way joist systems, reinforcement normal to the direction of ribs shall be permitted in the top slab as per regulation of 7.8. In two-way joist, reinforcement in two orthogonal directions and in accordance with requirements of 7.8 shall be provided.

10.7.5.4 In systems where removable forms are used and in fillers not complying with requirements of 10.7.5.3 the following limitations shall apply:

a) Slab thickness shall not be less than one-twelfth the clear distance between ribs, nor less than 50 mm
b) Reinforcement normal to the direction of ribs shall be provided in the top slab based on regulations for flexure and considering load concentrations, if any. Such reinforcement quantity shall be not less than that specified in 7.8.

10.7.5.5 Shear strength provided by the concrete in between ribs shall be permitted to be 10 percent more than that specified in chapter 12. Shear strength of ribs may be increased by using shear reinforcement or by widening rib bottoms.

10.7.5.6 Where conduits or pipes are embedded within the slab, as described in 7.9, slab thickness shall be at least 25 mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not significantly reduce the strength of the construction.

CHAPTER ELEVEN

FLEXURE AND AXIAL LOADS

❒ 11.0 Notation

A_c = area of core of spirally reinforced compress member measured to outside diameter of spiral, mm^2.

A_g = gross area of section , mm^2.

A_{st} = total area of longitudinal reinforcement, mm^2.

A_1 = loaded area, mm^2.

A_2 = support area, according to definitions 11.10.2 and 10.11.3.

b = width of compression face of member, mm.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm.

E_s = modulus of elasticity of reinforcement, MPa (N/mm^2).

f_c = specified compressive strength of concrete, MPa (N/mm^2).

f_y = specified yield strength of reinforcement (f_{yk}). MPa (N/mm^2).
(for simplicity, subscript k has been omitted in this notation).

M_r = required flexural moment.

M_u = ultimate flexural moment.

N_b = ultimate axial load strength corresponding to balanced section, N.

N_{rmax} = maximum axial load strength, N.

N_r = ultimate axial load strength, N.

N_u = ultimate axial load, N.

X = distance between neutral fiber to the extreme compression fiber at section, mm.

β_1= depth of equivalent tension rectangle (11.3.6).

ε_s = reinforcement strain.

ρ = tension reinforcement ration ($\rho = \frac{A_s}{bd}$).

ρ_b= tension reinforcement ratio at balanced section.

ρ_{min} = minimum ratio oftension reinforcement.

ρ_s = ratio of volume of spiral reinforcement to total volume of core (out-to-out of spirals) of spirally reinforced compression member.

ϕ_c= partial safety factor for concrete.

ϕ_s = partial safety factor for reinforcement.

❒ 11.1 Scope

11.1.1 Provisions of chapter shall apply for design of members subject to flexure or axial or their combination, in ultimate limit state of strength.

11.1.2 In strength design of members for flexure and compressive axial forces, slenderness effects shall be considered.
Effects due to slenderness of members and the way the effect on design is described in chapter 13.

11.1.3 In designing members for flexure or for combination of flexure and axial forces, in ultimate limit state of strength, observance of requirements related to integrity, and assurance about complete transmission of forces is required. Principles of chapter 18 shall therefore be applied.

11.1.4 In designing members for combined effects of flexure and axial forces in ultimate limit state of serviceability, no special requirements need be observed, but for members under flexure in limit state of serviceability, adoption of special requirements for deformation and cracking is mandatory. These requirements are described in chapter 14.

❒ 11.2 Ultimate limit state of strength, under flexure and axial forces

11.2.1 Ultimate limit state of strength at sections subject to flexure or axial forces or their combined effect is provided on the basis of the following relations:

$$M_u \leq M_r \qquad (11\text{-}1)$$

$$N_u \leq N_r \qquad (11\text{-}2)$$

In the above relations, M_u and N_u represent ultimate flexural moment and ultimate axial load at section under consideration, respectively: M_r and N_r represent the resisting ultimate moment at section under flexure and the resisting ultimate axial load, respectively. M_r and N_r are determined considering their mutual effect on each other.

11.2.2 Members subject to flexure and axial load shall be designed such that they have adequate strength against combination of axial loads and flexural moments to corresponding axial load.

11.2.3 The resisting ultimate flexural moment, M_r, and the resisting ultimate axial load, N_r, are determined from analysis of section according to assumptions given in 11.3 and incorporating balanced condition of forces and geometrical compatibility of strains at the section.

❒ 11.3 Design Assumptions

11.3.1 Strain distribution for reinforcement and concrete at the depth of each section is considered linear, except for very deep flexural members. Regarding such sections, refer to 11.7.

11.3.2 Maximum usable strain at extreme concrete compression fiber shall be assumed equal 0.003 to 0.0035.

11.3.3 stress in reinforcement for strains below the corresponding value $\phi_s f_y$ shall be equal to $E_s \varepsilon_s$ and for strains greater than the corresponding value $\phi_s f_y$ shall be considered independent of strain and equal to $\phi_s f_y$

11.3.4 In flexural calculation of reinforced concrete members, tensile strength of concrete is neglected.

11.3.5 The relationship between concrete compressive stress and corresponding concrete strain shall be assumed to be rectangular, trapezoidal parabolic or any other shape that results in prediction of strength in substantial arrangement with results of comprehensive tests.

11.3.6 Requirements of 11.3.5 are satisfied through adoption of maximum concrete strain at extreme compression fiber equal to 0.003 and by on equivalent rectangular concrete stress distribution defined by the following characteristics:

a) An stress equal to $0.85\phi_c f_c$ shall be assumed uniformly distributed over on equivalent compression zone bounded by edges of the cross section, and a straight line located parallel to the neutral axis at a distance $\beta_1 x$ from extreme compression fiber.
b) Distance from the fiber of maximum strain to the neutral axis shall be measured in a direction normal to that axis

c) Factor β_1 is equal to 0.85 for specified compressive strength of concrete up to 30 MPa (N/mm^2). For greater strength, β_1 is reduced linearly at rate 0.008 for each MPa (N/mm^2) in crease in specified compressive strength of concrete. Minimum value of β_1 is limited to 0.65.

❒ 11.4 General design principles

11.4.1 Balanced section is the one in which tensile reinforcement strain, in ultimate limit state of strength, reaches its corresponding *fy*, and simultaneously compressive concrete strain reaches its assumed ultimate value given in 11.3.2 or 11.3.6.

11.4.2 in order to provide adequate strength for flexural members, compressive reinforcement is used conjunction with tensile reinforcement.

11.4.3 For members subject to axial compression, maximum value of the resisting ultimate axial load shall not exceed 80 percent of the value obtained based on assumptions in 11.3. If assumptions in 11.3.6 applied, this load shall be equal to the following value:

$$N_{rmax} = 0.8\left[0.85\,\phi_c\, f_c\,(A_g - A_{st}) + \phi_s\, f_y\, A_{st}\right] \qquad (11\text{-}3)$$

11.4.4 For member subject to compression and flexure, the resisting ultimate axial load shall not be taken greater than the value obtained from 11.4.3.

❒ 11.5 Reinforcement limitation in flexural members

11.5.1 Maximum tensile reinforcement

For flexural members or for flexural-compressive members in which, axial load is less than both values $0.15\phi_c f_c A_g$ and N_b, ratio of tensile reinforcement, ρ, shall not exceed ρ_b. ρ_b is the ratio of tensile reinforcement at balanced section subject to flexure without axial load.

11.5.2 Minimum tensile strength

11.5.2.1 At every section of a flexural member (except when stated in 11.5.2.2 and 11.5.2.3) where tensile reinforcement Is required by analysis and calculations, the ratio of reinforcement used, ρ, shell not be less the greater OF following values:

$$\rho_{\min} = \frac{1.4}{f_y} \quad , \quad \rho_{\min} = \frac{0.25\sqrt{f_c}}{f_y} \qquad (11\text{-}4)$$

For T-section beams and joists with flange in tension, ρ is calculated with regard to web width.

11.5.2.2 The requirements of 11.5.2.1 may be neglected, if the area of tensile reinforcement resulting from calculations is taken at least 1.33 times the calculated section.

11.5.2.3 For slabs walls and footings, minimum reinforcement and maximum space between them is determined respectively according to the chapters 15,16 and 17.

11.5.3 Distribution of flexural reinforcement

11.5.3.1 Distribution of flexural reinforcement regarding cracking of beams and one-way slabs is done according to rules in chapter 14.

11.5.3.2 Distribution of flexural reinforcement in two-way slabs is done according to rules of 5.15.

❒ 11.6 Distance between lateral supports of flexural members

11.6.1 Except where structural durability analysis including torsional effects are carried out, supports for beams shell be in accordance with requirements of 11.6.2.

11.6.2 Spacing of lateral supports for simple or continuous beams shall not exceed 50 times width of compression flange or face.

❒ 11.7 Deep flexural members or deep beams

11.7.1 Flexural members with overall depth-to-clear span ratios greater than 2/5 for continuous span, or greater than 4/5 for simple spans, shall be considered as deep flexural members taking into account nonlinear distribution of strains at section depths, increase in lateral anchorage length and lateral buckling.

11.7.2 Shear strength in flexural deep beams shall be determined in accordance with 12.15.

11.7.3 Minimum flexural tension reinforcement shall conform to 11.5.2.

11.7.4 Minimum horizontal and vertical reinforcement in the side faces of flexural deep beams shall be greatest of the values in 12.15 or 16.4.

❒ 11.8 Design dimensions for compression members

11.8.1 Once structural analysis is performed load-effects against stiffness of real section of members are known, requirements, of 11.8.2 to 11.8.4 may be applied to design parts and to compute compressive reinforcement area.

11.8.2 Outer limits of the effective cross section of a spirally reinforced to tied reinforced compression member built monolithically with a concrete wall or pier shall be assumed not greater than 40 mm outside the spiral or tie reinforcement

11.8.3 As an alternative to using the full gross are for design for design of a compression member with a square or octagonal, or other shaped cross section, an equivalent circular section with a diameter equal to the least lateral dimension of the actual shape may be used. Total cross section area considered, required percentage of reinforcement, and design strength of member shall be determined based on the circular section.

11.8.4 Determination of design strength of section and the minimum required reinforcement for a compression member with across section larger than needed by considerations of loading, shall be based on a reduced effective area met less than one-half the total section area.

❐11.9 Reinforcement limitations in compression members (columns)

11.9.1 Area of longitudinal reinforcement for compression members shall be not less than 0.008, nor greater than 0.08 times gross section area. Limitation for maximum area of reinforcement shall also be observed at lap splices of reinforcement.

11.9.2 Minimum number of longitudinal bars in compression members is as following:

a) Within circular or rectangular ties , 4 bars.
b) Within triangular ties, 3 bars.
c) Enclosed inside spirals conforming to 11.9.3, 6 bars.

11.9.3 Volumetric ratio of spiral reinforcement to gross core volume, p_s, shall be not less than the value give by the following relation:

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c}{f_y} \qquad (11\text{-}5)$$

Where f_y (for spiral) shall be not greater than 400MPa (N/mm^2).

❒ 11.10 Bearing strength

11.10.1 Ultimate bearing strength on concrete shall not exceed $0.85\phi_c f_c\ A_l$, except for requirements of 11.10.2 and 11.10.3.

11.10.2 When the supporting surface is wider on all sides than the loaded area, ultimate bearing strength on the loaded area, calculated according to requirements of 11.10.1, may be multiplied by the factor $\sqrt{\frac{A_2}{A_1}}$. the factor shall not be greater than 2. A_2 is the supporting area.

11.10.3 When the supporting surface is sloped or steppe, A_2 shall be taken as the area of the lower base of a cone or pyramid plane or stepped edges contained wholly within the support. Upper base is considered equal to A_1 and the slope of lateral face 1:2 (1 vertical to 2 horizontal).

CHAPTER TWELVE

SHEAR AND TORSION

❒ 12.0 Notation

a = distance between effect point of force (concentrated load) and face of support – shear span.

A_c= area enclosed by outside perimeter concrete cross section including hole area (if any), mm^2.

A_{cv}= area of concrete section resisting shear transfer, mm^2.

A_f = area of flexure reinforcement in bracket corbel (12.16.2.5), mm^2.

A_g = gross area of section, mm^2.

A_h = area of shear reinforcement parallel to flexural tension reinforcement, mm^2.

A_ℓ = total area of longitudinal reinforcement to resist torsion, mm^2.

A_n = area of reinforcement in bracket or corbel resisting tensile force (12.16.2.6), mm^2.

A_o = gross area enclosed by shear flow path due to torsion in section, including punching area (if any), mm^2.

A_{oh} = gross area enclosed by shear flow path due to torsion of section, including gross area by centerline of the outermost closed transverse torsional stirrup, mm^2.

A_s = section area of tensile reinforcement, mm^2.

A_t = area of one leg of a closed stirrup resisting torsion within a distance S, mm^2.

A_v = area of shear reinforcement within a distance S, or area of shear reinforcement perpendicular to flexural tension reinforcement, mm^2.

A_{vf} = area of shear-friction reinforcement, mm^2.

A_{vh}= area of shear reinforcement parallel to flexural-torsional reinforcement within distance S_2, mm^2.

b_t= width of that portion of cross section containing the closed torsion resistant stirrup, mm.

b_w= web width, or diameter of circular section, mm

b_o = perimeter of critical section for slabs and footing, (12.18.2.1.b), mm.

b_{om}= perimeter of critical section for slabs with shearhead (12.18.2.7), mm.

c_1= size of rectangular or equivalent rectangular column, capital or drop panel measured in the direction of the span for which moments are being calculated, mm.

c_2= size of rectangular or equivalent rectangular column, capital or drop panel measured in the direction normal to the span for which moments are being calculated, mm.

d= distance from extreme compression fiber to control of longitudinal tension reinforcement, mm.

f_c= specified compressive strength of concrete, MPa.

f_y= specified yield strength of reinforcement (f_{yk}), MPa (N/mm^2). For simplicity, letter K has beam omitted from subscript, in this notation.

h= overall thickness of member,

h_b = distance between lower face of minor beam and lower face.

h_v= total depth of shearhead, mm.

h_w= total height of wall from base to top, mm.

h_l= total depth of main beam, (12.17.3), mm.

ℓ_n = clear span length-distance from face-to-face of supports, mm.

ℓ_v = minimum length of each shearhead from cent of concentrated load, mm.

ℓ_w = horizontal length of wall, mm.

M_m= modified moment, N-mm.

M_p= plastic moment of shearhead cross section, N.mm.

M_r = ultimate flexural strength of the section, N.mm.

M_u= ultimate flexural moment at section N.mm.

M_{uf}= fraction of unbalanced moment transferred by flexure, N.mm. Refer to 12.18.5.1.

M_{uv}= fraction of unbalanced moment transferred by shear, N.mm. Refer to 12.18.5.1.

M_v= additional flexural strength of each column strip induced in the slab due to shearhead, N.mm.

N_r= ultimate tensile strength of section, N.

N_u= ultimate axial load acting on cross section simultaneously with Vu, which is taken as positive for compression, negative for tension. This force includes cumulative effects of shrinkage and creep, N.

p_c = outside perimeter of concrete cross-section, mm.

p_h = perimeter of the area confined b centerline of outermost closed tersion stirrup at section, mm.

s = spacing between shear or torsion reinforcement layers is the direction parallel to longitudinal reinforcement, mm.

s_1= spacing of vertical reinforcement in wall, mm.

s_2 = spacing between shear or torsion reinforcement layers in the direction perpendicular to longitudinal reinforcement- or spacing of horizontal reinforcement in wall, mm.

T_{cr} = ultimate torsional strength against cracking, N.mm.

T_r = ultimate torsional strength, N-mm.

T_s = ultimate torsional strength provided by torsion reinforcement, N.mm.

T_u = ultimate torsional moment at section, N.mm.

v_c = shear strength of concrete (relation 12.4), MPa.

V_e = ultimate shear strength provided by concrete, N.

V_r = ultimate shear strength of section, N.

V_s = ultimate shear strength provided by shear reinforcement, N.

V_u= ultimate shear force at section, N.

α = angle between inclined stirrups and horizontal axis of member.

α_f = angle between shear–friction reinforcement and shear plane.

α_s = constant factor used to compute in slabs.

α_v = ratio flexural stiffness of shearhead or in to flexural stiffness of cracked section of the composite slab surrounding the arm (12.18.3.4.c).

β_c = ratio of length to width of surface area under concentrated load or limited support area

η= number of identical arms of shearhead.

μ= coefficient of friction.

ρ_h= ratio of horizontal shear reinforcement area to gross concrete area of vertical section.

ρ_n= ratio of vertical shear reinforcement area to gross concrete area of horizontal section.

$\rho_w = \dfrac{A_s}{b_w d}$

ϕ_c= partial safety factor for concrete.

ϕ_s= partial safety factor for reinforcement.

❒ 12.1 scope

12.1.1 Provisions of this chapter shall be observed for design of members subject to shear or torsion, or this combined effects in ultimate limit states of strength.

12.1.2 No special provisions regarding design of members subject to shear or torsion or their combined effects in ultimate limit slate is proposed. Favorable behavior of such members is assured by the state of cracking induced due to shear or torsion while observing limitations of shear and torsion reinforcement in accordance with provisions of this chapter and also limitations regarding reinforcement anchorage according to provisions of chapter 18.

12.1.3 Requirements for design of members in shear have been given in 12.2 to 12.7, for torsion in 12.13. Other parts of this chapter concerns design of deep flexural members, brackets and corbels, walls, slabs and footings and shear connections.

❐ 12.2 Ultimate limit state of strength in shear

Limit state of strength in shear, shall be based on the following relation:

$$V_u \le V_r \qquad (12\text{-}1)$$

Where V_u is the ultimate shear force at section under considered, obtained from design of structure under ultimate load, and V_r is ultimate (or nominal) shear strength at section computed in accordance with 12.2.2.

12.2.2 Nominal shear strength at section, V_r, is computer from the following relation:

$$V_r = V_c + V_s \qquad (12\text{-}2)$$

Where V_c is nominal shear strength provided by concrete, and V_s is nominal shear strength provided by shear reinforcement. These strengths are called “ultimate shear strength for concrete” and “ultimate shear strength for reinforcement”. Values of V_c and V_s are calculated in accordance with requirements of 12.3 and 12.4.

❒ 12.3 Shear strength provided by concrete

Ultimate shear strength of concrete, V_c, shall be calculated in accordance with requirements of 12.3.1.1 through 12.3.1.3 or with more detailed analysis according to 12.3.2.

12.3.1.1 For members subject to shear and flexure:

$$V_c = v_c b_w d \qquad (12\text{-}3)$$

Where V_c is shear strength of concrete calculated with the use of:

$$v_c = 0.2\ \phi_c \sqrt{f_C} \qquad (12\text{-}4)$$

12.3.1.2 For members subject to shear, flexural and axial compression

$$V_c = v_c \left(\frac{1 + N_u}{12\,A_g}\right) b_w d \qquad (12\text{-}5)$$

12.3.1.3 For members subject to shear, flexure and significant axial tension, V_c shall be assumed zero. In such cases, cases, shear reinforcement shall be designed to provide, solely, total shear strength of section.

12.3.2 Ultimate shear strength, V_c, shall be computed by the more detailed calculations in accordance with 12.3.2.1 and 12.3.2.2.

12.3.2.1 For members subject to shear and flexure:

$$V_c = (0.95\, v_c + 12 \rho_w \frac{V_u d}{M_u})\, b_w d \qquad (12\text{-}6)$$

V_c shall not be taken greater than $1.75 v_c b_w d$.

In computing V_c by equation 12.6, quantity of $\frac{V_u d}{M_u}$ shall not be taken greater than unity. M_u is the ultimate flexural moment occurring simultaneously with V_u at section considered.

12.3.2.2 For members subject to shear, flexure and axial compression:
It shall be permitted to compute V_c from relation 12.6 with M_m substituted for M_u and the quality $V_u d/M_u$ not limited to unity.

$$M_m = M_u - N_u \frac{4h - d}{8} \qquad (12\text{-}7)$$

V_c shall not be taken greater than the value obtained from the following relation

$$V_c = 1.75 v_c \sqrt{1 + \frac{N_u}{3A_g}} b_w d \qquad (12\text{-}8)$$

When the value of M_m as computed by equation 12.7 is negative, V_c shall be computed from 12.8.

12.3.2.3 For members subject to shear, flexure and significant axial compression:

$$V_c = v_c (1 + \frac{N_u}{3A_g})\, b_w d \qquad (12\text{-}9)$$

Where N_u is negative.

❒ 12.4 Shear strength provided by shear reinforcement

12.4.1 Shear reinforcement may include the following types:
a) Stirrups perpendicular to member axis.

b) Welded wire fabrics with wires located perpendicular to axis of member provided that transverse wires can gain on elongation equivalent to at leas 0.04 within a length equivalent to at least 100 mm.
c) Stirrups making an angle of 45 degree or more with longitudinal tension reinforcement such that probable diagonal cracks are so interrupted.
d) Longitudinal reinforcement of maximum 36 mm in diameter with best portion making an angle of 30 degree or more with the longitudinal tension reinforcement, such that probable diagonal cracks are so interrupted.
e) Combinations of stirrups and best longitudinal reinforcement with conditions stated in a, c and d
f) Spirals

12.4.2 Ultimate shear strength of (shear) reinforcement, V_s, shall be computed, in different cases, in accordance with requirements of 12.4.2.1 through 12.4.2.6.

12.4.2.1 When shear reinforcement perpendicular to axis of member is used

$$V_s = \phi_s A_v f_y \frac{d}{s} \qquad (12\text{-}10)$$

Where A_v is the area of shear reinforcement used within distance s.

12.4.2.2 when inclined stirrups is used as shear reinforcement

$$V_s = \phi_s A_v f_y (\sin\alpha + \cos\alpha) \frac{d}{s} \qquad (12\text{-}11)$$

12.4.2.3 When shear reinforcement consists of a single bar a single group o parallel bars, all bent up at the some distance from support,

$$V_s = \phi_s A_v f_y \sin\alpha \qquad (12\text{-}12)$$

V_s shall be not greater than $1.5 b_w d$.

12.4.2.4 When shear reinforcement consists of a series parallel bent-up bars or several groups of parallel bent-up bars at different distances from support, ultimate shear strength shall be taken equal to 0.75 times value obtained from relation 12.11, but V_s shall not be greater than $2.5v_c b_w d$.

12.4.2.5 A longitudinal bent-up bar shall be considered effective for shear reinforcement only at central three-fourths of its inclined portion. Spacing of such reinforcement shall be selected such that requirement stated in 12.6.4.2 locates at center three-fourths of inclined portion of reinforcement cements.

12.4.2.6 Where more than one type of shear reinforcement is used in the portion of a member, ultimate shear strength V_s shall be equal to the sum of the V_s values computed for the various types.

12.4.3 Ultimate shear strength, V_s, shall be not taken greater than $4V_c$.

❒ 12.5 General principles of design for shear

12.5.1 When calculating ultimate shear strength at section, V_r, effect of any opening in members shall be considered.

12.5.2 In calculating shear strength of concrete, V_c, effects of axial torsion due to creep and shrinkage in restrained members shall be considered, whenever applicable. Also effects of tension and inclined flexural compression in variable depth members shall be included If the effects of tension and inclined flexural compression are in favorable direction, they may be neglected.

12.5.3 Maximum factored shear force at supports, V_u, may be reduced in accordance with 12.5.4 provided that:

a) Support reaction in direction of applied shear introduces compression in to the end regions of member.
b) No concentrated load occurs between internal face of support and location of critical section, in accordance with 12.5.4.

12.5.4 All section located at a distance less than d from internal face of supports may be designed for the shear, V_u, as that computed at a distance d.

❒ 12.6 Limitations of shear reinforcement

12.6.1 Specified yield strength shear reinforcement shall not exceed 400 MPa (N.mm^2).

12.6.2 Stirrups, bent longitudinal reinforcement and wire fabrics used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends according to 18.3.4 to develop the design yield strength of reinforcement.

12.6.3 Minimum shear reinforcement

12.6.3.1 A minimum area of shear reinforcement shall be provided force V_u exceeds one-half the shear strength provided by concrete V_c area of shear reinforcement shall not be less than the following:

$$A_v = 0.35 \frac{b_w s}{f_y} \qquad (12\text{-}13)$$

12.6.3.2 Requirements 12.6.3.1 may be neglected in the following cases:

a) Slabs and footings.
b) Roofs made of concrete joist construction as defined in 10.7.5.

c) Beams with total depth not less than 250 mm.
d) Beams cast integrally with slab, having total depth less than 2.5 times slab thickness or one-half width of web or 600 mm.

12.6.3.3 Minimum shear reinforcement requirements of 12.6.3.1 shall be permitted to be ignored if shown by acceptable tests that required flexural and shear strengths can be developed when shear reinforcement is omitted. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature changes, based on a realistic assessment of such effects occurring at service conditions.

12.6.3.4 When on the basis of 12.8.1, design for torsion is required, the minimum area of shear and torsion reinforcement is cumulatively calculated from the following equation:

$$A_V + 2A_t = 0.35\frac{b_w d}{f_y} \qquad (12\text{-}14)$$

These reinforcement shall be of closed stirrup type.

12.6.4 Maximum spacing for shear reinforcement

12.6.4.1 Spacing between shear reinforcement layers placed perpendicular to member axis shall not exceed d/2.

12.6.4.2 Inclined stirrups or bent longitudinal reinforcement shall be so spaced that every 45-degree line, extending toward the reaction from middepth of member d/2 to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

12.6.4.3 When nominal shear reinforcement, V_s, exceeds $2v_c b_w d$, maximum spacing given in 12.6.4.1 and 12.6.4.2 shall be reduced one-half.

❐ 12.7 Ultimate limit state of strength in torsion

12.7.1 When ultimate torsional moment, T_u, determined from structural analysis based on stiffness of the uncracked section, is less than 0.25 T_{cr}, it shall be permitted to neglect torsional effects; in which case, design for torsion need not be carried out. T_{cr} may be obtained from the following relation:

$$T_{cr} = 2\left(\frac{A_c^2}{P_c}\right) v_c \qquad (12\text{-}15)$$

v_c is calculated from equation 12.4 .

12.7.2 For sections subject to torsion, whenever design for torsion is required, controls of ultimate limit state of strength shall be based on the following relation:

$$\mathbf{T_u} \leq \mathbf{T_r} \qquad (12\text{-}16)$$

Where T_u is ultimate torsional moment at section considered, and is obtained from analysis of structure under ultimate loads; T_u is ultimate torsional strength of section, determined using the relation below:

$$T_r = T_s \qquad (12\text{-}17)$$

Where T_s is ultimate torsional moment provided by torsion reinforcement. For short, this strength is called “factored torsional moment of reinforcement of strength”. Due to cracking, distribution of concrete to provide torsional strength is waived. T_s is calculated in accordance with requirements of 12.8 through 12.10.

❒ 12.8 Ultimate torsional strength provided by torsion reinforcement

12.8.1 Torsion reinforcement shall consist of both longitudinal and transverse bars. Longitudinal reinforcement is similar to flexural bar and is distributed uniformly about the section. Transverse reinforcement may be in one of the following forms:

a) Closed stirrups perpendicular to member axis.
b) A closed cage of welded wire fabric having conditions stated in 12.4.1.b, with transverse wires perpendicular to the axis of the member.
c) Spirals.

12.8.2 Ultimate torsional moment of strength for reinforcement, $T_{s,}$ is determined using the following relation, provided that longitudinal torsion reinforcement is made available in accordance with 12.8.3.

$$T_s = 2\phi_c A_o A_t \frac{f_y}{s} \qquad (12\text{-}18)$$

Where A_o is area of the surface confined by torsion-induced shear at section. When more detailed calculation is not used, surface area of this portion may be taken equal to 0.85 A_{oh}.

12.8.3 The additional longitudinal reinforcement, A_r, required to provide torsional strength T_s, is determined from the following relation:

$$A_\ell = A_t \frac{P_h}{s} \qquad (12\text{-}19)$$

Reinforcement bars shall be uniformly distributed about section.

12.8.4 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to $M_u/(0.9d f_y)$, where M_u is the factored moment acting at the section simultaneously with T_u.

12.8.5 For hollow sections in torsion, distance from centerline of the transverse torsion reinforcement to the inside face of the wall of the hollow section shall not be less than $0.5A_{oh} / P_h$.

❒ 12.9 General principles of design for torsion

12.9.1 Regarding shear-torsion and flexure-torsion interactions, torsion reinforcement is permitted to be combined with reinforcement needed for other load-effects, provided that area of reinforcement used is equal to sum of values obtained for each part considered. The most restrictive requirements for reinforcement spacing and placement shall be met.

12.9.2 All sections located less than a distance d from the inside face of a support, shall be designed for the same torsion, T_u, computed at a distance d, provided that no concentrated flexural moment occurs within this distance.

12.9.3 Torsion reinforcement shall extend for a distance of at least (b_t+d) beyond the point that theoretically does not require torsion reinforcement.

❒ 12.10 Limitations of torsion reinforcement

12.10.1 Specified yield strength of transverse torsion reinforcement shall be taken not less than 400 MPa (N/mm^2).

12.10.2 Stirrups, wire fabric and transversal torsion reinforcement shall extend to a distance d from extreme compression fiber.

12.10.3 There shall be at least one longitudinal bar or tendon with diameter equivalent to s/16 or more, in each corner of the stirrup.

12.10.4 Torsion reinforcement shall be anchored in accordance with requirements of chapter 18.

12.10.5 Minimum transversal torsion reinforcement in members under torsion, which are to be designed for torsion in accordance with 12.7.1, are determined according to 12.6.3.4.

12.10.6 Axial spacing between transversal torsion reinforcement layers shall not exceed the two quantities $P_h/8$ and 300 mm.

12.10.7 Axial spacing of longitudinal torsion reinforcement distributed on the inside perimeter of transversal reinforcement shall not exceed 300 mm.

❒ 12.11 Calculation of factored torsional moment in statically indeterminate structures

12.11.1 When resistance against factored torsional moment,T_u, in a member is required to maintain its equilibrium, the member shall be designed to carry the factored torsional moment in accordance with requirements of 12.7.

12.11.2 In a statically indeterminate structure where reduction of the torsional moment in a member can occur due to redistribution of internal forces, the maximum factored torsional moment, T_u, may be reduced to $0.67T_{cr}$,provided that effect of reduced moments and shears on the other adjacent members is calculated and applied to the design, using balance relations.

12.11.3 Unless determined by a more exact analysis and that requirements of 12.11.2 are not applied, the factored torsional moment due to slabs acting on load-carrying beams may be substituted by a uniform linear distribution.

❒ 12.12 Combined effect of shear and torsion

12.12.1 In the design of members subject to combined effect of shear and torsion, section shall be designed once for shear and once for torsion. Reinforcement required for torsion shall be added to that required for shear, moment, and axial force that act in combination with the torsion. If necessary, reinforcement required for other load-effects shall also be added.

12.12.2 Dimensions of sections subject to combined effect of shear and torsion shall be designed such that the following relation is satisfied:

$$\frac{V_u}{b_w d} + \frac{T_u P_h}{A_{oh}^2} \leq 0.25\, \phi_c f_c \qquad (12\text{-}20)$$

❒ 12.13 Shear–friction

12.13.1 scope

Provisions of this part is applied for shear transfer across two planes (or faces) such as the following:

a) An existing crack or potential cracking across two faces
b) An interface between two planes of dissimilar materials
c) Interface between two concretes cast in different times

12.13.2 Ultimate limit state of strength

12.13.2.1 At sections where shear transfer is conducted as shear-friction, ultimate limit state of strength shall be based on the Eq. (12-1), where the factored shear force for strength at section, V_r, is determined in accordance with 12.13.2.2.

12.13.2.2 Assuming a crack along the entire shear plane (or section) considered, factored shear strength at section, V_r, is calculated in accordance with requirements of 12.13.2.3 through 12.13.2.6 or 12.13.2.7. Requirements of 12.13.3 shall also be regarded for each of the above cases.

12.13.2.3 Where shear-friction reinforcement is inclined to shear plane, such that the shear force produces tension in shear-friction reinforcement:

$$V_r = \phi_s A_{vf} f_y (\mu \sin\alpha_f + \cos\alpha_f) \qquad (12\text{-}21)$$

Where α_f is angle between shear reinforcement and shear plane.

12.13.2.4 When shear –friction reinforcement is perpendicular to shear plane:

$$V_r = \mu\phi_s A_{vf} f_y \qquad (12\text{-}22)$$

Where μ is the coefficient of friction in accordance with 12.13.2.5.

12.13.2.5 Coefficient of friction μ in equations (12-21) and (12-22) is taken equal to one of the following values:

a) Concrete placed monolithically 1.25
b) Concrete placed against an already hardened concrete with surface roughness specified in 12.13.3.5 0.90
c) Concrete placed against an already hardened concrete with surface roughness less than that specified in 12.13.3.5 0.50
d) Concrete anchored to structural steel by headed studs or by reinforcement bars 0.60

12.13.2.6 Factored shear strength at section, V_r, shall not be taken greater than $0.25\phi_c f_c' A_{cv}$ and $6.5\phi_c A_{cv}$, where A_{cv} is the area of concrete section resisting shear transfer.

12.13.2.7 Factored shear strength at section, V_r, may be determined by any other design method, validity of which is verified by comprehensive tests.

12.13.3 Principles of shear-friction design

12.13.3.1 Design yield strength of shear–friction reinforcement shall not exceed 400MPa (N/mm^2).

12.13.3.2 When shear plane, in addition to shear force is subject to tensile force, additional reinforcement shall be provided to carry tension in the direction of applied tensile forces.

12.13.3.3 When shear plane, in addition to shear force is subject to permanent compression force, the new force is added to the force $\phi_s A_{vf} f_y$, due to shear-friction reinforcement, in equation (12-21).

12.13.3.4 Shear–friction reinforcement shall be appropriately distributed along the shear plane and shall be anchored on both sides by mechanical tools to develop the specified yield strength.

12.13.3.5 When concrete is placed against previously hardened concrete, the interface area for shear transfer shall be clean and free of laitance. If μ is to be assumed 0.90, interface area shall be intentionally roughened to a depth of approximately 5mm.

12.13.3.6 When shear is transferred between as–rolled structural steel and concrete using headed studs or welded reinforcement bars, steel shall be clean and free of paint.

❐ 12.14 Special provisions for deep flexural members (deep beams)

12.14.1 Scope

Provisions of this part shall apply to flexural members meeting the following conditions:

a) Ratio of clear span length to effective depth $\frac{\ell_n}{d}$ is less than 5.

b) Load on compression face of beam is supported on the opposite face so that compression struts can develop between loads and the supports.

12.14.2 Ultimate limit state of strength in flexure.

12.14.2.1 Ultimate limit state of strength in flexural deep beams shall be based on Eq. (12-1) and (12-2), where the shear strength of concrete, V_c, and the shear strength for steel bars, V_s, shall be in accordance with 12.14.2.4 and 12.14.2.5.

12.14.2.2 Ultimate limit state of control for strength in flexural deep beams is developed only at the critical section of member as defined in 12.14.2.3 and the required shear reinforcement in the critical section shall be used throughout the span.

12.14.2.3 Critical section in deep beams measured from face of support, shall be taken at a distance $0.15\,\ell_n$ for uniformly distributed beams and 0.5a for beams with concentrated loads. The distance shall not be taken greater than d.

12.14.2.4 Ultimate shear strength of concrete V_c may be obtained from the following relation:

$$V_c = v_c\, b_w\, d \qquad (12\text{-}23)$$

And with more details from the following:

$$V_c = (3.5 - 2.5\frac{M_u}{V_u d})(0.95 v_c + 12\rho_w \frac{V_u d}{M_u})\, b_w d \qquad (12\text{-}24)$$

Argument $(3.5 - 2.5\frac{M_u}{V_u d})$ shall not exceed 2.5 and V_c shall be taken greater than $3v_c b_w d$. M_u is the factored flexural moment occurring simultaneously with V_u at the critical section defined in 12.14.2.3.

12.14.2.5 Factored shear strength for steel bar is computed using the equation:

$$V_s = \left[\frac{A_V}{12s}(1 + \frac{\ell_n}{d}) + \frac{A_{vh}}{12s_2}(11 - \frac{\ell_n}{d})\right]\phi_s f_y\, d \qquad (12\text{-}25)$$

Where A_v is the area of shear reinforcement perpendicular to the horizontal flexure reinforcement within a distance s, and A_{vh} is the area of shear reinforcement within a distance s_2.

12.14.2.6 Required shear strength of concrete at the section, V_r, shall not be greater than the following values:

a) When $\frac{\ell_n}{d}$ is less than 2:

$$V_r \le 4 v_c b_w d \qquad (12\text{-}26)$$

b) When $\frac{\ell_n}{d}$ is between 2 and 5 :

$$V_r \leq \frac{1}{3} v_c (10 + \frac{\ell_n}{d}) b_w d \qquad (12\text{-}27)$$

12.14.3 Limitations of shear reinforcement

12.4.3.1 Area of shear reinforcement A_r shall be not less than $0.0015 b_w s$. Spacing of such reinforcement shall not exceed d/5 and 350mm.

12.14.3.2 Area of the horizontal shear reinforcement A_{vh} shall be not less than $0.0025 b_w s_2$. Spacing of such reinforcement shall not be greater than d/3, and 350mm.

❒ 12.15 Special provisions for brackets and corbels

12.15.1 Scope

Provisions of this part shall apply to brackets and corbels meeting the following conditions:

a) Ratio of shear span to effective depth of the section at support face, a/d, does not exceed unity.
b) Ultimate tensile strength acting on them, N_u, is greater than the ultimate shear force, V_u, acting on then.
c) Effective depth of section at outside edge of bearing area shall not be less than 0.5d.

12.15.2 Ultimate limit state of strength in shear, flexure and tension

12.15.2.1 Concerning brackets and corbels, ultimate limit state of control for strength in shear flexure and axial tensile force shall be based on equations 12-1, 11-1, 11-2.

Where V_u, M_u and N_u are ultimate shear force, ultimate flexural moment and ultimate axial tensile force respectively, acting simultaneously on support section. V_r, M_r and N_r are ultimate strength of section in shear and in flexure, and required axial tensile force, respectively. The strengths are calculated in accordance with 12.15.2.4 through 12.15.2.6.

12.15.2.2 V_u and N_u are determined from structural analysis under ultimate loads. Unless special provisions are made to avoid tensile forces, N_u used in the analysis shall not be less than $0.2V_u$.Tensile load N_u shall continuously be regarded as live load.

12.15.2.3 Ultimate flexural moment M_u is computed from the equation:

$$M_u = V_u a + N_u (h - d) \qquad (12\text{-}28)$$

Where a is the distance from centroid of force to the support face, and h and d are the total depth and effective depth of section at support face.

12.15.2.4 Ultimate shear strength at section, V_r, is computed in accordance with requirements of 12.13, assuming a shear-friction force at the section. Shear-friction reinforcement, A_{vf}, to resist shear strength (V_r) shall not be taken greater than the following two values: $0.25\phi_c f_c b_w d$ and $6.5\phi_c b_w d$.

12.15.2.5 Ultimate flexural moment in section, M_r, is designed in accordance with requirements of chapter 11 tensile reinforcement, providing strength (M_u) is denoted as (A_r).

12.15.2.6 Ultimate tensile strength of section, N_r, is computed using the equation:

$$N_r = \phi_s A_n f_y \qquad (12\text{-}29)$$

Where A_n is the tensile reinforcement providing strength N_r.

12.15.3 General principles of design

12.15.3.1 Area of primary tension reinforcement, A_s, shall be neither less than (A_f+A_n), nor ($\frac{2}{3}$ A_f+A_n).

12.15.3.2 Closed stirrups parallel to A_s with a total area of A_h equal to greater than 0.5(A_s-A_n), shall be uniformly distributed within two-thirds of effective depth adjacent to A_s.

12.15.3.3 Ratio of tensile reinforcement $\rho=A_s/bd$ shall be less than $0.04f_c/f_y$.

12.15.3.4 Primary tension reinforcement at front face of bracket or corbel shall be anchored by one of the following:

a) A structural weld to a transverse bar of at least equal size to primary tensile reinforcement; weld strength shall be adequate to transfer $A_s f_y$ force (i.e. develop specified yield strength f_y for A_s bars).
b) Bending primary tension bars A_s back to form a horizontal loop.
c) Other methods.

12.15.3.5 Bearing area of load on bracket or corbel shall not project beyond interior face of transverse anchor bar (if one is provided).

❒ 12.16 Special provisions for walls

12.16.1 Scope

12.16.1.1 Provisions of this chapter applies to design of horizontal shear forces in plane of wall.

12.16.1.2 Design of shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in 12.17.

12.16.2 Ultimate limit state of strength in shear

12.16.2.1 Ultimate limit state of control in design of horizontal section for shear in plane of wall shall be based on equations (12-1) and (12-2), where shear strength of concrete (V_c) and shear strength of steel reinforcement (V_s) shall be in accordance with 12.16.2.1 through 12.16.2.5.

12.16.2.2 When wall is subject to both shear and compression, shear strength of concrete may be computed from equation (12-3), and when the wall is subject to shear and tension simultaneously, shear strength is given by (12.9). A more detailed analysis shall be in accordance with 12.6.3.2.

12.6.2.3 When shear strength of concrete V_c is to be computed more exact and with more details, it shall be taken the lesser of the following two equations:

$$V_c = 1.65 v_c hd + \frac{N_u d}{5\ell_w} \qquad (12\text{-}30)$$

$$V_c = \left[0.3 v_c \frac{\ell_w (0.6 v_c + 0.15 \frac{N_u}{\ell_w h})}{(\frac{M_u}{V_u} - \frac{\ell_w}{2})} \right] hd \qquad (12\text{-}31)$$

Where N_u is positive for pressure and negative for tension. $(\frac{M_u}{V_u} - \frac{\ell_w}{2})$ is negative, equation (12-31) shall not apply.

12.16.2.4 Sections located close to wall base than a distance $\frac{\ell_w}{2}$ or $\frac{h_w}{2}$, whichever is less, shall be permitted to be designed for the same shear strength of concrete V_c as that computed at a distance equal to lesser of the above values.

12.16.2.5 Factored shear strength of reinforcement shall be computed by:

$$V_s = \phi_s A_v f_y \frac{d}{s_2} \tag{12-32}$$

Where A_v is the area of shear reinforcement in the direction of shear within a distance s_2. Distance d shall be in accordance with 12.16.3.2. Adequate shear strength, V_s, shall be developed when vertical shear reinforcement (as defined by requirements of 12.16.4.2) is also provided in the wall, in addition to horizontal shear reinforcement A_v vertical reinforcement are is determined in accordance with 12.16.4.2.

12.16.2.6 Required shear strength V_r shall not be taken greater than $5V_c hd$.

12.16.3 General principles of design

12.16.3.1 When factored shear force V_u exceeds $0.5V_c$, wall reinforcement to resist shear shall be provided in accordance with requirements of 12.16.2. Restrictions of 12.6.4 shall apply to the wall reinforcement. When V_u is less than $0.5V_c$, reinforcement shall be provided in accordance with 12.16.4 or by requirements of load-carrying walls in chapter 16.

12.16.3.2 In designing walls for shear, effective depth of section, d, shall be taken equal to $0.8\ell_w$. A larger value of d, equal to the distance from extreme compression fiber to the center of force of all reinforcement in tension, provided that tensile forces considered are determined with regard to compatibility of strains at the section.

12.16.3.3 Shear strength V_r at construction joints of walls shall be based on shear-friction conduct, in accordance with 12.13.

12.16.4 Reinforcement limitations

12.16.4.1 Ratio of shear reinforcement area in the extension of shear to gross concrete area of vertical section, ρ_h, shall not be less than 0.0025. Spacing of shear reinforcement s_2 shall not exceed 3h, $\frac{\ell_w}{5}$ or 350mm.

12.16.4.2 Ratio of vertical shear reinforcement area to gross concrete area of horizontal section, ρ_n, shall be neither less than 0.0025 nor less than the value given by:

$$\rho_n = 0.0025 + 0.5(2.5 - \frac{h_w}{\ell_w})(\rho_h - 0.0025) \qquad (12\text{-}33)$$

ρ_n need not be greater than ρ_h. Spacing of vertical shear reinforcement s_1 shall not exceed 3h, $\frac{\ell_w}{3}$ nor 350mm.

❒ 12.17 Special provisions for slabs and footings

12.17.1 Scope

12.17.1.1 Provisions of this part shall apply to shear control in slabs and footings such as flat slab over column and the footing subject to column load, both of which are under concentrated load or that they transfer loads to limited area supports.

12.17.1.2 Slabs that are subject to distributed loads and that transfer loads to beams or walls, have a behavior like that of beams and they are not subject to requirements of members under shear and flexure. Shear control in such slabs shall be in accordance with requirements of 12.2 through 12.6.

12.17.1.3 Slabs subject to gravity load, wind, earthquake, or other lateral forces, directly transfer flexural moments to columns. Shear force contribution of the moment is transferred to slab sections in the vicinity of columns. The resulting shear force shall be considered in the design of shear in accordance with requirements of 12.17.5.

12.17.2 Ultimate limit state of strength in shear

12.17.2.1 Shear strength of slabs and footings in the vicinity of concentrated loads and limited area supports shall be governed by two types of action as follows:

a) One-way action as a beam, where slab or footing shall carry all shear forces on its entire width just like a beam the critical section on which, slab or footing strength is to be controlled, is considered as a plane perpendicular to slab at a distance d from the edge of concentrated load action area or from drop panel face or from any other changes in the thickness of slab or support, across the entire width of slab.
b) Two-way action where slab or footing shall carry shear force on two directions, but in an area restricted to the vicinity of concentrated load or support. Critical section in this case is the lateral face of a prism with its faces perpendicular to slab surface and their distance from edges and corners of the action area of concentrated load or support, or from those sections of slab with varying thickness is at least equal to d/2. Critical section shall be located so that the perimeter of its orthogonal base, b_o, is a minimum. For square or rectangular columns, concentrated loads, or

reaction areas, the critical sections with four straight sides shall be permitted.

12.17.2.2 For slabs and footings, shear control in ultimate limit state of strength is similar to beam with regard to one-way action and shall be designed in accordance with requirements of 12.2 through 12.16.

12.17.2.3 For slabs and footings, shear in ultimate limit state of strength, for two-way action, shall be based on equations (12-1) and (12-2). V_r or V_c and V_s in the above formulas are given by requirements of 12.17.2.4 through 12.17.2.6.

12.17.2.4 For slabs and footings, in which shear reinforcement or shearhead is not applied, ultimate shear strength of concrete, V_c, is taken equal to the least value given by the following three equations:

$$V_c = (1+\frac{2}{\beta_c})v_c b_o d \qquad (12\text{-}34)$$

$$V_c = (\frac{\alpha_s d}{b_o}+1)v_c b_o d \qquad (12\text{-}35)$$

$$V_c = 2v_c b_o d \qquad (12\text{-}36)$$

Where β_c is the ratio of length to width of the concentrated load or reaction area. α_s is 20 for interior columns, 15 for columns and 10 for corner columns.

12.17.2.5 Shear reinforcement consisting of bars or welded wire fabric shall be permitted in slabs and footings to provide for shear strength. V_c and V_s are determined in accordance with the following:

a) Ultimate shear strength of concrete, V_c, is computed from:

$$V_C = v_c b_o d \qquad (12\text{-}37)$$

b) Ultimate shear strength for reinforcement ,V_s, is computed using requirements of 12.14

c) Ultimate shear strength of section , V_r , shall not be taken greater than $3v_c\, b_o\, d$

12.17.2.6 In slabs where shearhead is used as I-shaped or channel-shaped section steel profiles to provide shear strength, ultimate shear strength of the section, V_r, shall be based on the following, provided that requirements of 12.17.3 are met:

a) where slabs can only transfer shear due to gravity load, V_r is taken equal to the least of the following two values:

$$V_c = 3.5\, v_c b_o d \qquad (12\text{-}38)$$

$$V_c = 2\, v_c b_{om} d \qquad (12\text{-}39)$$

b_o is perimeter of the orthogonal critical section as defined in 12.17.2.1.b and b_{om} is perimeter of special orthogonal critical section as defined in 12.17.2.7.

b) Where flexural moment, in addition to shear due to gravity load, is transferred to column, V_r shall be such that requirements of 12.17.5.3.b shall apply.

12.17.2.7 Special critical section that is to be used in shearhead slabs to control the ultimate shear strength, is the lateral area of prism that its faces are perpendicular to slab and is located at distance 0.75 ($\ell_v - 0.5C_1$) from column face. Special critical section shall be considered so that perimeter of orthogonal base of prism, b_{om}, is at minimum. Distance between prism faces to column edge need not be loss than d/2.

12.17.3 Principles and limitations of shearhead

12.17.3.1 Each shearhead shall consist of I-shaped or channel-shaped steel members fabricated by welding with a full penetration weld into identical arms at right angles. Shearhead arms shell not be interrupted within the column section.

12.17.3.2 The plastic flexural moment strength M_p required for each arm of the shearhead shall not be less than the following value:

$$M_p = \frac{V_u}{2n}\left[h_v + \alpha_v(\ell_v - 0.5c_1)\right] \qquad (12\text{-}40)$$

Where η is the number of arms and ℓ_v is the minimum length of each shearhead arm required to comply with requirements of 12.17.2.6.

12.17.3.3 The ends of each shearhead arm may be cut at angles greater than 30 degree with the horizontal, provided that the plastic flexural moment strength of the remaining tapered section is adequate to resist the shear force allocated to that arm of the shearhead.

12.17.3.4 Shearhead section shall be selected based on the following conditions:

a) A shearhead shall not be deeper than 70 times its web thickness.
b) All compression flanges of shearhead section shall be located within 0.3d of extreme compression fiber of the slab.
c) The ratio α_v between flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width (c_2+d) shall not be less than 0.15.

12.17.3.5 Flexural strength of shearhead arms may be contributed to flexural moment strength of each slab column strip. The contribution of each arm is determined by the following equation:

$$M_v = \frac{\alpha_v V_u}{2\eta}(\ell_v - 0.5c_1) \qquad (12\text{-}41)$$

Where l_v is the length of each shearhead arm actually provided. M_v shall not be taken greater than the following values:

a) 30 percent of the ultimate flexural moment occurred in slab column strip.

b) The change in slab column strip flexural moment over the length ℓ_v.

c) The value of shearhead plastic moment strength M_p

12.17.3.6 When the slab is to transfer flexural moment to the column, the shearhead must have adequate anchorage transmit M_p.

12.17.4 Openings in slabs

12.17.4.1 When openings in slabs are located at a distance less than 10 times the slab thickness from the concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in 15.4.5, the critical slab sections for shear control, defined in 12.17.2.1.b and 12.17.2.7 shall be modified in accordance with 12.17.4.2 and 12.17.4.3.

12.17.4.2 For slabs without shearhead, that part of the perimeter of the critical section that is enclosed by straight lines projected from the centroid of the column, concentrated load, or reaction area and tangent to the boundaries of the openings shall be considered ineffective.

12.17.4.3 For slabs with shearhead, the ineffective portion of the perimeter shall be one-half of that defined in 12.17.4.2.

12.17.5 Transfer of moment in slab-column connections

12.17.5.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment M_u between a slab and a column, a fraction M_{uf} of the unbalanced moment shall be transferred by flexure in accordance with 15.4.3. The remainder of the unbalanced moment given by M_{uv} shall be considered to be transferred by eccentricity of shear about the centroid of the column in the slab. M_{uv} is computed from the following equation:

$$M_{uv} = (1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b_1}{b_2}}}) M_u \qquad (12\text{-}42)$$

12.17.5.2 The determining the shear stress due to flexural moment M_{uv}, it is assumed that the maximum shear stress as defined in 12.17.2.1.b is created in the critical section. The shear stress in each fiber in this section is assumed to vary linearly with the distance of fiber from the centroid of critical section.

12.17.5.3 When the slab, in addition to shear force, V_u, is subject to shear due to flexural moment transfer, ultimate shear strength of slab shall be adequate to resist the sum of two effects. In the design of shear strength of slab, for the ultimate limit state, the following requirements shall be observed:

a) For slabs without shear head, the sum of shear stress due to vertical loads acting on the critical section, in accordance with 12.17.2.1.b, and maximum shear stress calculated in 12.17.5.2, shall not be less than $\frac{V_r}{b_o d}$.

   Where V_r is the ultimate shear strength of the critical section.

b) For slabs with shear head, the sum of shear stress due to vertical loads acting on the special critical section, defined in 12.17.2.7, and maximum shear stress calculated in 12.17.5.2, shall be less than $2v_c$.

❒ 12.18 Special provisions for frame connections

12.18.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.

12.18.2 At the restrained connection between framing elements and columns, the equivalent shear reinforcement which is at least equal to that given by equation (12-13), shall be installed in the column. The shear reinforcement shall extend to a depth at least equal to that the deepest connection of framing elements to the columns.

12.18.3 When column connection is confined on the four sides by frame elements of approximately equal depth, observance of requirements in 12.18.2 is compulsory.

12.18.4 At the connections of frames not part of resisting elements against earthquake lateral loads, special requirements shall be observed. These priciples are given in chapter 20.

CHAPTER THIRTEEN

SLENDERNESS EFFECTS-BUCKLING

❒ 13.0 Notation

A_g = gross area of section, mm^2.
C_m= factor relating actual moment diagram to an equivalent uniform moment diagram.
e = load eccentricity used in the calculation of flexural moment of the member and slenderness effect, mm.
e_{min}= minimum eccentricity of load used in the calculation of flexural moment of the member and slenderness effect, mm.
e_s = load eccentricity, based on short column, mm.
EI_e = flexural stiffness of compression member, Eqs. (13-11), (13-13).
E_c = modulus of elasticity of concrete, MPa (N/mm^2).
E_s = modulus of elasticity of reinforcement, MPa (N/mm^2) .
f_c= specified compressive strength of concrete, MPa (N/mm^2).
f_y = specified yield strength of reinforcement (f_{yk}), MPa (N/mm^2). For simplicity, letter k is omitted from subscript in this chapter.
h = overall thickness of member, mm.
h_s = story height, mm.
H_u= overall ultimate lateral load on story, N.
I_g= moment of inertia of gross concrete section, mm^4.
I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, mm^4.

k= effective length factor, Eqs. (13-2) through (13-6).

k´= effective length factor, Eq. (13-5).

ℓ_u= unsupported length of compression member-unanchored length of compression member, mm.

M_c= magnified ultimate flexural moment, N.mm.

M_ℓ = ultimate flexural moment on long column, N.mm.

M_1= smaller factored flexural moment on both ends of a compression member, positive if bent in single curvature, negative if bent in double curvature, N.mm.

M_{1b} = ultimate flexural end moment on a compression member at the end of which M1 acts, due to loads that cause no appreciable sidesway, N.mm.

M_{1s} = ultimate flexural end moment on compression member at the end at which M1 acts, due to loads that cause appreciable sidesway, N.mm.

M_2 = larger factored end moment on a compression member, positive, N.mm.

M_{2b} = ultimate flexural end moment on a compression member at the end at which M_2 acts, due to loads that cause no appreciable sidesway, N.mm.

M_{2s} = ultimate flexural end moment at the end of which M_2 acts, due to loads that cause appreciable sidesway, N.mm.

N_c = critical load, N.

N_ℓ= compressive strength of column against load of given eccentricity, N.

N_o = compressive strength of column against load with zero eccentricity, N.

N_u= ultimate compressive axial load, N.

Q= stability index for a story.

r = radius of gyration, mm.

R= load–carrying capacity reduction factor in compression members.

β_d = a) for braced frames and for stability control of sway frames, β_d is the ratio of the maximum factored axial dead load to the total factored axial load.

b) for sway frames, β_d is the ratio of the maximum factored sustained shear within a story to the total factored shear in that story.

δ_b= magnification factor to reflect the effect of member curvature.

δ_s = magnification factor to reflect lateral drift.

δ_u= drift of a story relative to its lower floor for any specific combination of loads.

ϕ_n = modification factor.

ψ = parameter showing end support conditions for compression member, equal to the ratio sum stiffness of columns to that of beams, all of which end up into one joint in one plate.

ψ_m= average value of ψ at two ends of compression member.

ψ_{min} = smallest value of ψ at two ends of compression member.

❒ 13.1 Scope

13.1.1 Effects of axial loads acting on slender prismatic elements subject to compression with no flexure or with given flexure, and the method to incorporate loads in the design of members shall be investigated in this chapter. These effects are collectively referred to as slenderness effects.

13.1.2 Slenderness effects, including effects due to occurrence of curvature in the member and effects of drift at two ends of the member are as following:

a) Effects curvature development on the member, are the flexural moments created due to non-coincidence of the centroid of section over the line that connects its two ends .
b) Effects of drift are the flexural moments and other load-effects that are developed in the sections of the member, due to eccentricity of drift in one end of member relative to its other end. The drift at both ends may be due to vertical or lateral loads or combinations of them.

❒ 13.2 General

13.2.1 Design of compression members, their restraining beams and other elements that carry such member loads shall be done for forces and moments derived from structural analysis. In addition to forces acting on the structure, considered in the usual analysis of the structures, slenderness effects as defined in 13.1, effects of changes in moment of inertia (due to cracking, non-linear behavior of materials, and shrinkage), time dependent effects of long-term loads shall also be incorporated in the analysis.

13.2.2 If the effects mentioned in 13.2.1 are not considered in the structural design, they may be calculated with a good approximation using "flexural moment magnification" method in accordance with 13.8 and observing restrictions stated in 13.7.3.

13.2.3 In usual short buildings, up to four stories from ground level, if the effects stated in 13.2.1 are not incorporated in the structural analysis, slenderness effects may be calculated with good approximation using "load-carrying capacity reduction" method in accordance with requirements of 13.9 and with observance of restrictions in 13.7.3. Effects of cracking and those of time-dependent long-term loads shall also be implicitly considered in this method.

❒ 13.3 Laterally braced stories

13.3.1 Braced story is the story in which relative drift is negligible. If the stability index of the story, obtained from 13.1 is less than 0.05, the story is referred to as laterally braced story. In this case, all compression members in the story are called nonsway.

$$Q = \frac{\sum N_u \delta_u}{H_u h_s} \tag{13-1}$$

Where N_u is the ultimate compressive axial load, H_u is the ultimate total lateral load acting on the story, δ_u is the story drift relative to the next lower floor due to above forces, and h_s is the total height of the story in question.

13.3.2 In usual short buildings, when sum of lateral stiffness for each restraining member in the story, such as shear wall and bracing, is equal to or greater than 6 times the sum lateral stiffness for the columns of that story, the story may be referred to as braced.

❒ 13.4 Unsupported length of compression members

13.4.1 The unsupported length of a compression member, ℓ_u shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support for that member.

13.4.2 Where column capitals or drop panels are present, the unsupported length shall be measured to the lower surface of capital or drop panel.

❒ 13.5 Effective length of compression member

13.5.1 Effective length of compression member, $K\ell_u$ may be calculated in accordance with 13.5.2 through 13.5.4, unless by carrying out a more detailed analysis, in which member cracking effects on their lateral stiffness are incorporated, another effective length is obtained.

13.5.2 For compression members in a braced frame, the effective length factor may be taken equal to 1 or smaller of the following two Eqs. (13-2) and (13-3):

$$k = 0.7 + 0.1\psi_m \le 1 \qquad (13\text{-}2)$$

$$k = 0.85 + 0.05\,\psi_{min} \le 1 \qquad (13\text{-}3)$$

13.5.3 For compression members in a sway frame restrained at ends, the effective length factor, k, may be obtained by the Eq. (13-4) or (13-5):

When $\psi_m < 2$

$$k = (1 - 0.05\psi_m)\sqrt{1 + \psi_m} \ge 1 \qquad (13\text{-}4)$$

When $\psi_m \ge 2$

$$k = 0.9\sqrt{1 + \psi_m} \qquad (13\text{-}5)$$

13.5.4 For compression members in a sway frame hinged at one end, the effective length factor, k, may be obtained from Eq. (13-6):

$$k = 2 + 0.3\psi \qquad (13\text{-}6)$$

Where ψ is the value at restrained (unhinged) end.

13.5.5 In calculating moments of inertia to determine ψ, Eq. (13-2) through (13.6) shall be applied in accordance with requirements of 13.8.1.

❐ 13.6 Radius of gyration

13.6.1 Radius of gyration, r, may be calculated as following:

a) For rectangular sections: 0.3 times the overall dimension of section in the direction where slenderness effect is investigated .

b) For other sections: radius of gyration in the extension considered is obtained from:

$$r = \sqrt{\frac{I_g}{A_g}} \qquad (13\text{-}7)$$

❒ 13.7 Provisions for slenderness effect

13.7.1 For nonsway compression members that satisfy $k\frac{\ell_u}{r} \leq 34 - 12\frac{M_1}{M_2}$, it shall be permitted to ignore slenderness effect. $\frac{M_1}{M_2}$ shall not be less than -0.5.

13.7.2 For nonsway compression members, if $k\frac{\ell_u}{r} \leq 22$ is satisfied, slenderness effect may be ignored.

13.7.3 For compression members that satisfy $k\frac{\ell_u}{r} > 100$, slenderness effect shall be investigated based on accurate analysis, in accordance with 13.2.1.

13.7.4 Utilization of compression members with $k\frac{\ell_u}{r} > 200$ is not permitted.

❒ 13.8 Flexural moment magnification method

13.8.1 According to this method, ultimate flexural moments obtained from normal elastic analysis are magnified through applying requirements of 13.8.2 for braced frames, and those of 13.8.3 for sway frames. The moments along with ultimate axial load resulting from the said analysis is taken as basis for design of compression member. Cracking effects of structural elements and influence of long term loads shall be properly incorporated in the linear elastic analysis. Utilization of the recommended values in 10.3.4.2

is thus acceptable. In presence of long-term lateral loads, EI values shall be divided to $(1+\beta_d)$.

13.8.2 Braced stories

13.8.2.1 For compression members of braced stories, the amplified flexural moment, M_c, is calculated by the Eq. (13-8):

$$M_c = \delta_b M_2 \tag{13-8}$$

The coefficient δ_b is obtained from 13-9:

$$\delta_b = \frac{C_m}{1-\dfrac{N_u}{\phi_n N_c}} \geq 1 \tag{13-9}$$

Where ϕ_n=0.65. Factor C_m and critical load N_c are determined as following:

$$C_m = 0.6 + 0.4\left(\frac{M_{1b}}{M_{2b}}\right) \geq 0.4 \tag{13-10}$$

For other cases C_m=1.

In the Eq. (13-10), $\dfrac{M_{1b}}{M_{2b}}$ is positive if the two moments cause the column to bend in single curvature and is negative if bent in double curvature. Critical load N_c is calculated by the Eq. (13-11):

$$N_C = \frac{\pi^2 EI_e}{(K\ell_n)^2} \tag{13-11}$$

$$EI_e = \frac{0.2E_c I_g + E_s I_{se}}{1+\beta_d} \tag{13-12}$$

Or approximately:

$$EI_e = 0.25E_C I_g \tag{13-13}$$

The factor K in the Eq. (13-11) is given by 13.5.2 for calculation of δ_b .

13.8.3 Sway stories

13.8.3.1 The moments M_1 and M_2 in compression members of sway stories are given by Eqs. (13-14) and (13-15):

$$M_1 = M_{1b} + \delta_s M_{1s} \qquad (13\text{-}14)$$

$$M_2 = M_{2b} + \delta_s M_{2s} \qquad (13\text{-}15)$$

Values of $\delta_s \mathbf{M_{1s}}$ and $\delta_s M_{2s}$ may be obtained from one of the methods in 13.8.3.2 through 13.8.3.4.

13.8.3.2 The magnified moments $\delta_s \mathbf{M_{1s}}$ and $\delta_s \mathbf{M_{2s}}$ are the column end moments calculated using a second-order elastic analysis based on materials specification and geometric conditions of the section, in accordance with requirements of 13.8.1.

13.8.3.3 An alternative method to calculate magnified moments $\delta_s \mathbf{M_{1s}}$ and $\delta_s M_{2s}$ is the use of Eq. (13-16).
Utilization of this method is permitted only if stability index of the story does not exceed 1/3.

$$\delta_s M_s = \frac{M_s}{1 - Q} \geq M_s \qquad (13\text{-}16)$$

13.8.3.4 Another alternative method to calculate magnified moments $\delta_s \mathbf{M_{1s}}$ and $\delta_s M_{2s}$ is using the Eq. (13-17):

$$\delta_s M_s = \frac{M_s}{1-\dfrac{\sum N_n}{\phi_n \sum N_n}} \geq M_s \qquad (13\text{-}17)$$

Where modification factor $\phi_n = 0.65$, $\sum N_u$ is the sum of ultimate gravity loads and $\sum N_c$ is sum of critical loads for those story columns resisting lateral deflection. For any compression member, N_c is calculated by Eq. (13-11) through (13-13). Coefficient K in the Eq. (13-11) is calculated in accordance with 13.5.3 or 13.5.4.

13.8.3.5 When a compression member satisfies the following equation:

$$\frac{\ell_u}{r} > \frac{35}{\sqrt{\dfrac{N_u}{f_c A_g}}} \qquad (13\text{-}18)$$

The compression member shall be designed for the ultimate axial load and the critical flexural moment at each end of the member, M_c, is calculated by Eq. (13-19):

$$M_c = \delta_b (M_b + \delta_s M_s) \qquad (13\text{-}19)$$

13.8.3.6 Considering calculation method for $\delta_s M_s$, the strength and stability of the structure as a whole under ultimate gravity loads shall be controlled by one of the following methods:

a) When $\delta_s M_s$ is calculated by 13.8.3.2, the ratio of second-order lateral deflections to first-order lateral deflections due to factored gravity loads and lateral loads applied to the structure shall not exceed 2.5.

b) When $\delta_s M_s$ is calculated by 13.8.3.3, story stability ratio, Q, calculated by factored loads shall not exceed 0.6 .

c) When $\delta_s M_s$ is calculated by 13.8.3.4, the factor δ_s calculated by $\sum N_u$ and $\sum N_c$ corresponding to the factored gravity loads shall be positive and shall not exceed 2.5.

❐ 13.9 Load-carrying capacity reduction method

13.9.1 Effect of slenderness, in this method, is permitted to be considered as a reduction in load-carrying capacity strength of the compression member. As such, axial load and flexural moment strength in slender member are not taken less than the corresponding values for non-slender member.
Load-carrying capacity reduction factor applied in this method, R, is less than unity and is either multiplied by axial load and the flexural moment strength of the member, or alternatively, axial load and flexural members resulting from usual elastic analysis of the structure are divided to it to provide a basis for design of member.

13.9.2 Load-carrying capacity reduction method shall not be applied to compression member with two hinged ends.

13.9.3 Load-carrying capacity reduction method may be used for braced compression members only if $\frac{\ell_u}{r} < 80$ is satisfied.

13.9.4 Load-carrying capacity reduction method may be used for sway compression member if $K' \frac{\ell_u}{r} < 40$ is satisfied and that the percentage of negative reinforcement in beams connected to both ends of the member is not less than 0.01.
Effective length factor is calculated by Eq. (13-20):

$$\mathbf{k}' = 0.78 + 0.22\,\Psi_m \geq 1 \qquad (13\text{-}20)$$

Where ψ_m is the average of the two ψ values at two ends of the compression member.

13.9.5 Load-carrying reduction factor in nonsway compression members is given as follows:

a) When compression member is bent in double curvature:

$$R = 1.32 - 0.006\frac{\ell_u}{r} \leq 1 \qquad (13\text{-}21)$$

b) When compression member is bent in single curvature:

if $\frac{e}{h} \leq 0.1$

$$R = 1.23 - 0.008\frac{\ell_u}{r} \leq 1 \qquad (13\text{-}22)$$

if $\frac{e}{h} > 0.1$

$$R = 1.07 - 0.008\frac{\ell_u}{r} \leq 1 \qquad (13\text{-}23)$$

In the above equations, e is the eccentricity of compressive load in the member.

c) When axial load acting on the compression member is less than $0.1 f_c A_g$, factor R may be considered greater than the values given by (a) and (b). R factor may be obtained from linear interpolation between R=1 for zero axial force and R as calculated by the above articles for axial force equal to $0.1\,f_c\,A_g$.

13.9.6 Load-carrying reduction factor in sway compression member is calculated by Eq. (13-24):

$$R = 1 - 0.008K'\frac{\ell_u}{r} \qquad (13\text{-}24)$$

❐ 13.10 Minimum eccentricity of load

13.10.1 When axial load and flexural moment resulting from usual elastic analysis in compression member is so that eccentricity of load acting on them is less than the following:

$$e_{min} = 15 + 0.03h \qquad (13\text{-}25)$$

e_{min} as the minimum eccentricity of load shall be incorporated in calculation of flexural moment of member and its slenderness effect, both. This eccentricity shall be applied separately for flexure about both principal axes of the section.

13.10.2 When the magnification method for flexural moments are applied for calculation of slenderness effect, in accordance with 13.8, the flexural moment calculated on the basis of e_{min}, replaces M_{2b} in Eqs. (13-14) and (13-15) for sway compression member . $\frac{M_{1b}}{M_{2b}}$ in Eq. (13-10) is accordingly calculated by the following criteria:

a) When load eccentricity at two ends of compression member is not equal to zero, $\frac{M_{1b}}{M_{2b}}$ is given by their real values.
b) When load eccentricity at two ends of compression member is equal to zero, $\frac{M_{1b}}{M_{2b}}$ is taken equal to 1.

❒ 13.11 Slender effect on compression members subjected to biaxial flexure

13.11.1 Slender effect for compression members subjected to biaxial flexure about both principal axes shall be calculated separately based on the conditions of restraint corresponding to that axis.

❒ 13.12 Flexural moment magnification in flexure members connected to compression members

13.12.1 Flexural members connected to compression member in sway frames shall be designed for the sum of magnified moments at each end of compression member.

13.12.2 For braced frames, when slenderness effect is computed in accordance with 13.9, using load-carrying capacity reduction method, flexural moment to be incorporated in design of flexural members connected to both ends of compression member, shall be given by the Eq. (13-26).

$$M_\ell = N_\ell e_s \left[\frac{1 - \dfrac{N_\ell}{N_o}}{R - \dfrac{N_\ell}{N_O}} \right] \tag{13-26}$$

N_o in the Eq. (13-26) is calculated by:

$$N_o = 0.85 \phi_c f_c (A_g - A_{st}) + \phi_s f_y A_{st} \tag{13-27}$$

CHAPTER FOURTEEN

DEFLECTION AND CRACKING

❒ 14.0 Notation

A = effective tension area of concrete divided by total number of bars or wires. Effective tension area of concrete is the area with its center coinciding on the centroid of tension reinforcement. When different area bars exist, their total number is taken as their total area divided by the largest area of reinforcement (bar or wire used), (mm^2).

A_{sk} = cross-sectional area of skin reinforcement, (mm^2), refer to 14.3.1.5.

d = effective depth of section.

d_c = thickness of concrete cover measured from extreme tension fiber to the center of bar or wire located closest thereto, mm.

f_c = specified compressive strength of concrete, MPa.

f_r = modulus of rupture of concrete, MPa.

f_s= calculated stress in tension reinforcement at service loads, MPa.

f_y= specified yield strength of nonprestressed reinforcement (f_{yk}), MPa. For simplicity, the letter K has been omitted from subscript in this chapter.

h= overall thickness of member, mm.

I_{cr}= moment of inertia of cracked section, considering reinforcement effect, mm^4.

I_e= effective moment of inertia at section, mm^4.

I_g = moment of inertia of uncracked section about centroidal axis, neglecting reinforcement effect.

ℓ = effective span length (refer to 10.3.4.1), mm.

ℓ_n= clear span length in long direction of two-way slabs, measured face-to-face of supports or support beams, mm.

M_a = maximum flexural moments in service state, mm.

M_{cr} = cracking moment (refer to 14.2.2.2), N.mm.

y_t = distance from neutral fiber at centroidal axis of uncracked section, neglecting reinforcement, to extreme fiber in tension,

w= cracking width, mm.

α = ratio of flexural stiffness of beam section, as defined in 15.2.7, to flexural stiffness of slab bounded laterally by centerlines of adjacent panels (is any) on each side of beam.

α_m = average value of α for all beams on edges of a panel.

β = ratio of clear spans in long to short direction of two- way slabs.

ξ = time-dependent factor for sustained load, refer to 14.2.2.3.

λ = multiplier for additional long-term deflection, refer to 14.2.2.3.

ρ' = reinforcement ratio for nonprestressed compression reinforcement.

❒ 14.1 Scope

14.1.1 Provisions of this chapter shall apply to control design of member subject to flexure in limit state of serviceability. These requirements include procedures to compute deformations or deflections, cracking and related limitations.

14.1.2 Loads and load-effects used in this chapter to calculate deflection and cracking are service loads in accordance with 10.6.2.

❒ 14.2 Deformations or deflections

14.2.1 General

14.2.1.1 Members subject to flexure shall be designed to have adequate stiffness to limit induced deformations that adversely effect their strength and serviceability.

14.2.1.2 In determining member stiffness, concrete cracking effects as well as reinforcement effects shall be taken into account. When more accurate analysis methods or laboratory methods are not used, requirements stated in 14.2.2 shall be assumed adequate.

14.2.1.3 Where deflections are to be computed, in addition to deflection due to immediate loading named as "immediate deflection", the effect of sustained loads shall also be considered.

14.2.2 Computation of deflection for beams and one-way slabs

14.2.2.1 Immediate deflection of members shall be computed by usual structural analysis methods, or by formulas developed for linear elastic deflection of members. Modulus of elasticity of concrete, E_c, shall be based on Eq. (10-1), and the effective moment of inertia of member shall be in accordance with requirements of 14.2.2.2.

14.2.2.2 Effective moment of inertia for members shall be based on section specifications and the intensity of cracking, as follows:

a) For simply supported and cantilevered members, effective moment of inertia shall be permitted to be obtained from the following equation at midspan for simple support spans, and at support for cantilevers respectively, based on section specifications:

$$I_e = I_{cr} + (I_g - I_{cr})(\frac{M_{cr}}{M_a})^3 \qquad (14\text{-}1)$$

In this formula, cracking moment at the section, M_{cr}, is calculated as follows:

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (14\text{-}2)$$

Modulus of rupture for concrete is obtained using the following relation:

$$f_r = 0.6\sqrt{f_C} \qquad (14\text{-}3)$$

I_c shall never be taken greater than I_g.

b) For continuous members, effective moment of inertia shall be permitted to be taken as the average of values obtained for effective moment of inertia of member, based on critical section at midspan and at support, using the formula (14-1), provided that calculated effective moment of inertia at critical section of the midspan is incorporated in the averaging process. For prismatic members, effective moment of inertia is taken as the value obtained at midspan of the critical section.

14.2.2.3 Unless values are obtained by a more comprehensive analysis, additional "long-term deflection" shall be determined by multiplying the immediate deflection caused by sustained load considered, by the factor

$$\lambda = \frac{\xi}{1 + 50\rho'} \qquad (14\text{-}4)$$

Where ρ' is the ratio of compression reinforcement at midspan for simple and continuous spans, and at support for cantilever members. The time-dependent factor, ξ, shall be taken equal to:

- 5 years or more	2.0
- 12 months	1.4
- 6 months	1.2
- 3 months	1.0

14.2.3 Computing deflection in two-way slabs

14.2.3.1 For two-way slabs, immediate deflection may be computed using usual methods of plate analysis and the formulas derived based on their linear elastic behavior. In these methods and formulas, modulus of elasticity of concrete, E_c, and the effective moment of inertia of the slab shall be based on the Eqs. (10-1) and (14-1), respectively. Provided that the results are confirmed by laboratory tests, other values of E_c and I_c may also be used to compute immediate deflection.

14.2.3.2 For two-way slabs, long-term additional deflection shall be in accordance with requirements of 14.2.2.3.

14.2.4 Deflection limitations for beams and slabs

14.2.4.1 Deflection induced in beams and slabs shall not exceed the values specified in table 14.2.4.1.

14.2.4.2 Observance of limitations no. 2 and 4 from table 14.2.4.1 shall be considered adequate for usual residential, administrative and commercial buildings.

14.2.4.3 For conventional buildings subjected to usual loadings on beams and on one-way slabs with depth or thickness more than the values stipulated in the table 14.2.4.3, calculation of deflection need not be done, provided that the beams or slabs are not supporting or attached to non-structural elements such as partitions, and that they are damaged by large deflections.

Table (14.2.4.1): Deflection limitations for beams and slabs

<table>
<tr><th>Member types</th><th>Deflection to be considered</th><th>Deflection limitation</th><th>Conside-rations</th></tr>
<tr><td>1- Flat roofs not supporting or attached to non-structural elements likely to be damaged by large deflections</td><td>Immediate deflection due to live load L</td><td>$\frac{\ell}{180}$</td><td>-</td></tr>
<tr><td>2- like above regarding slabs</td><td>Immediate deflection due to live load L</td><td>$\frac{\ell}{360}$</td><td>-</td></tr>
<tr><td>3- roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections</td><td rowspan="2">That part of the total deflection occurring after attachment of non-structural elements (sum of the long- term deflection due to all sustained loads and the immediate deflection due to any additional live load).
Note 3</td><td>$\frac{\ell}{480}$</td><td>Note 1</td></tr>
<tr><td>4- Roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections</td><td>$\frac{\ell}{240}$</td><td>Note 2</td></tr>
</table>

Note 1- Limit may be reduced if adequate measures are taken to prevent damage to supported or attached elements.

Note2-Deflection shall not exceed the tolerance provided for nonstructural elements. Specified limit may be applied to the total deflection minus camber, if camber is provided.

Note3-Long additional deflection shall be determined in accordance with requirements of 14.2.2.3, but may be reduced by the amount of long-term additional deflection calculated to occur before attachment of non-structural elements. The remainder value shall be incorporated in calculation of considered deflection.

Table (14.2.4.3): Minimum depth or thickness of beams or one-way slabs

Member	Simply supported	One end continuous	Both ends continuous	Cantilever
Beams or ribbed one-way slabs	$\frac{\ell}{16}$	$\frac{\ell}{18.5}$	$\frac{\ell}{21}$	$\frac{\ell}{8}$
Solid one-way slabs joist and block roofing	$\frac{\ell}{20}$	$\frac{\ell}{24}$	$\frac{\ell}{28}$	$\frac{\ell}{10}$

Note- The above table is prepared for S400 type reinforcement. For other types of reinforcement, the values shall be multiplied by $(0.4+\frac{f_y}{670})$.

14.2.4.4 For two-way slab systems designed in accordance with the provisions of chapter 15 and conforming to the requirements of 15.7.1.3; calculation of deflection need not be done when slab thickness is greater than the values specified in 14.2.4.5 and 14.2.4.6.

14.2.4.5 For slabs without interior beams spanning between supports, the minimum thickness shall be in accordance with provisions table 14.2.4.5 and shall not be less than the following values:

a) Slabs without drop panels as defined in 15.4.4.2 and 15.4.4.3 125mm

b) Slabs with drop panels as defined in 15.4.4.2 and 15.4.4.3 100mm

Table (14.2.4.5): Minimum thickness of flat slabs without interior beams

<table>
<tr><th rowspan="3">Type Of Steel</th><th colspan="3">Without drop panels</th><th colspan="3">With drop panels</th></tr>
<tr><th colspan="2">Exterior panels</th><th>Interior panels</th><th colspan="2">Exterior panels</th><th>Interior panels</th></tr>
<tr><th>Without edge beams</th><th>With edge beams</th><th>– – –</th><th>Without edge beams</th><th>With edge beams</th><th>– – –</th></tr>
<tr><td>S300</td><td>$\frac{\ell_n}{33}$</td><td>$\frac{\ell_n}{36}$</td><td>$\frac{\ell_n}{36}$</td><td>$\frac{\ell_n}{36}$</td><td>$\frac{\ell_n}{40}$</td><td>$\frac{\ell_n}{40}$</td></tr>
<tr><td>S400</td><td>$\frac{\ell_n}{30}$</td><td>$\frac{\ell_n}{33}$</td><td>$\frac{\ell_n}{33}$</td><td>$\frac{\ell_n}{33}$</td><td>$\frac{\ell_n}{36}$</td><td>$\frac{\ell_n}{36}$</td></tr>
</table>

Note-1 Drop panels or column caps presented in the above table shall be in accordance with definitions of 15-4-4-2 and 15-4-4-3.
Note-2 Edge beams with stiffness, α, shall be at least equal to 0.8.

14.2.4.6 For slabs being supported on all sides by beams, and having a ratio of long to short span less than 2, the minimum thickness is defined as follows:

a) For slabs with α_m less than or equal to 0.2, the provisions of 14.2.4.5 shall apply

b) For slabs with ratio α_m greater than 0.2 and less than 2, the following formula shall apply

$$h = \frac{\ell_n (800 + 0.6 f_y)}{36000 + 5000\beta(\alpha_m - 0.2)} \quad (14\text{-}5)$$

Slab thickness shall be taken not less than 125mm

c) For slabs with ratio α_m greater than or equal to 2, the following formula shall apply:

$$h = \frac{\ell_n (800 - 0.6 f_y)}{36000 + 9000\beta} \quad (14\text{-}6)$$

Slab thickness shall not be less 90mm.

14.2.4.7 For beams in which span-to-depth ratio limitations of the table 14.2.4.3 do not apply, or in columns where reinforcement percentage is more than 4.5 percent, compressive stress is limited to $0.45f_c$ for unfactored long-term loads (outside splice location), and to $0.6f_c$ for service loads.

❒ 14.3 Cracking

14.3.1 General

14.3.1.1 For members subject to flexural, tensile reinforcement and the way in which it is distributed shall be such that cracks induced in the member due to tension, does not have adverse effect on its strength and serviceability.

14.3.1.2 For beams and one-way slabs, crack width may be determined in accordance with 14.3.2. Observance of the limitations stated in 14.3.3 is required for these members.

14.3.1.3 For two-way or flat slabs, crack width need not be calculated and mere observance of the requirements related to temperature and shrinkage reinforcement in accordance with 8.7 shall be adequate.

14.3.1.4 When flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width, or a width equal to one-tenth the span, whichever is smaller. If the effective flange width exceeds one-tenth the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

14.3.1.5 If the effective depth d of a beam exceeds 600mm, longitudinal skin reinforcement A_{sk} shall be uniformly distributed along both side faces of the member for a distance $\frac{d}{2}$ higher than tensile reinforcement. The area of skin reinforcement A_{sk}, regardless of its dimension aspects shall be $(d-750) \geq 150\ mm^2$ per unit height in one side.

Total area of longitudinal skin reinforcement in both faces need not exceed one-half of the tensile reinforcement in the beam. The maximum spacing of the skin reinforcement shall not exceed $\frac{d}{6}$ or 300mm. Contribution of such reinforcement to flexural strength of the beam shall also be considered in calculations.

14.3.2 Crack width computation

14.3.2.1 Unless more exact analysis is carried out, crack width for beams and for one-way slabs is calculated by:

$$w = 13 \times 10^{-6} f_s \sqrt[3]{d_c A} \qquad (14\text{-}7)$$

14.3.2.2 Unless a more exact analysis is made, $0.6 f_y$ may be substituted for f_s in Eq. (14-17) to determine stress in the tensile reinforcement.

14.3.3 Crack width limitation

14.3.3.1 For beams and one-way slabs, crack width shall not exceed 0.35mm.

14.3.3.2 Provisions of 14.3.3.1 are not sufficient for structures subject to very aggressive exposure or designed to be watertight. For such structures, special investigations and precautions are required.

CHAPTER FIFTEEN

TWO-WAY SLAB SYSTEM

❒ 15.0 Notations

A_{sb} = minimum area of the continuous lower reinforcement, mm^2.

b_1 and b_2 = dimensions of critical section for punching shear at a distance $\frac{d}{2}$ from support edge. (b_1 in the direction of longitudinal axis of the slab-beam strip and b_2 in its transversal direction), mm.

b_w = total width of the beam web, mm.

c_1 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm.

c_2 = size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to the span for which moments are being calculated, mm.

C = torsional stiffness factor, mm^4.

C_A^- = negative moment coefficient at short span of the slab.

C_{AD}^+ = positive moment coefficient for dead load at short span of the slab.

C_{AL}^+ = positive moment coefficient for live load at short span of the slab.

C_B^- = negative moment coefficient at long span of the slab.

C_{BD}^+ = positive moment coefficient for dead load at long span of the slab,

C_{BL}^+ = positive moment coefficient for live load at long span of the slab.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm.

d_b = reinforcement size, mm.

E_{cb} = modulus of elasticity of beam concrete, MPa (N/mm^2).

E_{cs} = modulus of elasticity of slab concrete, MPa (N/mm^2).

f_y = specified yield strength of nonprestressed reinforcement (f_{yk}), MPa (N/mm^2). For simplicity, the letter k has been omitted from subscript, in this chapter.

h_b = overall thickness of beam.

h_s = overall thickness of slab.

I_b = moment of inertia about centroidal axis of gross section of beam, mm^4.

I_s = moment of inertia of slab in the slab-beam strip about centroidal axis of gross section of slab, mm^4.

I_{sb} = moment of inertia of slab-beam in the slab-beam strip, about centroidal axis of gross section of a complete slab-beam system, mm^4.

K_c = column flexural stiffness, N.mm.

K_{cc} = equivalent column flexural stiffness, N.mm.

K_t = torsional stiffness of the torsion member, N.mm.

K_{ta} = torsional stiffness of the member, according to Eq. (15-4).

ℓ_A = clear length of short span in two-way slab, mm.

ℓ_B = clear length of long span in two-way slab, mm.

ℓ_n = length of clear span in direction that moments are being determined, measured face-to-face of supports, mm.

ℓ_1 = length of span in direction that moments are being determined, measured center-to-center of support, mm.

ℓ_{1n} = length of clear span in direction that moments are being determined, mm.

ℓ_2 = length of span transverse to ℓ_1, measured center-to-center of support, mm.

ℓ_d = development length, mm.

m = ratio of short span length to the length of long span in two-way slab, $\dfrac{\ell_A}{\ell_B}$.

M^+_{AD} = positive moment for dead load in the short span of slab, N.mm.

$M^-_{A(D+L)}$ = negative moment for dead and live loads in the short span of slab, N.mm.

M^+_{AL} = positive moment for live load in the short span of slab, N.mm.

M^+_{BD} = positive moment for dead load in the long span of slab, N.mm.

$M^-_{B(D+L)}$ = negative moment for dead and live load in the long span of slab, N.mm.

M^+_{BL} = positive moment for live load in the long span of slab, N.mm.

M_o = total factored static moment, N.mm.

M_u = ultimate flexural moment at section, N.mm.

M_{uf} = fraction of unbalanced moment transferred by flexure, N.mm.

M_{uv} = fraction of unbalanced moment transferred by shear, N.mm.

w_d = factored dead load per unit area, MPa.

w_D = dead load per unit area, MPa.

$w_{(D+L)}$ = sum of dead and live load per unit area, MPa.

w_L = live load per unit area, MPa.

w_s = vertical load per unit area, MPa.

w_u = ultimate load per unit area, MPa.

x = shorter overall dimension of rectangular part of the cross section, mm.

y = longer overall dimension of rectangular part of the cross section, mm.

α = ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam. ($\alpha = E_{cb}I_b/E_{cs}I_s$).

α_c = ratio of flexural stiffness for columns above and below the slab to the sum of flexural stiffness of beams and slabs at joints taken in the direction of spans for which moments are being determined.

α_{min} = minimum value of α_c .

$\alpha_1 = \alpha$ in direction of ℓ_1 .

$\alpha_2 = \alpha$ in direction of ℓ_2

β_a = ratio of dead load per unit area to live load per unit area with zero load coefficients.

β_t = ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center–to-center of supports ($\beta_t = \dfrac{E_{cb}C}{2E_{cs}I_s}$) .

ϕ_s = partial safety factor of reinforcement.

❒ 15.1 Scope

15.1.1 Provisions of this chapter shall apply for design of slab systems in which slabs are subject to flexure in two directions and they are reinforced in those directions, with or without beams between supports.

15.1.2 For a slab system supported by columns or walls, dimensions c_1 and c_2 , and the clear span length ℓ_n shall be based on an effective support area defined by the intersection of the bottom surface of the slab, or of the drop panel, if any, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and capital or bracket and are oriented no less than 45 degree to the axis of the column.

15.1.3 Waffle slabs with or without filler elements between ribs or joists in two directions are included within the scope of this chapter and are subject to the requirements of this chapter.

❒ 15.2 Definitions

Slab system
Slab system is referred to all planar structural members, with or without beam, subjected to loads in the direction orthogonal to their middle plates. Usual slab systems include slab-beam, flat slab, flat plates and waffle slab.

Equivalent frame
Refer to definition in 15.6.3

Lap strip
Lap strip is referred to that strip of slab system on both sides of columns located in the same row on the plan and is limited to centroidal axes of the adjacent panels.

Column strip
Column strip is a design strip with a width on each side of a column centerline equal to $0.25\ell_2$ or $0.25\ell_1$, whichever is less. Column strip includes beams (if any) between columns.

Middle strip
A design strip bounded by two column strips.

Side strip

A strip of slab, in slab-beam, located inside column strip on each side of beam.

Beam in slab–beam system

A beam, in slabs, includes beam web and that portion of slab on each side of the beam extending a distance equal to the 45 degree inclined projection of that portion of beam web above or below the slab, whichever is greater in depth, provided the width of extended portion does not exceed four times the slab thickness.

Slab Panel

Slab panel is a portion of slab system bounded by column, beam, or support wall centerlines on all sides.

❒ 15.3 Design procedures

15.3.1 Four design procedures are recommended in this code for slab systems, each with its own special application, regarding limits to be satisfied for each case. A slab system may be designed by any procedure, provided that conditions of equilibrium in forces and compatibility of deformations are satisfied and load-carrying component of the slab in each section is not less than load-effects acting on it and that all serviceability conditions including limits on deflections, are met.

15.3.2 The four recommended design procedures are as follows:

a) “ Equivalent frame” method
b) “ Direct” method
c) “Flexural moment factors” method

d) "Plastic" method

Methods (a) and (b) are used for analysis and design of combination of slabs and support beams (if any), and the methods (c) and (d) are used separately for analysis and design of slabs. Details of these procedures are given in 15.6 through 15.9.

❒ 15.4 General principles for design of slabs

15.4.1 Slab thickness

15.4.1.1 In determining slab thickness within scope of this chapter, special requirements of limit states of serviceability shall be satisfied.

15.4.2 Design for flexure and shear

15.4.2.1 Slabs and bearing beams shall be designed for flexure and shear load-effects in each section, according to principles of chapter 11 and 12. Requirements for minimum tensile reinforcement stated in 11.5.2 need not be met. Reinforcement placement requirements for slabs are given in 15.5.

15.4.2.2 Design for load transmission (as shear or flexure) from slab to column or to bearing beam shall be based on provisions of chapter 12.

15.4.3 Flexural moment transmission in slab-column connection

15.4.3.1 When gravity load, wind, or earthquake cause transfer of unbalanced flexural moment, M_u, between slab (without beam) and column, a fraction of M_{uf}, shall be transferred by flexure and the remainder, M_{uv}, shall be transferred by eccentricity of shear force about column in slab. M_{uf} is calculated by:

$$M_{uf} = \frac{M_u}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \qquad (15\text{-}1)$$

15.4.3.2 A fraction of the unbalanced flexural moment, M_{uf}, shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thickness (1.5 h) outside opposite faces of the column or capital. Required reinforcement to carry the flexural moment shall be placed in the same slab width.

15.4.3.3 Design for that fraction of flexural moment transferred by eccentricity of shear about slab or drop panel, M_{uv}, shall be based on requirements of 12.17.5 .

15.4.4 Drop panels

15.4.4.1 When a drop panel is used to reduce amount of negative moment reinforcement over the column of a flat plate or flat slab, size of drop panel shall be in accordance with 15.4.4.2 through 15.4.4.4 .

15.4.4.2 Drop panel shall extend in each direction from centerline of support a distance not less than one-sixth the span length measured from center–to-center of supports in that direction.

15.4.4.3 Drop panel thickness shall not be less than one-fourth the slab thickness.

5.4.4.4 In computing required negative reinforcement, the thickness of the drop panel shall not be assumed greater than one-quarter the distance from edge of drop panel to edge of column or column capital.

15.4.5 Openings in slab systems

15.4.5.1 Openings of any size shall be permitted in slab systems if shown by analysis that the design strength is at least equal to the required strength and that all serviceability conditions, including the specified limits on deflections, is met.

15.4.5.2 As an alternative to special analysis, openings shall be permitted in slab 15.4.5.3 through 15.4.5.5 An amount of reinforcement equivalent to that interrupted by the opening shall be added on the sides of the opening.

15.4.5.3 In the area common to two intersecting middle strips, any opening with any size may be provisioned.

15.4.5.4 In the area common to two intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings.

15.4.5.5 In area common to one column strip and one middle strip not more than one–quarter of the reinforcement in either strip shall be interrupted by openings.

15.4.5.6 When an opening is created in the slab system, shear design requirement shall apply in accordance with 12.17.4.

15.4.5.7 In slab–beam plates, openings shall not intersect beams, unless satisfactory reasons are presented.

❐ 15.5 Reinforcement placement in slabs

15.5.1 General principles of reinforcement placement

15.5.1.1 Area of reinforcement in each direction for different section of slab shall be determined from flexural moments at those critical sections, but shall not be less than the corresponding area for temperature and shrinkage reinforcement, in accordance with 8.7.

15.5.1.2 Spacing of flexural reinforcement at critical sections of slabs shall not exceed two times the slab thickness, nor 350 mm, except for portions of slab area of cellular or ribbed construction. Minimum reinforcement in waffle slabs over cellular spaces and pockets shall be in accordance with 8.7.

15.5.1.3 Positive flexural reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150mm in spandrel beams, columns or walls.

15.5.1.4 Negative flexural reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, to be developed at face of support according to provisions of chapter 18.

15.5.1.5 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement perpendicular to the edge shall be permitted within the slab.

15.5.2 Special details of reinforcement in slabs with beams

15.5.2.1 For slab-beam systems in which, α is greater than unity, special top and bottom reinforcement shall be provided at exterior corners in accordance with 15.5.2.2 through 15.5.2.5.

15.5.2.2 The special reinforcement in both top and bottom of slab in unit width, shall be sufficient to resist a flexural moment equal to maximum positive flexural moment in the slab.

15.5.2.3 The special reinforcement shall be placed in a band parallel to the 45deg diagonal in the top of the slab and a band perpendicular to the diagonal in the bottom of the slab.

15.5.2.4 The special reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

15.5.2.5 The special reinforcement shall be placed in two layers prescribed in 15.5.2.3 or in two curtains parallel to the sides of the slab in either the top or bottom of the slab.

15.5.3 Special details of reinforcement in slabs without beams
15.5.3.1 For reinforcement placement in flat plates and flat slabs, in addition to 15.5.1, requirement of 15.5.3.2 through 15.5.3.7 shall apply.

15.5.3.2 In determining bending point or intersection point of reinforcement, minimum extensions specified in Fig. 15.5.3.2 shall be required.

15.5.3.3 Where adjacent spans are unequal, extensions of negative reinforcement beyond the face of support as prescribed in Fig. 15.5.3.2 shall be based on requirements of the longer span.

15.5.3.4 Bending positive reinforcement to be extended as negative reinforcement is only permitted if in provision of minimum extensions as prescribed in Fig. 15.5.3.2, bends greater than 45 degree shall be considered.

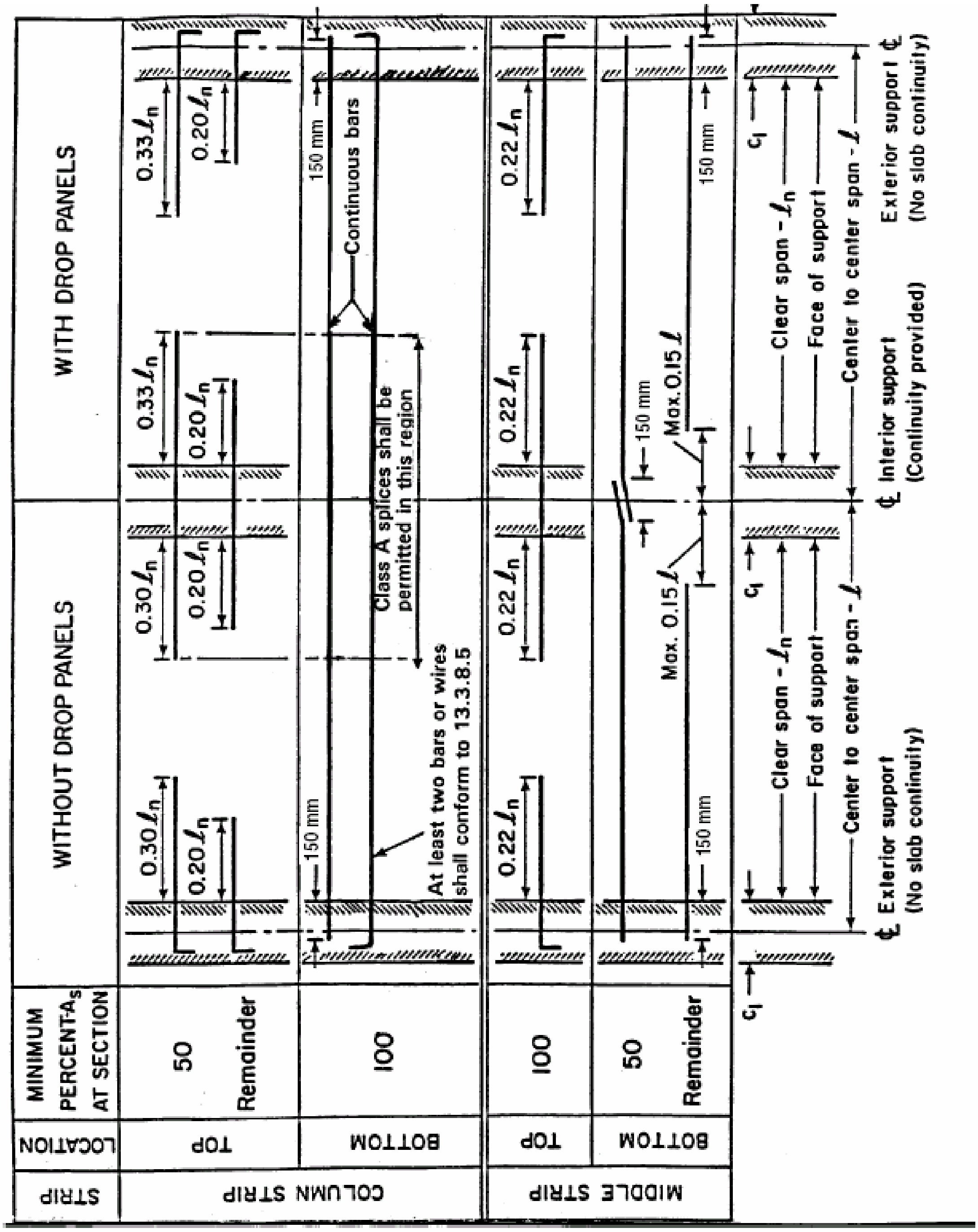
15.5.3.5 For slabs in frames not braced against side-sway, location of bend or intersection of bars shall be computed by analysis but shall not be less than those prescribed in Fig. 15.5.3.2.

15.5.3.6 All bottom bars within the column strip in each direction shall be continuous or spliced with class "A" splices located in accordance with 18.14.2. At least two column strip bottom bars in each direction shall pass within the column core and shall be anchored at exterior supports. If the reinforcement splice is located outside the column core, splice length shall be at least 2 ℓ_d.

15.5.3.7 Overall section area of reinforcement in 15.5.3.6, shall not be less than the value obtained from the following formula:

$$\mathrm{A_{sb}} = \frac{0.5\mathrm{w_s}\ell_2\ell_{1n}}{\phi_s f_y} \qquad (15\text{-}2)$$

Where w_s is the gravity load acting on the slab in service conditions and its value is limited to two times the dead load.


STRIP
LOCATION
MINIMUM PERCENT A_s AT SECTION
COLUMN STRIP
MIDDLE STRIP
TOP
BOTTOM
50
Remainder
100
WITHOUT DROP PANELS
WITH DROP PANELS
$0.30\ell_n$
$0.20\ell_n$
$0.33\ell_n$
$0.22\ell_n$
150 mm
Continuous bars
Class A splices shall be permitted in this region
At least two bars or wires shall conform to 13.3.8.5
Max. 0.15ℓ
c_1
Clear span - ℓ_n
Face of support
Center to center span - ℓ
℄ Exterior support (No slab continuity)
℄ Interior support (Continuity provided)
Exterior support ℄ (No slab continuity)

❐ 15.6 Equivalent frame method

15.6.1 Scope

15.6.1.1 Equivalent frame method may be used to design systems, for gravity loads, in which slabs and beams (if any) between supports and supporting columns form orthogonal frames.

15.6.1.2 Results of gravity load analysis may be combined with the results of the lateral load analysis to be used in design, along with proper assumptions. Unless proper assumptions are used to combine the results of gravity and lateral load analyses, load-effects due to lateral loads may only be transferred to beams and columns, but in the absence of beams, loads may be transferred to column strips and columns.

15.6.2 Design procedure

15.6.2.1 Design of slab systems and beams (if any) between supports and columns or walls by equivalent frames subjected to gravity loads, shall be based on assumptions in 15.6.3 through 15.6.6.

15.6.2.2 Flexural moments obtained from equivalent frame analysis shall be distributed to beams and slabs in accordance with requirements of 15.6.9.

15.6.2.3 Design of slabs and beams for shear and flexure shall be based on the values obtained from 15.6.2.2.

15.6.3 Equivalent frame

15.6.3.1 The structure shall be considered to be made up of equivalent frames, normal to each other, on column lines taken longitudinally or transversely through the building.

15.6.3.2 Each frame shall consist of a row of columns or supports and slab-beam strips including beams between columns and walls and a portion of the slab width, bounded laterally by the centerline of panels on each side of the centerline of columns or supports.

15.6.3.3 A frame is analyzed for the loads acting on its slab–beam strip.

15.6.3.4 Effects of torsional stiffness of slab-beam strips located transversal to the equivalent frame, on torsional stiffness of columns and support walls in the equivalent frame, shall be assumed in the calculations, considering torsional members in accordance with 15.6.5 and 15.6.6.

15.6.3.5 In frames adjacent and parallel to an edge, slab-beam strip consists of beams between columns or walls and the portion of slab width bounded by centerlines of adjacent panels.

15.6.3.6 Each equivalent frame may be analyzed as a complete frame for gravity loads, or as a sub-frame consisting of slab-beam strip in each story with columns above and below that story, in accordance with requirements of 10.3.9.2 and 10.3.9.3.

15.6.3.7 Where metal column capitals are used, it shall be permitted to take account of their contributions to stiffness and resistance to moment and to shear.

15.6.3.8 Neglecting the change in length of columns and slabs due to direct stress, and deflections due to shear, shall be permitted.

15.6.4 Moment of inertia of members in equivalent frame

15.6.4.1 Moment of inertia of all member sections in the equivalent frame may be determined based on uncracked concrete section.

15.6.4.2 Variations in moment of inertia due to changes in the dimensions of beams, slabs and columns or walls shall be incorporated in calculations.

15.6.5 Torsional members

15.6.5.1 Torsional members, as defined in 15.6.3.4 shall be assumed to have a length equivalent to the width slab-beam strip, and that their cross section is not less than the following:

a) A portion of slab having a width equal to that of the column, bracket, or column capital in the direction of the equivalent frame considered
b) For slab-beam system, a portion of slab specified in "a)" plus web portion of the transverse beam above and below the slab
c) Transverse beam normal to the equivalent frame considered, as defined in 15.2

15.6.5.2 Dividing section area into a number of separate rectangular parts and using the equation below, torsional stiffness coefficient of the section, C, may be calculated as follows:

$$C = \frac{1}{3}\sum(1-0.63\frac{x}{y})x^3y \qquad (15\text{-}3)$$

x and y are length and width of a rectangle, respectively. Division of the member section shall be such that C has the greatest possible value.

15.6.5.3 Torsional stiffness of the member, K_{ta}, is determined by:

$$K_{ta} = (\frac{I_{sb}}{I_s})K_t \qquad (15\text{-}4)$$

$$K_t = \sum \frac{9E_{cs}C}{\ell_2 (1 - \frac{c_2}{\ell_2})^3} \qquad (15\text{-}5)$$

I_{sb} is the moment of inertia of the slab-beam combination in the slab-beam strip and, I_s is the moment of inertia of the lone slab in the slab-beam strip, each relative to its middle fiber. Parameters c_2 and ℓ_2 relate to spans transversal to the equivalent frame on both sides.

15.6.6 Column flexural stiffness in equivalent frames

15.6.6.1 Incorporation of the effects torsional members in each joint of the equivalent frame may involve considering a column with equivalent flexural stiffness, K_{ec}, computed from the following equation:

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_{ta}} \qquad (15\text{-}6)$$

Where $\sum K_c$ is the sum of flexural stiffness of the columns above and below, and K_{ta} is torsional stiffness of the torsional member.

15.6.7 Alternate loading

15.6.7.1 When loading pattern for live loads is specified, the equivalent frame shall be analyzed and designed for that load.

15.6.7.2 When live load is variable but is less than three-quarters of the dead load, and less than 5000 (N/mm^2), or the nature of live load is such that all panels are loaded simultaneously, the frame shall be designed assuming that maximum factored live loads occur simultaneously at all sections.

15.6.7.3 When none of the conditions specified in 15.6.7.1 and 15.6.7.2 is satisfied, alternate loading shall be required. Accordingly, it shall be permitted to assume that maximum positive ultimate flexural moment near midspan of a panel occurs with three-quarters of the full factored live load on the panel and on the alternate panels. It shall also be permitted to assume that maximum negative factored flexural moment in the slab at a support occurs with three-quarters of the full live load on adjacent panels only.

15.6.7.4 Flexural moments used in the design of members shall not be less than those occurred with full factored live load on all panels.

15.6.8 Flexural moments in slab-beam strip

15.6.8.1 Maximum negative flexural moments at internal supports, slab-beam strip, and middle strips, shall be taken equal to the flexural moment in the critical section at face of rectilinear supports, but not greater than 0.175 ℓ_1 from center of a column.

15.6.8.2 At exterior supports with column capital or drop panel, maximum negative flexural moment at the span perpendicular to slab edge shall be taken equal to the flexural moment at a section with distance from face of supporting element equal to one-half the horizontal projection of column capital or drop panel beyond face of supporting element.

15.6.8.3 Circular or rectangular polygon shaped supports shall be treated as square supports with the same area. The support face shall be measured from the face of the square.

15.6.8.4 When slab system is within limitations of 15.7.1 with regards to application of direct design method, absolute sum of the positive, and average negative flexural moments in each panel of the equivalent frame may be reduced to the preliminary value obtained form Eq. (15-8) and then correct the corresponding positive and negative flexural moments appropriately.

15.6.9 Distribution of flexural moments in slab-beam strip

15.6.9.1 Flexural moments calculated in slab-beam strip may be distributed between column and middle strips and then between slab and beam, in accordance with conditions of 15.6.9.2 and 15.6.9.3. The above requirements concerning slabs that act integrally with four side-beams are applied only if the ratio of beam stiffness in two orthogonal directions is satisfied within the following equation. If the above equation does not hold true for slabs, a more exact procedure for distribution of flexural moments shall be carried out:

$$0.2 \le \frac{\alpha_1 \ell_2^2}{\alpha_1 \ell_1^2} \le 5 \qquad (15\text{-}7)$$

15.6.9.2 Flexural moments in the slab–beam strip is distributed between column strip and middle strip, in accordance with the following conditions:

a) Percentages of flexural moments in slab-beam strip that are proportioned to column strip are shown in the table 15.6.9.2. Difference between these moments and the moments in each section of slab-beam strip is the share of middle strip.
b) Where supports consist of columns or walls extending for a distance equal to or greater than three-quarters the span length ℓ_2, negative flexural moments in the slab-beam strip shall be considered to be uniformly distributed across ℓ_2.
c) Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.
d) A middle strip adjacent to and parallel with an edge supported by a wall shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

Table (15.6.9.2): Share of column strip from flexural moment in slab-beam strip

<table>
<tr><td colspan="3"></td><td colspan="3">ℓ_2 / ℓ_1</td></tr>
<tr><td colspan="3"></td><td>0.5</td><td>1</td><td>2</td></tr>
<tr><td rowspan="2">Percentage of negative flexural moment at the interior supports</td><td>$\alpha_1 \ell_2 / \ell_1 = 0$</td><td></td><td>75</td><td>75</td><td>75</td></tr>
<tr><td>$\alpha_1 \ell_2 / \ell_1 \geq 1$</td><td></td><td>90</td><td>75</td><td>45</td></tr>
<tr><td rowspan="4">Percentage of negative flexural moment at edge supports</td><td rowspan="2">$\alpha_1 \ell_2 / \ell_1 = 0$</td><td>$\beta_t = 0$</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>$\beta_t > 2.5$</td><td>75</td><td>75</td><td>75</td></tr>
<tr><td rowspan="2">$\alpha_1 \ell_2 / \ell_1 \geq 1$</td><td>$\beta_t = 0$</td><td>100</td><td>100</td><td>100</td></tr>
<tr><td>$\beta_t > 2.5$</td><td>90</td><td>75</td><td>45</td></tr>
<tr><td rowspan="2">Percentage of positive flexural moment at the midspan</td><td>$\alpha_1 \ell_2 / \ell_1 = 0$</td><td></td><td>60</td><td>60</td><td>60</td></tr>
<tr><td>$\alpha_1 \ell_2 / \ell_1 \geq 1$</td><td></td><td>90</td><td>75</td><td>45</td></tr>
</table>

Linear interpolations shall be made between values shown

15.6.9.3 Flexural moments in column strip is distributed between beam and slab, in accordance with following conditions:

a) Beams between supports shall be proportioned to resist 85 percent of column strip moments if $\alpha_1 \ell_2 / \ell_1$ is equal to or greater than 1.0. For values of $\alpha_1 \ell_2 / \ell_1$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

b) Slab contribution of flexural moment in the column strip is equal to the difference between flexural moment in the column strip is equal to the difference between flexural moment in this strip and flexural moment in the beam.

c) In addition to flexural moments transferred from slab system, beams shall be proportioned to resist all moments caused by concentrated or linear loads applied directly to beams, including weight of projecting beam stem above or below the slab.

15.6.10 Factored shear in slab systems with beams

Factored shear in slabs and beams in slab-beam systems shall be determined in accordance with requirements of 15.7.8.

❒ 15.7 Direct method

15.7.1 Scope

15.7.1.1 Design of slab systems within limitations of 15.7.1.2 through 15.7.1.7 and requirements of 15.6.1, both, by the Direct Design Method shall be permitted.

15.7.1.2 Slab system shall have a minimum of three continuous spans in each direction.

15.7.1.3 Slabs shall be rectangular, with a ratio of longer to shorter span center–to-center of supports within a panel not greater than 2.

15.7.1.4 Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.

15.7.1.5 Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines of successive columns shall be permitted.

15.7.1.6 Gravity loads acting on the slab system shall be uniformly distributed over an entire panel. Live load shall not exceed two times dead load.

15.7.1.7 For slabs built integrally with beams supporting it on all four sides, the relative stiffness of beams in two perpendicular directions shall satisfy the Eq. (15-7).

15.7.2 Design method

15.7.2.1 A structure may be divided into a number of equivalent frames in each direction, as defined in 15.6.3.

15.7.2.2 Absolute sum of maximum positive flexural moments and average negative flexural moments of supports in each panel of the equivalent frame, called Static Flexural Moment is determined in accordance with 15.7.3.

15.7.2.3 Total factored static moment, obtained for each panel in accordance with 15.7.4, shall be distributed between positive flexural moments at midspan and negative flexural moments at supports within slab-beam strip.

15.7.2.4 Positive and negative flexural members in the slab-beam strip are distributed between beam and slabs in accordance with 15.6.9; design of beam and slabs shall be based on those moments.

15.7.2.5 Effect on flexural moments due to alternate loading of spans should be determined in accordance with requirements 15.7.6.

15.7.2.6 Flexural moments at columns and at support walls are determined in accordance with requirements of 15.7.6.

15.7.2.7 Shear load.effects in beams and slabs is determined using 15.7.7.

15.7.3 Static flexural member at each span

15.7.3.1 For a slab system, absolute sum of positive flexural moments at midspan and average negative flexural moment at supports, in each span of the slab-beam strip is equal to the static flexural moments of the strip in that span and is determined from the equation:

$$M_o \frac{w_u \ell_2 \ell_{1n}^2}{8} \qquad (15\text{-}8)$$

15.7.3.2 Clear span length ℓ_{1n}, obtained from Eq. (15-8) is the distance from face-to-face of columns, capitals, brackets, or support walls. ℓ_{1n} shall not be taken less than 0.85 ℓ_1 in calculations. Circular or regular shaped supports shall be treated as square supports with the same area. The extent of ℓ_{1n} shall be considered up to face of the hypothetical section.

15.7.4 Positive and negative flexural moments for each span

15.7.4.1 For an interior span of slab-beam strip, total static flexural moment, M_0, obtained from 15.7.3 is distributed between positive flexural moments at midspan and negative flexural moment at supports as follows:

a) Negative flexural moment in each support

$0.65M_0$

b) Positive flexural moment at midspan

$0.35M_0$

15.7.4.2 In each end span of slab-beam strip, total static flexural moment, M_o, obtained from 15.7.3, is distributed between positive flexural moments at midspan and negative flexural moments at supports as prescribed in table 15.7.4.2:

Table (15.7.4.2): Static flexural moment distribution at end span

Flexural moment	Support conditions				
	End support		**Slab with beam**	**Flat slab**	
	Non-restrained	**Fully Restrained**	**Cast integrally**	**With edge beam**	**Without edge beam**
Negative at interior support	0.75	0.65	0.70	0.70	0.70
Positive at midspan	0.63	0.35	0.57	0.50	0.52
Negative at end support	0.0	0.65	0.16	0.30	0.26

15.7.4.3 Negative flexural moments of 15.7.4.1 and 15.7.4.2 are considered as flexural moments at the face of supports, as defined in 15.7.3.2.

15.7.4.4 Sections adjacent to interior supports shall be designed for the largest flexural moment in both sides of the support.

15.7.4.5 Edge beams or edges of slab shall be proportioned to resist torsional moment equal to slab share of exterior negative flexural moments.

15.7.4.6 For end supports, the flexural moment transferred between slab system and edge column in accordance with 15.4.3 is treated as the resisting flexural moment in column strip. Such flexural moment is incorporated only for calculations related to shear in slab.

15.7.5 Modification of positive and negative flexural moments

15.7.5.1 Redistribution of flexural moments according to chapter 10, is not valid for slab systems designed by the direct method, but modification of positive and negative flexural moments by 10 percent for any span shall be permitted, provided that the effects on other flexural moments is accounted for.

15.7.6 Flexural moments in columns and walls

15.7.6.1 Columns and walls that make up exterior support of the equivalent frame, shall be designed support considered, as determined from 15.7.4.2; and are distributed in direct proportion to the corresponding flexural stiffness of the supporting elements above and below the floor considered.

15.7.6.2 Columns and walls that make up interior support of the equivalent frame, shall be designed to resist flexural moments specified by the following formula and are distributed in direct proportion to the corresponding flexural stiffness of the supporting elements above and below the floor considered:

$$M_u = 0.07\left[(w_d + 0.5w_l)\ell_2\ell_{1n}^2 - w_d\ell'_2\ell'^2_{1n}\right] \qquad (15\text{-}9)$$

Where ℓ'_2, ℓ'_{1n} and ℓ'^2_{1n} refer to shorter span.

15.7.7 Factored shear in slab systems with beams

15.7.7.1 Beams with $\alpha_1\ell_2/\ell_1$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45 degree lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides. In other words, the same load produced by trapezoidal–triangular distribution.

15.7.7.2 In proportioning of beams with $\alpha_1\ell_2/\ell_1$ less than 1.0 to resist shear, linear interpolation may be used between the value stated in 15.7.7.1 for $\alpha_1\ell_2/\ell_1$ equal to unity and zero for α_1, assuming beams carry no load at $\alpha_1 = 0$.

15.7.7.3 In addition to shears transferred from slabs, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

15.7.7.4 Computation of slab shear strength along the strip common with beam, shall be such that the slab can resist shear transferred from slab to beam as per 15.7.7.1 and 15.7.7.2.

15.7.7.5 Shear strength of slab shall be determined according to chapter 12.

❒ 15.8 Flexural moment coefficients method

15.8.1 Scope

15.8.1.1 Flexural moment coefficient method may be applied to square shape slabs within limitations of 15.8.1.2 through 15.8.1.5.

15.8.1.2 Slab is supported on four sides by beams or walls.

15.8.1.3 Bearing beams dimensions shall satisfy the following equation:

$$\frac{b_w h_b^3}{\ell_n h_s^3} \geq 2 \qquad (15\text{-}10)$$

15.8.1.4 Clear length-to-width ratio of slab is less than or equal to 2.

15.8.1.5 Gravity loads are the only loads acting uniformly on the slab.

15.8.2 Design method

15.8.2.1 Considering the end conditions at supports, each slab shall be designed separately and divided into strips with the following specifications, in each direction:

a) Middle strip with width equal to one-half the depth of slab at its middle half
b) End strip with a width equal to one-fourth the depth of slab on both sides of middle strip

15.8.2.2 Variations of positive and negative flexural moments in the span of middle strip is considered uniform.

15.8.2.3 Variations of positive and negative flexural moments in the span of each of the end strips is considered non-uniform, but linear. For the common border with middle strip, these moments are equal to the values of middle strip but for the exterior border, they shall be equal to one-third those values.

15.8.2.4 Maximum values of positive flexural moments at midspan, and negative flexural moments at face of support per unit width of middle strip is determined as follows:

a) For dead and live loads, negative flexural moments at face of supports are determined using coefficients given in table 15.8.2.4.a and the following formulas:

$$M^-_{A(D+L)} = C^-_A W_{(D+L)} \ell^2_A \qquad (15\text{-}11)$$

$$M^-_{B(D+L)} = C^-_B W_{(D+L)} \ell^2_B \qquad (15\text{-}12)$$

b) For dead loads, positive flexural moments at midspan are determined using coefficients given in table 15.8.2.4.b and the following formulas:

$$M^{+}_{AD} = C^{+}_{AD} W_D \ell^2_A \tag{15-13}$$

$$M^{+}_{BD} = C^{+}_{BD} W_D \ell^2_B \tag{15-14}$$

c) For live loads, positive flexural moments at midspan are determined using coefficients given in table 15.8.2.4.b and the following formulas:

$$M^{+}_{AL} = C^{+}_{AL} W_L \ell^2_A \tag{15-15}$$

$$M^{+}_{BL} = C^{+}_{BL} W_L \ell^2_B \tag{15-16}$$

15.8.2.5 Where flexural moment at one side of the support for two slabs is less than 80 percent of this same moment at the other side of support, moment difference shall be distributed between two slabs in direct proportion to their corresponding flexural stiffness.

15.8.2.6 When a slab is not extended to the other support side, it shall be designed, at any strip, for a negative flexural moment equivalent to three-fourths positive flexural moment at midspan, in the same strip.

15.8.3 Slab thickness

15.8.3.1 Slab thickness in this method shall not be taken less than the following values:

a) For slabs that are discontinuous in one or more sides, slab perimeter divided by 140
b) For slabs that are continuous on all four sides, slab perimeter divided by 160
c) 100 mm

15.8.4 Shear in beam and slab

15.8.4.1 Beams shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45 degree lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long side. That is, the load obtained from trapezoidal–triangular distribution.

15.8.4.2 In addition to shears transferred from slabs, beams shall be proportioned to resist shear caused by factored loads applied directly on beams.

15.8.4.3 Computation of slab shear strength along the strip common with beam, shall be such that the slab can resist shear transferred from slab to beam as per 15.8.4.1. Shear is assumed to be uniformly distributed along support length.

15.8.4.4 Shear in slabs and loads acting on beams may be determined with the help of coefficients in table 15.8.4.4 in which, distribution ratios for loads acting uniformly on slabs and transferred in two directions A and B, are given. It is assumed that such shears are distributed uniformly along the length of supports.

15.8.4.5 Slab shear strength shall be determined according to requirements of chapter 12.

15.8.5 Flexural moments in beams

15.8.5.1 Flexural moments in beams are computed either on the basis of loads transferred to them by the slabs, in accordance with 15.8.4.1, or on the basis of an equivalent uniform load equal to the following values:

a) For support beams at shorter dimension of slab:

$$\frac{w_u \ell_A}{3} \qquad (15\text{-}17)$$

b) For support beams at longer dimension of slab:

$$\left(\frac{w_u \ell_A}{3}\right)\left(\frac{3-m^2}{2}\right) \qquad (15\text{-}18)$$

Where ℓ_A is the shorter dimension of the slab, and m is the ratio of longer to shorter dimension of slab

❐ 15.9 Plastic method

15.9.1 Scope

15.9.1.1 Plastic design method for slabs can be applied for all slabs subjected to gravity loads in ultimate limit state of strength, regardless of their geometric shape and boundary.

15.9.1.2 When this design method is used, assurance should be provided regarding shear strength of slab in ultimate limit state of strength.

15.9.1.3 When this design method is used, assurance should be provided regarding proper performance of slab in ultimate limit state of serviceability, deformations and cracking, in accordance with requirement of chapter 14.

15.9.2 General design procedures

15.9.2.1 Plastic design may be carried out as following procedures:

a) Strips method or static method referred to as “lower limit”

b) Failure hinges method or cinematic method, referred to as “upper limit”

15.9.2.2 Reinforcement placement in slab shall be so as to provide assurance regarding adequacy of rotation capacity in slab sections. Accordingly, the ratio of tensile reinforcement in each extension shall be taken less than one-half the ratio corresponding to balanced section by an amount $0.5\rho_b$.

15.9.2.3 Ratio of flexural moments in continuous supports, to flexural moments at midspan shall not be taken less than 0.5 nor greater than 2.

15.9.2.4 When strip design method is used, flexural moments distribution function shall be preferably estimated in accordance with linear elastic analysis of slab as much as possible. Determination of required reinforcement in the slab shall be based on plastic change of the distribution or provision of equivalent conditions.

15.9.2.5 When failure hinge design method is used, different probable failure mechanisms for slabs shall be considered. Assurance shall also be provided so that the ultimate load prescribed for the slab is the minimum possible.

Table (15.8.2.4.a): Coefficient of negative moments

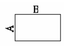
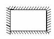
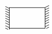
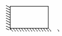
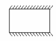
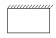
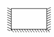
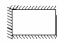
$m = \frac{\ell_A}{\ell_B}$	Coefficient	Case1 	Case2 	Case3 	Case4 	Case5 	Case6 	Case7 	Case8 	Case9 
1.00	C_A^-	_	0.045	_	0.050	0.075	0.071	_	0.033	0.061
	C_B^-	_	0.045	0.076	0.050	_	_	0.071	0.061	0.033
0.95	C_A^-	_	0.050	_	0.055	0.079	0.075	_	0.038	0.065
	C_B^-	_	0.041	0.072	0.045	_	_	0.067	0.056	0.029
0.90	C_A^-	_	0.055	_	0.060	0.080	0.079	_	0.043	0.068
	C_B^-	_	0.036	0.070	0.040	_	_	0.062	0.052	0.025
0.85	C_A^-	_	0.060	_	0.066	0.082	0.083	_	0.049	0.072
	C_B^-	_	0.031	0.065	0.034	_	_	0.057	0.046	0.021
0.80	C_A^-	_	0.065	_	0.071	0.084	0.086	_	0.055	0.075
	C_B^-	_	0.026	0.061	0.029	_	_	0.051	0.041	0.017
0.75	C_A^-	_	0.069	_	0.076	0.085	0.088	_	0.061	0.078
	C_B^-	_	0.022	0.056	0.024	_	_	0.044	0.036	0.014
0.70	C_A^-	_	0.074	_	0.081	0.086	0.091	_	0.068	0.081
	C_B^-	_	0.017	0.050	0.019	_	_	0.038	0.029	0.011
0.65	C_A^-	_	0.077	_	0.085	0.087	0.093	_	0.074	0.083
	C_B^-	_	0.014	0.043	0.015	_	_	0.031	0.025	0.008
0.60	C_A^-	_	0.081	_	0.089	0.088	0.095	_	0.080	0.085
	C_B^-	_	0.010	0.035	0.011	_	_	0.024	0.018	0.006

Table (15.8.2.4.a): Continued

$m = \frac{\ell_A}{\ell_B}$	Coefficient	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.55	C_A^-	_	0.084	_	0.092	0.089	0.096	_	0.085	0.086
	C_B^-	_	0.007	0.028	0.008	_	_	0.019	0.014	0.005
0.50	C_A^-	_	0.086	_	0.094	0.090	0.097	_	0.089	0.088
	C_B^-	_	0.006	0.022	0.006	_	_	0.014	0.010	0.003

Table (15.8.2.4.b): Coefficient of positive moments

$m = \frac{\ell_A}{\ell_B}$	Coefficient	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
1/0	C_{AL}^+	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	C_{AD}^+	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	C_{BL}^+	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
	C_{BD}^+	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020
0.95	C_{AL}^+	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	C_{AD}^+	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	C_{BL}^+	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
	C_{BD}^+	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017

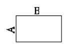
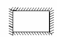
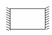
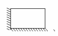
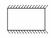
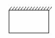
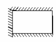
Table (15.8.2.4.b): Continued

$m = \frac{\ell_A}{\ell_B}$	Coefficient	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.9	C_{AL}^{+}	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	C_{AD}^{+}	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	C_{BL}^{+}	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
	C_{BD}^{+}	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	C_{AL}^{+}	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	C_{AD}^{+}	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	C_{BL}^{+}	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
	C_{BD}^{+}	0.026	0.012	0.023	0.019	0.011	0.017	0.025	0.017	0.013
0.8	C_{AL}^{+}	0.055	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	C_{AD}^{+}	0.055	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	C_{BL}^{+}	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
	C_{BD}^{+}	0.023	0.011	0.020	0.016	0.009	0.014	0.022	0.025	0.010
0.75	C_{AL}^{+}	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	C_{AD}^{+}	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	C_{BL}^{+}	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.014
	C_{BD}^{+}	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	C_{AL}^{+}	0.68	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	C_{AD}^{+}	0.68	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	C_{BL}^{+}	0.16	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.012
	C_{BD}^{+}	0.16	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006

Table (15.8.2.4.b): Continued

$m = \frac{\ell_A}{\ell_B}$	Coefficient	Case1	Case2	Case3	Case4	Case5	Case6	Case7	Case8	Case9
0.65	C^+_{AL}	0.74	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	C^+_{AD}	0.74	0.053	0.054	0.050	0.036	0.053	0.065	0.044	0.034
	C^+_{BL}	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
	C^+_{BD}	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	C^+_{AL}	0.081	0.058	0.072	0.067	0.059	0.068	0.077	0.065	0.059
	C^+_{AD}	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	C^+_{BL}	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
	C^+_{BD}	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	C^+_{AL}	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	0.063
	C^+_{AD}	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	C^+_{BL}	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
	C^+_{BD}	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	C^+_{AL}	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	C^+_{AD}	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	C^+_{BL}	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004
	C^+_{BD}	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

Table (15.8.4.4): Ratio for uniform distribution of load acting on the slab in the directions of ℓ_A and ℓ_B

$m = \dfrac{\ell_A}{\ell_B}$	Coefficient	Case1 	Case2 	Case3 	Case4 	Case5 	Case6 	Case7 	Case8 	Case9 
1/00	W_A	0.5	0.5	0.17	0.5	0.83	0.71	0.29	0.33	0.67
	W_B	0.5	0.5	0.83	0.5	0.17	0.29	0.71	0.67	0.33
0.95	W_A	0.55	0.55	0.20	0.55	0.86	0.75	0.33	0.38	0.71
	W_B	0.45	0.45	0.80	0.45	0.14	0.25	0.67	0.62	0.29
0.9	W_A	0.60	0.60	0.23	0.60	0.88	0.79	0.38	0.43	0.75
	W_B	0.40	0.40	0.77	0.40	0.12	0.21	0.62	0.57	0.25
0.85	W_A	0.66	0.66	0.28	0.66	0.90	0.83	0.43	0.49	0.79
	W_B	0.34	0.34	0.72	0.34	0.10	0.17	0.57	0.51	0.21
0.80	W_A	0.71	0.71	0.33	0.71	0.92	0.86	0.49	0.55	0.83
	W_B	0.29	0.29	0.67	0.29	0.08	0.14	0.51	0.45	0.17
0.75	W_A	0.76	0.76	0.39	0.76	0.94	0.88	0.56	0.61	0.86
	W_B	0.24	0.24	0.61	0.24	0.06	0.12	0.44	0.39	0.14
0.70	W_A	0.81	0.81	0.45	0.81	0.95	0.91	0.62	0.68	0.89
	W_B	0.19	0.19	0.55	0.19	0.05	0.09	0.38	0.32	0.11
0.65	W_A	0.85	0.85	0.53	0.85	0.96	0.93	0.69	0.74	0.92
	W_B	0.15	0.15	0.47	0.15	0.04	0.07	0.31	0.26	0.08
0.60	W_A	0.89	0.89	0.61	0.89	0.97	0.95	0.76	0.80	0.94
	W_B	0.11	0.11	0.39	0.11	0.03	0.05	0.24	0.20	0.06
0.55	W_A	0.92	0.92	0.69	0.92	0.98	0.96	0.81	0.85	0.95
	W_B	0.08	0.08	0.31	0.08	0.02	0.04	0.19	0.15	0.05
0.50	W_A	0.94	0.94	0.76	0.94	0.99	0.97	0.86	0.89	0.97
	W_B	0.06	0.06	0.24	0.06	0.01	0.03	0.14	0.11	0.03

CHAPTER SIXTEEN

WALLS

❒ 16.0 Notations

A_g = gross area of section, mm^2.

f_c = specified compressive strength of concrete, MPa (N/mm^2).

h = overall thickness of member, mm.

k = effective length factor.

ℓ_c = vertical distance between supports, mm.

N_r = design ultimate axial load strength of wall, N.

ϕ_c = partial safety factor for concrete.

❒ 16.1 Scope

Provisions of this chapter shall apply for design of reinforced concrete walls.

❒ 16.2 Definitions

Load-carrying wall

Is a wall that is mainly subjected to gravity loads acting on its middle plate, with or without flexure.

Shear wall
Is a wall that is mainly subjected to horizontal loads acting on its middle plate.

Retaining wall
Is a wall that is mainly subjected to vertical loads acting on its middle plate.

Grade beam wall
Is a wall with behavior similar to deep beam, which transfers or distributes load in the foundation system and maintains connection between structure and footing.

❒ 16.3 General design principles

16.3.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

16.3.2 Compression members built integrally with walls shall conform to 11.8.2

16.3.3 Walls shall be anchored to adjacent intersecting elements such as floors, roofs, columns, buttresses, pilasters, intersecting walls or footings, to provide stability.

16.3.4 Transfer of force to footing at base of wall shall be in accordance with 17.6.

❐ 16.4 Reinforcement limitation

16.4.1 Minimum vertical and horizontal reinforcement in the wall shall be taken less than corresponding values in 16.4.2 and 16.4.3 respectively. For walls subjected to shear force, observance of minimum reinforcement in accordance with 12.16.4 is required.

16.4.2 Minimum ratio of vertical reinforcement area to gross concrete area for different bars shall be:

a) For deformed 16mm bars with a specific yield strength not less than 400MPa (N/mm^2)	0.0012
b) For other deformed bars	0.0015
c) For up to 16mm welded wire fabric (plain or deformed)	0.0012

16.4.3 Minimum ratio of horizontal reinforcement area to gross concrete area for different bars shall be:

a) For 16mm deformed bars with a specified yield strength not less than 400MPa (N/mm^2)	0.0020
b) For other deformed bars	0.0025
c) For up to 16mm welded wire fabric (plain or deformed)	0.0020

16.4.4 Ratio of vertical and horizontal reinforcement area to gross concrete area shall not be taken greater than 0.04. Limitation of the maximum value shall be applied at the location of splices.

16.4.5 Walls more than 250mm thick, except basement and retaining walls shall have vertical and horizontal reinforcement for each direction placed in two layers parallel with faces of wall in accordance with 16.4.5.1 or 16.4.5.2.

16.4.5.1 For exterior walls directly exposed to soil or outside weather:

a) A grid layer consisting of not less than one-half and not more than two-thirds of total reinforcement required for each direction shall be placed not less than 50mm nor more than one–third the thickness of wall from exterior surface exposed to soil or outside weather.

b) A grid layer consisting of the balance of required reinforcement in that direction shall be placed not less than 20mm nor more than one-third the thickness of wall from interior surface. Concrete layer over reinforcement shall also be provided in accordance with 8.2.8.

16.4.5.2 For other walls, a grid layer consisting of one-half reinforcement required for each direction shall be placed more than 20mm and less than one-third the thickness of wall from any surface. Concrete layer over reinforcement shall also be provided in accordance with 8.2.8.

16.4.6 Adjacent vertical and horizontal reinforcement in each layer shall not be spaced farther apart than three times the wall thickness, nor 350mm.

16.4.7 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than one percent gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

16.4.8 At least two 16mm bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corners of the openings.

❒ 16.5 Load-carrying walls

16.5.1 For ultimate limit state of strength, horizontal sections in load-carrying wall shall be designed as sections subject to compression and flexure in accordance with requirements of 11.2 through 11.4 and those of chapter 13, related to slenderness effects.

16.5.2 For walls of solid rectangular section in which load eccentricity at ultimate limit state of strength is less than one-sixth thickness of wall, its section shall be designed for ultimate limit state of strength in accordance with 11.2. Calculated strength of section against axial load, N_r, is obtained from the following empirical relation:

$$N_r = 0.55\,\phi_c f_c A_g \left[1 - \left(\frac{K\ell_c}{32h}\right)^2\right] \qquad (16\text{-}1)$$

Where K is the effective length factor of the wall adopted as follows:

a) For walls braced against lateral translation at top and bottom, which are restrained against rotation at one or both ends (above and below the wall): K= 0.8

b) For walls braced against lateral translation at top and bottom, which are unrestrained against rotation at both ends (above and below the wall): K= 1.0

c) For walls not braced against lateral translation: K= 2.0

16.5.3 Thickness of walls designed in accordance with 16.5.2 shall be less than the following values:

a) one-twenty-fifth the clear height or length, whichever is shorter, and 150mm

b) For exterior basement walls and other walls directly exposed to soil: 200mm

16.5.4 Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each concentrated load shall not exceed width of bearing plus two times the wall thickness, nor center-to-center distance between loads.

❒ 16.6 Shear walls

16.6.1 Design of shear walls to resist against shear shall be based on requirements of 12.16

16.6.2 Design of shear walls to resist against flexural moment and axial forces shall be based on requirements of chapter 11.

❒ 16.7 Retaining walls

16.7.1 Retaining walls shall be designed as flexural members according to chapter 11 and chapter 15.

❒ 16.8 Grade beams walls

16.8.1 Grade beam walls shall be designed for shear and flexure according to chapters 12 and 11, respectively.

16.8.2 A portion of grade beam walls above grade shall also meet the requirements of reinforcement placement according to 16.4.

CHAPTER SEVENTEEN

FOOTINGS

❒ 17.0 Notation

d = effective depth of section, mm.
d_p = diameter of pile at concrete pile cap.
β = ratio of long side to short side of footing.

❒ 17.1 Scope

17.1.1 Provisions of this chapter shall apply for design of footings on soil or on piles, and for concrete piles

17.1.2 Design of footings and piles for flexure, axial loads and shears shall meet the requirements of chapter 11 and 12 and those of chapter13 for buckling and slenderness, and also requirements of chapter 18 regarding bond between concrete and reinforcement. Provisions of this chapter are related to additional or substitutive requirements that are to be satisfied for footings and piles.

❐ 17.2 Definitions

According to this code, footing is referred to that portion of structure above which, columns or walls are located with its lower face either directly on the ground or on pile so as to transfer structural load to the ground. The footing on pile is referred to as pile cap.

17.2.1 Types of footing

17.2.1.1 Isolated footings are referred to those footings that are proportioned to transfer to ground, load of one or two columns at expansion joint. Single spread footing may be rectangular, regular polygon, circular or any other irregular shaped and its section may also be rectangular, trapezoidal or stepped. Closely arranged single spread footings may act together as a " combined footing".

17.2.1.2 Strip footing is referred to the continuous footing that transfers to the ground, the load of a wall or of a few columns arranged in a row. Footing section may be rectangular or trapezoidal shaped or heeled (inverted T). When strip footing transfers merely the wall load to the ground, it is referred to as, "wall supporting foundation".

17.2.1.3 Mat (or raft) foundation is referred to the footing that transfers to the ground, the load of few walls or columns that are arranged in rows of different directions. Mat foundation may be in the shape of slab, beam-slab combination, or box.

17.2.1.4 Strap footing is a combination of two spread footings where the resultant of loads acting on one of them, has great eccentricity relative to the centroid of footing, and that footings are connected to each other through a rigid beam. The rigid beam that transfers the corresponding load of one footing to the other is assumed not bearing on the soil.

17.2.2 Pile types

Piles are elements of deep foundation that transfer structural loads to the ground, piles may have isolated or group arrangement.

17.2.2.1 Isolated pile is the pile that directly receives and transfers to the ground, all loads of a column.

17.2.2.2 Pile group is referred to a number of piles that receive the load of one or a number of columns through a pile cap.

❒ 17.3 General design principles

17.3.1 Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure or permissible pile capacity selected through principles of soil mechanics.

17.3.2 Combinations of loading forces considered in 17.3.1, are those stated in 10.5.3 in which, partial safety factors for loads are taken equal to 1.

17.3.3 Where wind or earthquake is one of the forces of the governing load combination for footing, permitted soil stress or permitted pile load may be increased by maximum 33 percent.

17.3.4 Design of footing and pile sections for axial loads, flexure, flexure and axial load and shear shall be done in ultimate limit state of strength and according to chapters 11, 12 and 13.

17.3.5 Checking for buckling need not be done for piles with full length inside compact soil layers, but for piles with full or partial length inside loose soil or out of soil, a buckling check is required considering special support conditions.

17.3.6 Bond control between concrete and reinforcement at different sections of footing and pile cap, and the procedure for development of reinforcement in the sections shall be based on requirements of chapter 18. Critical sections for bond control, in addition to sections specified in 17.4.3.2 for flexure, include also those sections in which changes in dimensions or reinforcement area occur.

17.3.7 Longitudinal reinforcement of piles, in pile group, shall be properly extended and anchored inside pile cap, considering type of selected connection (restrained or hinged).

17.3.8 Depth of footings shall not be less than 250mm and thickness of pile cap for group-pile shall not be taken less than 400mm.

❒ 17.4 Principles for determination of forces in footings

17.4.1 General

17.4.1.1 Flexural moments and ultimate shears used in design of different sections of footing shall be determined for ultimate loads and corresponding induced reactions, based on established principles for structural design.

17.4.1.2 Instead of using requirements of 17.3.1.1 in footings, flexural moments and ultimate shears at different sections may be determined with approximation from the product of the values of these forces (under unfactored loads) and some overall safety factor. The overall safety factor shall be properly determined from dividing ultimate loads to service loads.

17.4.1.3 For mat foundation, flexural moments and shears may be computed using equivalent frame method based on requirements of 15.6.

17.4.1.4 Regarding footings on piles, computations for flexural moments and shears shall be permitted to be on the assumption that the reaction from any pile is concentrated at pile center.

17.4.1.5 For footings supporting circular or regular polygon shaped concrete columns, location of the critical sections in flexure and shear shall be determined by treating column or pedestal sections as square members with the same area.

17.4.1.6 Sloped or stepped isolated footings designed as a unit shall be constructed to ensure action as an integrated unit.

17.4.2 Soil pressure distribution

17.4.2.1 Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure, and with established principles of soil mechanics.

17.4.2.2 If more detailed analysis is not done, soil pressure distribution in isolated footings may be determined assuming solid footing.

17.4.2.3 Soil pressure distribution in isolated footings shall be such that pressure on soil in some part of the footing is equal to zero, provided the length of that part does not exceed one-fourth the dimension of footing in any extension.

17.4.2.4 When forces acting on the footing are of tensile type, special provisions like use of tie-rod shall be considered to prevent uplift of the footing. The provisions shall be such that safety factor against uplift forces is least equal to 1.5.

17.4.2.5 For strap footings, the connecting beam between footings shall be adequately solid to prevent rotation of footing subjected to eccentric loading. Unless a more precise analysis is done, moment of inertia of the beam section shall be taken at least equal to the moment of inertia of the footing section subjected to eccentric loading. The beam shall be designed for flexure and shear. Therefore, distribution of the soil pressure below footings may be considered uniform.

17.4.3 Flexural moment

17.4.3.1 Effective flexural moment at each footing section shall be determined by passing a vertical plane through the footing, and computing the flexural moment of forces acting over entire area of footing on one side of that vertical plane.

17.4.3.2 Maximum flexural moment for an isolated footing shall be computed at critical sections, in the vicinity of columns, pedestals and walls, located as follows:

a) At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall

b) Halfway between middle and edge of wall, for footings supporting a masonry wall
c) Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate

17.4.3.3 For isolated footings supporting a wall, the possibility for occurrence of a negative moment and the necessity for reinforcement placement above footing section shall be checked.

17.4.4 Shear in footings

17.4.4.1 Critical section for shear shall be measured at a distance d from the following specified locations:

a) From face of column, pedestal, or wall, for footings supporting a column, pedestal, or wall.
b) Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

17.4.4.2 Computation of shear on any section through a footing supported on piles (or pile cap) shall be in accordance with the following:

a) Entire reaction from any pile whose center is located $\frac{d_p}{2}$ or more outside the section shall be considered as producing shear on that section.
b) Reaction from any pile whose center is located $\frac{d_p}{2}$ or more inside the section shall be considered as producing no shear on that section.
c) For intermediate positions of pile center, the position of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $\frac{d_p}{2}$ outside the section and zero value at $\frac{d_p}{2}$ inside the section.

❒ 17.5 Reinforcement limitation

17.5.1 For isolated and strap footings, and mat foundations (except connector beam), minimum percentage of flexural reinforcement shall not be less than total area of temperature and shrinkage reinforcement specified in 8.7. For connector beams in between strap footings, minimum reinforcement shall be based on 11.5.2.1.

17.5.2 Percentage of flexural reinforcement in strip footings shall not be taken less than 0.15, unless the area of reinforcement is at least one-third greater than the reinforcement computed. In this case, percentage may not be taken less than 0.1. Meeting the requirements of 8.7 across the entire section is required for footings.

17.5.3 Section area of reinforcement in footings shall not be less than 10mm and spacing between bars, center-to-center, shall not be less than 100mm, nor greater than 350mm.

17.5.4 For mass footings, in which dimensions and volume of concrete are considered independently of computed needs, minimum flexural moment in accordance with 17.5.1, need not be applied. If surface crack control is to be considered, a network of skin reinforcement in those surfaces shall be applied in accordance with 8.7.5. Maximum spacing of skin reinforcement is 350mm.

17.5.5 In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing. In two-way rectangular footings, reinforcement shall be distributed as follows:

a) Longitudinal reinforcement of the footing shall be uniformly distributed across entire width of footing.

b) A portion of the total transverse reinforcement of the footing, given by Eq. (17-1) shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing and the remainder of reinforcement required in short direction shall be distributed uniformly outside center band width of footing.

$$\frac{\text{Reinforcement in center band width in transversal direction}}{\text{Total reinforcement in short direction of footing}} = \frac{2}{1+\beta} \qquad (17\text{-}1)$$

17.5.6 Minimum and maximum percentage of longitudinal reinforcement in piles shall be computed similar to columns and considering requirements of chapter 11.

17.5.7 Transverse reinforcement of piles shall be considered as tie or spiral.

❒ 17.6 Force transfer from column base, wall, or concrete pedestal, to the footing

17.6.1 Forces and moments at base of column, wall or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels, and mechanical connectors.

17.6.2 Bearing on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for each of the contact surfaces as given by 11.10.

17.6.3 Column longitudinal reinforcement, dowels, or mechanical connectors between supporting and supported members shall be adequate to transfer forces. In addition, limitations of 17.6.6 and 17.6.7 shall be satisfied:

a) That portion of the compressive force that exceeds concrete bearing strength between two members.
b) Any computed tensile force across interface.

17.6.4 In transferring calculated flexural moments to pedestal or footing, reinforcement, dowels, or mechanical connectors shall satisfy bond requirements between concrete and reinforcement, according to chapter 18.

17.6.5 Shear forces shall be transferred to supporting pedestal or footing in accordance with shear-friction action and requirements of 12.14, or by other appropriate means.

17.6.6 For columns and pedestals, area of reinforcement crossing interface between supporting and supported members shall not be less than 0.005 times gross area of supported member.

17.6.7 For walls, area of reinforcement crossing interface between wall and footing shall not be less than minimum vertical reinforcement given in 16.4.2.

17.6.8 When using mechanical devices to create pinned or rocker connection between column and footing, special requirements of 17.6.1 through 17.6.5, and those of 17.6.9, regarding connection of mechanical devices of column and footing shall be satisfied.

17.6.9 Anchor bolts and mechanical connectors shall be designed to reach their design strength prior to anchorage failure or failure of surrounding concrete.

❒ 17.7 Connector ties between footings

17.7.1 Separate ties in a structure shall be connected together in two preferably orthogonal directions by connector ties so that they prevent movement of the two footings relative to each other. In single story structures with large span such as industrial buildings, hangers and etc… in which, footings have adequate depth and stability to withstand lateral loads, provision of tie in the direction of span may be ignored. The embankment around such footing should later be rammed and compacted.

17.7.2 Connector ties between footings shall be designed for tensile force equivalent to 10 percent of the largest ultimate axial load acting on the columns on both sides.

17.7.3 Connector section dimensions shall be proportional to footing dimensions and shall be taken at least 250mm.

17.7.4 Number of longitudinal tie reinforcement shall be at least four with minimum size 12mm. The reinforcement shall be tied together by transversal bars of at least 6mm in size, spaced maximum 250mm from each other.

17.7.5 Longitudinal tie bars shall extend through middle footings, and shall be anchored at the face of side footings.

CHAPTER EIGHTEEN

ANCHORAGE AND SPLICES OF REINFORCEMENT

❒ 18.0 Notation

A_b = area of an individual bar, mm^2.
A_{tr} = total cross–sectional area of all transverse reinforcement which is within the spacing S and which crosses the potential plane of splitting through the reinforcement being developed or spliced, mm^2.
A_w = area of an individual wire to be developed or spliced, mm^2.
b_w = web width, or diameter of circular section, mm.
c = concrete cover over reinforcement, or center-to-center spacing of bars, mm.
d = distance from extreme compression fiber to centroid of tension reinforcement, mm.
d_b = nominal diameter of bar or wire, mm.
f_b = bond strength of concrete, MPa (N/mm^2).
f_{bd} = design bond strength of concrete, MPa.
f_c = specified compressive strength of concrete, MPa.
$\sqrt{f_c}$ = square root of compressive strength of concrete, MPa.
f_y = specified yield strength of reinforcements, (f_{yk}), MPa. For simplicity, letter K is omitted from subscript, in this notation.
h = overall thickness of member, mm.

k_{tr} = transverse reinforcement index, mm.

ℓ_a = additional embedment length at support or at point of inflection, mm.

ℓ_d = development length of tension reinforcement, mm.

ℓ_{db} = basic development length of tension reinforcement, mm.

ℓ_{dc} = development length of compressive reinforcement, mm.

ℓ_{dcb} = basic development length of compressive reinforcement, mm.

ℓ_{dh} = development length of a standard hook in tension, as defined in 18.2.1.3, mm.

ℓ_{dhb} = basic development length of a standard hook in tension, mm.

M_r = ultimate flexural strength of section, N.mm.

N = number of bars in reinforcement layer, which are being developed or spliced along critical section.

n = number of bars being developed or spliced in a location.

s = maximum center-to-center spacing of transverse reinforcement, mm.

s_w = spacing of wire to be developed or spliced, mm.

V_r = ultimate shear strength at section, N.

V_u = ultimate shear force at section, N.

β_b = ratio of area of reinforcement cut off to total area of tension reinforcement at section.

ϕ_c = partial safety factor for concrete.

ϕ_s = partial safety factor for reinforcement.

❐ 18.1 Scope

18.1.1 Provisions of this chapter shall apply to provide reinforcement embedment in concrete, and the method to splice bars together, in all reinforced concrete members.

18.1.2 Provisions of this chapter incorporates all bars and welded wire fabric that are subjected mainly to static loads. Regulations and requirements of this chapter shall not apply for the structures subjected mainly to dynamic loads. For structures exposed to lateral loads such as earthquake, in excess to requirements of this chapter, additional requirements of chapter 21 shall also apply.

18.1.3 Rules and regulations concerning reinforcement placement details, chapter eight, shall be considered as pre – requisite to this chapter and shall be applied in full, along with requirements of this chapter.

18.1.4 In this chapter, all calculations regarding bond between concrete and reinforcement are carried out in the ultimate limit state of strength, where both safety factors of strength ϕ_c and ϕ_s are taken equal to unity. When reference is made to other chapters, requirements of those chapters shall apply.

❐ 18.2 Reinforcement anchorage

18.2.1 General

18.2.1.1 Calculated tension or compression forces in reinforcement at each section of structural reinforced concrete members shall be developed on each side of the section and shall be transferred to concrete. Embedment of steel bars in concrete is carried out as one of the following three procedures or a combination thereof:

a) Bond between concrete and reinforcement at lateral face of bars.

b) Make hook at the ends of bars.

c) Application of mechanical devices along the length of bars.

18.2.1.2 Anchorage of reinforcement by bond between concrete and reinforcement requires that adequate length of steel bar on both sides of the section be embedded in the concrete so that bond stress developed at lateral side of the bar does not exceed bond strength of concrete. Minimum required length of bar to transfer $A_b f_y$ is referred to as “Development length of reinforcement”. Requirements to provide development length of tension and compression reinforcement are given in 18.2.2 and 18.2.3.

18.2.1.3 Anchorage of tension reinforcement by hook involves bending end section of bar into a hook. Requirements related to dimensions of standard hooks are given in 8.2.4.2 Bending reinforcement bar into a hook is not adequate to transfer $A_b f_y$ force from reinforcement to concrete and in addition, the excess straight part of bar from the section considered to the beginning of hook shall be embedded in the concrete. The Minimum excess length plus its end hook radius and reinforcement size, required to transfer $A_b f_y$ force, is referred to as “Development length of hooked bar”. Requirements to provide development length of hooked bar in tension are given in 18.2.5.

Hooks shall not be considered effective to develop bars in compression.

18.2.1.4 Development of reinforcement with the help of mechanical devices may involve connecting some gear such as metal plates, intersecting rods or similar, along the length of bar to prevent it from movement in concrete. Requirements for such development method are given in 18.2.6.

18.2.2 Development length of deformed bars and deformed wires in tension

18.2.2.1 Development length of a deformed bar or deformed wire in tension, ℓ_d, shall be at least equal to:

$$\ell_d = k_1\, k_2\, k_3\, \ell_{db} \tag{18-1}$$

Where ℓ_{db} is the basic development length calculated in accordance with 18.2.2.2, and factors k_1, k_2, and k_3 are taken as follows. However, the product of $k_1.k_2$ need not be taken greater than 1.7:

a) Factor k_1, or reinforcement location factor is taken equal to 1.3 for horizontal reinforcement so placed that at least 300mm of fresh concrete is cast in the member below the development length. For other reinforcement it is equal to 1.0.

b) Factor k_2, or coating factor is taken equal to 1.5 for epoxy–coated bars with concrete cover layer over reinforcement less than $3d_b$ or clear spacing less than $6d_b$. For all other epoxy-coated bars, it is equal to 1.2 and for uncoated reinforcement equal to unity.

c) Factor k_3, or excess reinforcement factor is used when reinforcement provided in the section is in excess of that required by structural analysis. The factor is equal to the ratio of reinforcement required area to the reinforcement area provided. When development of reinforcement is done specifically for transmission of stress f_y, and also for structures with high ductility, discussed in chapter 20, factor k_3 is taken equal to 1. Development length, ℓ_d, shall not be less than 300mm.

18.2.2.2 Basic development length of a bar, ℓ_{db}, is calculated by:

$$\ell_{db} = \frac{d_b f_y}{4 f_b} \tag{18-2}$$

Where f_b is the bond strength between concrete and reinforcement in tension, hereafter called bond strength. The strength is calculated from the following formula:

$$f_b = \lambda_1 \lambda_2 f_{bd} \qquad (18\text{-}3)$$

Where f_{bd} is the basic bond strength, which is given by the Eq. (18-4) for deformed bars and is taken to be one–half the value obtained from above formula for plain bars. Factors λ_1 and λ_2 are determined in accordance with 18.2.2.3 through 18.2.2.5.

$$f_{bd} = 0.65\sqrt{f_c} \qquad (18\text{-}4)$$

18.2.2.3 Factor λ_1, or reinforcement size factor is equal to 1 for bar size less than or equal to 20mm, and equal to 0.8 for bar size larger than 20mm.

18.2.2.4 Factor λ_2, or reinforcement spacing factor is calculated by:

$$\lambda_2 = \frac{c + k_{tr}}{1.8 d_b} \qquad (18\text{-}5)$$

c is the smaller of either the distance from the center of the bar to the nearest concrete surface or one-half the center-to-center spacing of the bars being spliced or truncated at the same location. K_{tr} is the index calculated by the Eq. (18-6), considering the area of transverse reinforcement available along the development length:

$$K_{tr} = \frac{A_{tr} f_y}{10sn} \qquad (18\text{-}6)$$

When n is the number of bars being developed or spliced at the same location. λ_2 shall not be taken greater than 1.4.

18.2.2.5 Factor λ_2, instead of being calculated by the Eq. (18-5), may be taken as follows:

a) In beams and columns.

- When concrete cover layer on reinforcement is greater than d_b and clear spacing of bars being spliced or truncated at the same location is equal to or greater than d_b and that the tie area along development length is at least equal to the values given in 12.6.3 and 8.4: $\lambda_2 = 0.85$

For other cases : $\lambda_2 = 0.60$

b) In slabs and other members.
- When concrete cover layer on reinforcement is greater than d_b and clear spacing of bars being spliced or developed at the same location is equal to or greater than $2d_b$: $\lambda_2 = 0.85$

For other cases : $\lambda_2 = 0.60$

18.2.3 Development of deformed bars in compression

18.2.3.1 Development of a deformed bar in compression, ℓ_{dc}, shall be computed by:

$$\ell_{dc} = \alpha_1 \alpha_2 \ell_{dcb} \quad (18\text{-}7)$$

Where ℓ_{dcb} is the basic development length to be calculated in accordance with 18.2.3.2; modification factors α_1 and α_2 are determined in accordance with 18.2.3.3 and 18.2.3.4.

18.2.3.2 Development length of bars in compression, ℓ_{dcb}, is calculated by the Eq. (18-2), incorporating bond strength between concrete and reinforcement equal to $1.5f_{bd}$ for deformed bars and $0.75f_{bd}$ for plain bars. Bond strength of concrete shall not be greater than 6.5 MPa.

18.2.3.3 Factor α_1 is used when reinforcement in the section is in excess of that required by analysis. The index is equal to ratio of reinforcement area required to reinforcement area provided.

18.2.3.4 Unless reinforcement is enclosed within spiral bars not less than 6mm diameter and not more than 100mm pitch or within No. 12 ties spaced at not more than 100mm on center, in conformance with tie placement requirements in compression members, in which case α_2 is taken equal to 0.75, factor α_2 is taken equal to 1 at all times.

18.2.4 Development length of bundled bars

18.2.4.1 Development length of three-bar and four-bar bundles in compression and tension shall be taken 1.2 and 1.3 times that of an individual bar, respectively. For two-bar bundles, development length need not be increased.

18.2.4.2 For determining development length of an individual bar in bundled bars, factors λ_1 and λ_2 in the Eq. (18-3) shall be based on diameter of a hypothetical bar with cross – sectional area equivalent to that of the whole bundle.

18.2.5 Development length of hooked bars in tension

18.2.5.1 Development length of a hooked bar in tension, ℓ_{dh}, shall be taken at least equal to:

$$\ell_{dh} = \beta_1 \beta_2 \beta_3 \, \ell_{dhb} \qquad (18\text{-}8)$$

Where ℓ_{dhb} is the basic development length of the hooked bar in tension, and factors β_1, β_2 and β_3 shall be determined in accordance with 18.2.5.2 through 18.2.5.6. ℓ_{dh} shall not be taken less than $8d_b$ or 150mm.

18.2.5.2 Basic development length of hooked bars in tension, ℓ_{dhb}, is calculated by the Eq. (18-2), incorporating bond strength between concrete and reinforcement equal to $1.5f_{bd}$ for deformed bars and $0.75f_{bd}$ for plain bars.

18.2.5.3 Factor β_1 is taken equal to 1 for all cases, except when concrete cover layer over the hook (normal to plane of hook) is thicker than 65mm for 180deg hook, and that concrete cover layer over the hook (normal to plane of hook and also in plane of hook) for 90deg hook is more than 65 and 50mm, respectively; in these cases β_1 is taken equal to 0.7.

18.2.5.4 Factor β_2 is taken equal to 1 for all cases, except when bars are enclosed within ties and stirrups spaced less than $3d_b$ along development length; in which case, factor β_2 may be taken equal to 0.8.

18.2.5.5 Factor β_3 is employed when reinforcement provided in the section is in excess of that required by structural analysis. The factor is equal to the ratio of reinforcement area required to reinforcement area provided. When development of reinforcement is done specifically for transmission of stress f_y, and also for structures with high ductility, discussed in chapter 20, factor β_3 is taken equal to 1.

18.2.5.6 For bars being developed by standard hook at discontinuous ends of members with side cover and top and bottom cover over hook (normal to plane of hook) less than 65mm, hooked bars shall be enclosed within ties or stirrups spaced along the full development length less than $3d_b$. Factor β_2 is also taken equal to 1.

18.2.6 Anchorage with mechanical devices

18.2.6.1 Any mechanical device capable of developing the strength of reinforcement without damage to concrete is allowed as anchorage. Adequacy of such mechanical devices to transfer forces shall be ensured through established tests and calculation methods.

18.2.6.2 Anchorage of bars in the concrete, in addition to considerations of mechanical device transfer capacity, may involve incorporation of induced bond along the full development length.

18.2.7 Development of welded deformed wire fabric in tension

18.2.7.1 Anchorage of welded deformed wire fabric in tension is usually done by providing development length on both sides of critical section, but for plain wire, in addition to development length, at least two cross wires within the development length is required. Spacing between nearest crosswire and the critical section should be greater than 50mm.

18.2.7.2 Development length of welded deformed wire fabric in tension is equal to the development length of an individual deformed wire in accordance with 18.2.2. For welded deformed wire fabric with at least one cross wire within the development length, spaced more than 50mm from the point of the critical section. Basic development length of wire, ℓ_{dh}, may be calculated by substituting (f_y – 140) for f_y in Eq. (18-2), provided that ℓ_{dh} is not taken less than $(1.6\frac{A_w}{s_w}.\frac{f_y}{f_{bd}})$. Development length in these fabrics shall not be taken less than 200mm, except for development at web, 18.3.4.2.c, and lap length at lap splices 18.4.5.

18.2.7.3 Development length of welded plain wire fabric in tension, considering limitation of 18.2.7.1 is measured from the point of the critical section to the outermost crosswire in free end of the fabric. Minimum development of these fabrics shall be computed by the Eq. (18-1), where k_1 and k_2 factors are taken equal to 1. In calculating ℓ_{dh} from the Eq. (18-2), f_b shall again be substituted by ($0.15f_{bd}$ $\frac{s_w}{bd}$). Except for computation of lap length at lap splices, according to 18.4.5, development length of fabric shall not be taken less than 150mm.

❐ 18.3 Principles for development of flexural reinforcement

18.3.1 General

18.3.1.1 Tension reinforcement in flexural members may be developed by considering limitations of 18.3.1.5 in tension zone of concrete, or by bending across the web to be anchored on the apposite face of member. The reinforcement may be used on the opposite face of the member as tension or compression reinforcement.

18.3.1.2 Critical sections, in flexural members, which are to be controlled on both sides for adequacy of reinforcement development are:
a) Sections of maximum stress.
b) Sections at which adjacent reinforcement terminates, or is bent.
For these members, in sections adjacent to simple supports and sections of inflection points, the deformation curve for changes in requirements of 18.3.2.3 shall also be satisfied.

18.3.1.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance at least equal to d or 12 d_b, whichever is greater, except at supports of simple spans and at free end of cantilevers.

18.3.1.4 Continuing reinforcement shall have embedment length at least equal to the development length ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

18.3.1.5 Flexural reinforcement shall not be terminated in a tension zone of concrete unless one of the following conditions is satisfied:
a) Resisting shear force of the section, V_r, at the point of cutoff is greater than the shear strength at the section, V_u, at least by 33 percent.
b) Transverse reinforcement area in excess of that required for shear and tension is provided along each terminated bar over a distance from termination point at least equal to 0.75d. Excess transverse reinforcement

area shall be at least equal to ($0.42 b_w \frac{s}{f_y}$). Spacing of transverse reinforcement in this zone shall not exceed $\frac{d}{8\beta_b}$ where β_b is the ratio of terminated reinforcement area to total area of tension reinforcement at the section.

c) Continuing reinforcement provides at least double the area required for flexure at the cutoff point and design shear strength at section, V_r , is greater than ultimate shear by at least 25 percent.

18.3.1.6 Adequacy of anchorage shall be controlled to be provided at different sections for tension reinforcement in flexural members where reinforcement stress in not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face.

18.3.2 Special provisions for development of positive moment reinforcement

18.3.2.1 At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150mm.

18.3.2.2 Where a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support in accordance with 18.3.2.1 shall be anchored to develop the specified yield strength f_y in tension at the face of support.

18.3.2.3 At simple supports and at points of inflection, in flexural members, positive moment tension reinforcement shall be limited to a diameter such that development length computed for f_y satisfies the following:

$$\ell_d \le \frac{M_r}{V_u} + \ell_a \qquad (18\text{-}9)$$

M_r is nominal moment strength at section, V_u is ultimate shear force at the section, ℓ_a, when the relation is being controlled at points of inflection shall be equal to d or $12d_b$, whichever is greater.

Eq. (18-9) need not be satisfied for reinforcement terminating beyond centerline of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

An increase of roughly 33 percent in the value of $\frac{M_r}{V_u}$ shall be permitted when the ends of positive reinforcement are confined by compressive reaction of support.

18.3.3 Special provisions for development of negative moment reinforcement

18.3.3.1 Negative flexural reinforcement in a continuous, restrained, or cantilever member, or in all members of continuous frames, shall be anchored in the supporting member by one of the methods (embedment length, hooks, or mechanical anchorage) stated in 18.2.1.1.

18.3.3.2 At least one-third the total negative flexural reinforcement provided at the support of a flexural member shall extend (as embedment length) beyond the point of inflection by at least d, $12d_b$, or one-sixteenth the net clear span, whichever is greater.

18.3.4 Special provisions for development of transverse reinforcement in flexural members web

18.3.4.1 Transverse reinforcement in flexural members' web shall be as close to the compression and tension surfaces of the member as concrete cover layer requirements and proximity of other reinforcement permits.

18.3.4.2 Both ends of single leg and U-shaped transverse reinforcement shall be anchored in accordance with one of the following methods:

a) For bars with 16mm or less diameter, for wires with 16mm or less diameter, and for bars with 16 to 25mm diameter of type S300 or lesser strength, standard hook shall be used. The hook shall encompass at least one longitudinal bar.

b) For 16 to 25mm diameter bars stronger than S300, in addition to the use of standard hook around at least one longitudinal bar, an embedment length equal to two-thirds the development length of hooked bar (requirements of 18.2.5) shall be used. Development length of the hooked bar is measured from effective midheight point of the member.

c) For each leg of welded plain wire fabric forming U-shape bend, one of the following two methods shall apply:
   - Two longitudinal wires spaced at a minimum 50mm spacing along the member at both ends of U.
   - One longitudinal wire located not more than $\frac{d}{4}$ from the compression face and a second wire distanced at least 50mm from the first wire and close to compression face of the member. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend greater than $8d_b$.

d) For each end of a single leg stirrup of welded plain or deformed wire fabric, two longitudinal wires at a minimum spacing of 50mm and with the inner wire at least the greater of $\frac{d}{4}$ or 50mm from the effective middepth of section, $\frac{d}{2}$. Outer longitudinal at tension zone of concrete shall not be farther from tension face than the portion of primary flexural reinforcement closest to the face.

18-3.4.3 At least one longitudinal bar shall be enclosed between anchored ends of simple U-stirrups or multiple U-stirrups in each bend within continuous portion of the stirrup.

18-3.4.4 Longitudinal bars bent to act as shear reinforcement, if extended into tension zone of concrete, shall act as tension reinforcement and, if extended into zone of compression, shall be anchored in this zone according to requirements of reinforcement embedment. Development length of the rods is measured from the effective middepth point, $\frac{d}{2}$.

18.3.4.5 For a pair of U-stirrups making a closed tie unit with the lap splice, development length shall be at least $1.3\ell_d$. In such stirrups with $A_b f_y$ less than 40 KN per leg, development length shall be taken less than $1.3\ell_d$, provided that each leg extends from the full length from U to the opposite face.

❒ 18.4 Splice of reinforcement

18.4.1 General

18.4.1.1 Splices of reinforcement shall be permitted by one of the following four methods:

a) Lap splice: is practiced on part of the length of two adjacent bars. The length at which two bars are located next to each other is called "Lap splice".

b) Welded splice: is carried out by welding together two bars.

c) Mechanical splice: is carried out by employing special mechanical devices.

d) End-bearing splice: is carried out by overlapping two ends of compression bar.

18.4.1.2 Lap splices are permitted only for bars with less than 36mm diameter.

18.4.1.3 Entire bundles shall not be lap spliced as one bundle, but bars may individually be lap spliced together. Individual bar splices within a bundle shall not overlap.

18.4.1.4 Lap length required for lap splice of each two bars in a bundle shall be based on lap splice length required for individual bars within the bundle, and requirements of 18.2.4 shall be satisfied.

18.4.1.5 Any two bars connected by lap splices in flexural members shall not be spaced farther apart than one-fifth the required lap splice length, nor 150mm.

18-4.1.6 Welded splices in bars shall be made as one of the methods prescribed in 8.2.5.3. Unless requirements of 18.4.2.2 are satisfied, strength of such splices in compression and tension shall be at least equal to $1.25A_b f_y$.

18.4.1.7 Unless requirements of 18.4.2.2 are satisfied, mechanical splices of bars shall have strength in tension and compression at least equal to $1.25A_b f_y$.

18.4.2 Splices of deformed bars and deformed wire in tension

18.4.2.1 Minimum lap length for tension lap splices shall be $1.3\ell_d$. When both of the following two conditions are met simultaneously, lap length may be decreased by ℓ_d:

a) The area of reinforcement provided in lap length zone is at least twice that required, and

b) At most, one-half of the area of reinforcement provided in the section is spliced within the required lap length.

Where ℓ_d is the tensile development length that shall be calculated in accordance with 18.2.2. In calculating ℓ_d, factor k_3 shall be taken equal to 1. Lap length shall not be taken less than 300mm.

18.4.2.2 For welded or mechanical splices, when the area of reinforcement provided at section is less than twice that required, splice strength shall be equal to $1.25A_b f_y$, but for other cases, splice strength may be taken less than the above value and in accordance with the following requirements:

a) Splice strength of each bar shall be such that total reinforcement provided at the section can withstand a force at least equivalent to twice the required forces. The force shall not be less than $140A_b$ for total reinforcement considered. Splices in different sequential sections shall be spaced at least 600mm.

b) Tensile force considered in part (a) shall be computed as following:
   - For spliced reinforcement, equal to strength force of splice.
   - For unspliced reinforcement, equal to that fraction of $A_b f_y$ lowered by an amount equal to the ratio of the shorter actual development length to the required development length.

18.4.2.3 Reinforcement splices in tension members shall only be made with a mechanical or welded splices, and that the requirements of 18.4.1.6 or 18.4.1.7 shall satisfy. Splices in adjacent bars shall be spaced more than 750mm.

18.4.3 Splices of deformed bars in compression

18.4.3.1 For reinforcement of the type S400 or with lesser strength, lap splice length shall be at least $0.07 f_y\, d_b$ and for stronger steel, it shall be $(0.13 f_y - 24)$ d_b. The lap length shall be taken less than 300mm. Lap length shall be increased by 33 percent when concrete strength is less than 20 MPa.

18.4.3.2 When bars of different size are lap spliced, lap length shall be the larger of either development length of large bar, or splice length of smaller bar. Reinforcement larger than 36mm in diameter may be spliced with lesser than 36mm size bars.

18.4.3.3 Welded splices and mechanical connection shall have splice strength in accordance with 18.4.1.6 or 18.4.1.7.

18.4.3.4 For end-bearing splices in which compression transmission among reinforcement is made by bearing of square cut ends held in concentric contact by a suitable device. Bar ends shall terminate in flat surfaces within 1.5 deg of a right angle to the axis of the bars and shall be fitted within 3 deg of full bearing after assembly. End-bearing splices shall be permitted only in members containing closed transverse stirrups, or spirals.

18.4.4 Special reinforcement splice requirements for columns

18.4.4.1 Reinforcement splice in columns shall be of the type: Lap splices, butt-welded splices, mechanical splices, or end-bearing splices. Reinforcement splices shall satisfy requirements for all load combinations for the column.

18.4.4.2 Lap splice in compression reinforcement satisfies requirements of splice in compression, and the lap splice in tension reinforcement satisfies requirements of splices in tension. When stress in tension reinforcement is less than $0.5f_y$, and the number of bars spliced within lap length is less than half tension reinforcement, lap length shall be at least equal to ℓ_d, otherwise it is taken at least equal to $1.3\ell_d$. In this case, splice spacing for different bars shall not be taken less than ℓ_d.

18.4.4.3 In compression members where transverse reinforcement in the form of ties throughout the lap splice length have cross-sectional area greater than

0.0015 hs, lap length shall be permitted to be reduced by 20 percent and when transverse reinforcement is in the form of spiral, lap length may be reduced by 25 percent; but it shall not be taken less than 300mm. Cross-sectional area of the legs perpendicular to dimension h shall solely be used in computing cross-sectional area of stirrups.

18.4.4.4 Mechanical or welded splices in columns shall meet the requirements of 18.4.1.6 or 18.4.1.7.

18.4.4.5 End-bearing reinforcement splices shall be used in columns in accordance with requirements of 18.4.3.4, provided that either this type of splice is used for any number of bars present at different sections, or additional bars are provided at splice location. Accordingly, strength of the bars continuing into splice location shall be at least equal to one-fourth the strength $A_b f_y$ for all reinforcement in the column.

18.4.5 Splices of welded wire fabric in tension

18.4.5.1 When deformed wire is used in such fabric, lap splice shall conform with the following requirements:

a) When there exists at least one cross wire within lap splice zone in each fabric, and spacing between any two adjacent fabric is greater than 50mm, lap length shall be equal to $1.3\ell_d$, or 200mm, whichever is greater. ℓ_d, development length of the fabric shall be based on requirements of 18.2.7.2.

b) When there is no cross wires within the lap splice length, lap length shall be determined in accordance with requirements of such lap splice in wires.

18.4.5.2 When all plain wires are present in the fabric, lap splice shall meet the following requirements:

a) When area of reinforcement provided is less than twice that required, spacing between outermost two cross wires in two fabrics within the overlap region shall be at least equal to the greater of the following three values: one spacing of cross wires in the fabric plus 50mm, $1.5\ell_d$, or 150mm.

b) When area of reinforcement provided is greater than twice that required, spacing between outermost two cross wires in two fabrics within the overlap region shall be at least equal to $1.5\ell_d$ or 50mm.

In (a) and (b), length of fabric overlap shall be in accordance with 18.2.7.3.

CHAPTER NINETEEN

SAFETY EVALUATION OF EXISTING STRUCTURES

❒ 19.0 Notation

a = maximum deflection, under load test, of a member relative to the joint line between two ends of span, or deflection of the free end of a cantilever relative to its support, mm.

h = overall thickness of member, mm.

ℓ_t = span of member under load test (the shorter span of the flat slabs and of slabs supported on four sides) is equal to the distance between centers of supports, or to the clear distance between supports plus depth of member whichever is smaller, mm (except for 19.1.4.9).

❒ 19.1 Scope

19.1.1 Provisions of this chapter shall apply to safety evaluation of existing reinforced concrete structures, all or portion of which do not satisfy limit state conditions for strength and serviceability, or there is doubt about satisfaction of the conditions, a safety evaluation may be required for the following conditions:

- Structures where analysis, design or construction of all or portion of themdo not satisfy the requirements of this code.

- Structures considered for a service change in conditions.
- Structures subjected to damages or deterioration.
- Structures that require re-evaluation due to their inconformity with code conditions.

19.1.2 Safety evaluation structures, wherever applicable, is limited to all or a portion of structure.

19.1.3 Safety evaluation of structures entails investigation and control of all the requirements stated in the present code, including limit states of strength and serviceability.

19.1.4 According to this code, safety evaluation of structures is carried out by analytical method or by load tests or a combination there of, in accordance with 19.2 through 19.6.

19.1.5 For other than flexural members, evaluation by analytical method is preferred.

❒ 19.2 Analytical method

When safety evaluation is to be carried out by analytical method, a close site investigation shall be done regarding dimensions and details of members, actual strengths and characteristics of the materials used, and other conditions related to structure as it is built, and also considerations related to loading and structural analysis, in accordance with 19.2.1 through 19.2.6.

19.2.1 The actual strengths and characteristics of the materials used in the structure shall be determined through tests and statistical investigations of their outcome results. Number of tests shall be adequately abundant so that it makes it possible to compute the specified strength and other character is tics of the materials in place with a definite level of assurance.

19.2.2 Dimensions of different sections of the structure shall be determined through on-the-site measurements, and any increase or decrease in dimensions relative to implementation plans of the structure due to construction deficiencies, corrosion, deterioration, or alike shall be considered in analytical investigations.

19.2.3 Location, type, and size of reinforcement shall be determined on the site by local destructive method, nondestructive method, or a combination thereof. The operations volume shall be such that it would be possible to determine, through usage of information gained and available site data, load-bearing capacity of the sections and members with adequate assurance.

19.2.4 Structural analysis model shall represent the actual situation of structure or portion of it with respect to boundary conditions.

19.2.5 Unless on the basis of accepted statistical evaluations and investigations a lower value is specified, reduction of loads and over loads relative to code values is not permitted. In special cases where there is deficiency and ambiguity in the load code, and that the user does not choose the value, it may be determined through responsibility and judgement of an expert.

19.2.6 Structural analysis and comparison of load effects shall be based on the methods prescribed in this code. When the results of analyses carried out using approximate methods is not satisfactory, a more exact analysis shall be done.

❒ 19.3 Investigation by loading test - General

19.3.1 When safety evaluation is done by load tests, a qualified engineer approved by inspection body shall control these tests.

19.3.2 A portion of the structure that is to undergo load tests, shall not be subject to load for at least 56 days after being constructed, unless the clients, the contractor and all involved parties agree to make the test at an earlier age.

19.3.3 When it is intended that only a portion of the structure be subjected to load test, the portion shall be so tested that the suspicious weakness factor is well investigated.

19.3.4 In order to incorporate the effects of dead load not yet applied on the structure by the time of load test, it is required that such load be applied on the structure 48 hours before application of load test, and that it remains on the site until the end of all load tests.

❒ 19.4 Load tests for flexural members

19.4.1 When it is intended that flexural members, including beams and slabs be load tested, requirements of 19.4 and 19.5 shall also be satisfied, in addition to other regulations.

19.4.2 The basis for readings (a basis for displacement measurement) shall be specified immediately prior to application of load test.

19.4.3 That portion of the structure selected for loading shall be subjected to 95 percent ultimate dead load (including dead load already in place) and live load. Determination of live load with consideration of overload decrease shall be based on valid loading codes.

19.4.4 Test load shall be applied in at least four steps with an almost uniform increase in each stop without applying impacts on the structure. Arching performance of the loading material shall be prevented.

19.4.5 Preliminary readings shall be taken at least 24 hours after application of test loads.

19.4.6 Tests load shall be removed immediately after preliminary reading. Final deflection readings shall be taken at least 24 hours after removal of test load

19.4.7 When failure or rupture is observed in that portion of structure subjected to load test, it may be concluded that the portion considered has failed tests and therefore shall not be allowed further load tests.

19.4.8 When failure or rupture is not readily visible in the portion of the structure subject to test, the following criteria shall be adopted as a sign of satisfactory behavior:

a) When maximum measured displacement, a, in beam, floor or roof is smaller than $\frac{\ell_t^2}{20000\text{h}}$.

b) When maximum measured displacement, a, in beam, floor or roof exceeds $\frac{\ell_t^2}{20000\text{h}}$, but return displacement within 24 hours after removal of test load is at least 75 percent of the maximum displacement.

For other cases, proper measures shall be taken in accordance with 19.6.

19.4.9 In articles 19.4.8.a and 19.4.8.b, for cantilever components, ℓ_t shall be taken as twice the distance between support and the end of cantilever. Displacement value shall be corrected on the basis of any possible displacement at the support.

❒ 19.5 Safety measures

19.5.1 Load test shall be conducted in such a manner as to provide for safety of life and structure during the test.

19.5.2 No safety measures shall interfere with load test procedures or affect results.

❒ 19.6 Unsafe structures

If in accordance with 19.2 through 19.5, some aspects of safety, serviceability or other requirements of this code are not satisfied with regard to the structure considered, special retrofitting measures shall be taken so as to permit the authorities to consider the structure as resistant.
The measures shall be carried out by structural expert. Structural expert refers to a qualified design engineers or executive engineers who is more capable than a usual designer.

CHAPTER TWENTY

SPECIAL PROVISIONS FOR SEISMIC DESIGN

❒ 20.0 Notation

A_g = gross area of section, mm^2.

A_c =a portion of cross-sectional area enclosed by spirals. The area is measured out-to-out the diameter of spiral, mm^2.

A_{ch} = a portion of cross-sectional area enclosed by transverse reinforcement, measured out-to-out of transverse reinforcement, mm^2.

A_{cp} =area of concrete section or an individual pier or horizontal wall segment resisting shear, mm^2.

A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm^2.

A_J = minimum cross-sectional area within a joint in a plane parallel to axial plane of reinforcement generating shear in the joint, mm^2.

The depth of this section shall be equal to the overall depth of the column. When the girder connects to a larger width support, effective width of the joint shall be taken the smaller of the following two values:

a) Beam width plus total depth of the joint section.

b) Twice the smaller perpendicular distance from the longitudinal axis of the beam to the outside column side.

A_{sh} = total cross-sectional area of transverse reinforcement (including single leg u-stirrups) within spacing S in the direction perpendicular to dimension h_c, mm^2.

A_v = total cross-sectional area of shear reinforcement within spacing s in the direction perpendicular to longitudinal axial of the member, mm^2.

A_{vd} = cross-sectional area of diagonal bars (refer to 20.5.3.4.2), mm^2.

b = effective compression flange width, mm.

d = effective depth of section, mm.

f_{bd} = basic bond strength of concrete, MPa.

f_c = specified compressive strength of concrete, MPa.

f_y = specified yield strength of reinforcement (f_{yk}), MPa. For simplicity, letter k is omitted form subscript, in this notation.

f_{yh} = specified yield strength of transverse reinforcement, MPa.

h_c = cross-sectional diameter of column core measured center-to-center of confining reinforcement, mm.

h_w = total height of wall or diaphragm, height of a segment of wall or diaphragm, mm.

ℓ_d = development length of a straight bar, mm.

ℓ_{dh} = development length of a hooked bar, mm.

ℓ_o = length of the critical area over which, special transverse reinforcement must be provided, mm.

ℓ_w = length of entire wall or diaphragm or that of a segment of wall or diaphragm considered, in the direction of shear effect, mm.

M_e = refer to 20.5.2.4.1, N.mm.

M_g = refer to 20.5.2.4.1, N.mm.

M_n = nominal flexural moment for strength, refer to 20.2.1, N.mm.

M_{pr} = probable flexural moment for strength, N.mm.

M_r = flexural moment of strength at section, N.mm.

N_u = ultimate axial force at the section, N.

s = spacing of transverse reinforcement layers along the longitudinal axis of the structural member, mm.

V_r = shear force strength at section, N.

V_u = ultimate shear force at section, N.

v_c = shear strength of concrete – Eq. (12-4), MPa (N/mm^2).

α_c = refer to 20.5.5.2.2.

ϕ_c = partial safety factor for concrete.

ϕ_s = partial safety factor for steel reinforcement.

ϕ_m = strength modification factor.

ρ_s = ratio of volume of spiral reinforcement to the volume of concrete core confined by the spiral reinforcement (measured out-to-out).

ρ_v = ratio of area of distributed reinforcement perpendicular to the plane of A_{cv} to gross concrete area A_{cv}.

ρ_n = ratio of area of horizontal shear reinforcement to a plane perpendicular to the shear plane A_{cv}.

❒ 20.1 Scope

20.1.1 Provisions of this chapter shall apply for design and construction of reinforced concrete members of the structures for which the design forces related to earthquake motions have been determined on the basis of energy dissipation in the nonlinear range of response of structures.

20.1.2 In designing structures subject to this chapter, observance of the criteria in other chapters of this code, except when stated in some other way throughout this chapter is required.

20.1.3 In designing structures subject to this chapter, requirements of chapter may be ignored, provided that laboratory and analytical evidence prove that resistance of structure against cyclic loads is less that the values given for this structure, designed based on this chapter's criteria.

❒ 20.2 General design criteria

20.2.1 Definitions

Special transverse reinforcement placement
Transverse reinforcement placement for members subjected to compression and flexure, carried out in accordance with requirements of 20.5.2.3.2 through 20.5.2.3.6.

Members subjected to compression and flexure and members under flexure
Members subjected to compression and flexure are those in which axial compression force is greater than $0.15\phi_c f_c\ A_g$. If the ultimate axial compression force is less than the above value, the member is considered to be flexural.

Collector elements
Elements that serve to transmit part of the seismic inertial forces within structural diaphragms to members of the lateral-force-resisting system.

Boundary elements
Elements along structural wall and structural diaphragm edges strengthened by longitudinal and transverse reinforcement. These elements may either be equal in thickness with, or greater than walls or diaphragms. Edges of openings within walls and diaphragms shall be provided with boundary elements as required.

Shell concrete
Concrete in the portion of member section outside the segment confined by transverse reinforcement, or the core area.

Base level
Level at which earthquake motions are assumed to be imparted to a building. For higher levels, the building has its own separate motion relative to the ground. Base level does not necessarily coincide with the ground level.

Hoop
A closed tie made up of several reinforcement elements each having special seismic hooks at both ends. A hoop can be a continuously wound tie that has seismic hook at both ends.

Structural diaphragms
Structural members like floor and roof slabs that transmit inertial forces to lateral-force resisting members.

Structural walls
Walls proportioned to resist combinations of shears, flexural moments, and axial forces, induced by earthquake load and gravity loads acting on their middle plate.

Shear wall
Shear wall is a type of structural wall.

Coupled walls
Structural elements composed of two or more individual shear walls that are tied (or connected) together in special arrangement by adequately ductile beams (coupler beams).

Seismic hook

A hook having a bend at least 135deg and an extension length at least eight times the bar diameter or 100mm that engages the longitudinal reinforcement and projects into the interior of the stirrup.

Lateral-force resisting system

That portion of structure computed to resist seismic lateral forces.

Ductility

The ability to dissipate energy by inelastic behavior of the entire structure or its members while subjected to large-scale cyclic deformations without considerable reduction in strength.

Crosstie

A continuous reinforcement bar having a hook not less than 135deg and a straight extension length at least eight times the bar diameter or 100mm at one end and a hook not less than 90deg with an eight-diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90deg hooks of successive crossties engaging the same longitudinal bars shall be alternated end for end.

Tie elements

Elements usually in the form of tension members that serve to transmit inertia forces and prevent separation of building components such as footings and walls.

Nominal flexural strength

Nominal flexural strength in a section of flexural members or of those subjected to compression and flexure is the largest moment that the member can withstand at that section. Based on requirements of chapter 11, this moment is assumed equal to 1.15 times the flexural moment for strength of the section. That is: $M_n = 1.15M_r$.

Probable flexural moment for strength
Probable flexural moment for strength is equal to the flexural moment for strength assuming .
$f_s = 1.25f_y$ and $\phi_c = \phi_s = 1$ (f_s is the reinforcement strength).

Critical zone
A zone with the probability of plastic hinge due to earthquake loads.

Compression member core
A portion of the cross-sectional area of member between confining transverse reinforcement, out-to-out.

Plastic hinge
A section of the member in which tension reinforcement has reached yield state.

Plastic region
A portion of member in which plastic rotation occurs.

20.2.2 Analysis of structure and of its members dimensions

20.2.2.1 The interaction of all structural and non-structural members that materially affect the linear and nonlinear response of the structure to lateral loads shall be considered in the analysis.

20.2.2.2 Rigid members assumed not be a part of the lateral-force resisting system shall be permitted provided that their effect on the response of the system is considered and accommodated in the structural design. Consequences of probable failure of structural and non-structural members, which are not a part of the lateral-force resisting system shall also be considered.

20.2.2.3 Structural members below base of structure that are required to transmit forces resulting from earthquake effects to the foundation shall be designed based on the requirements of this chapter.

20.2.2.4 Strength and toughness of structural members located between base level and footing shall not be less than the corresponding values in members above base level.

20.2.2.5 In structures designed for high ductility limit state, all structural members assumed not to be part of the lateral seismic-force-resisting system shall be designed in accordance with requirements of 20.5.6.

20.2.3 Material specification

20.2.3.1 Compressive strength of concrete in earthquake resisting members shall not be taken less than 20 MPa .

20.2.3.2 Specified yield strength of reinforcement in frame members and in structural wall boundary elements that are applied to resist earthquake induced lateral forces shall not be taken greater than 400 MPa (N/mm^2). Furthermore, the following two conditions shall satisfy for these steel bars:

a) The actual yield strength based on mill test does differ from the specified yield strength by more than 125 MPa .
b) The ratio of failure strength of steel to its specified yield strength shall not be less than 1.25.

20.2.3.3 Longitudinal reinforcement in all structures, regardless of their ductility, and transverse reinforcement in high ductility structures shall be of deformed type

20.2.3.4 Application of welded connections in longitudinal reinforcement is only permitted if the required conditions of 20.5.1.2.7 and 20.5.2.2.4 are satisfied. In addition, welding of stirrups and other similar elements to longitudinal reinforcement shall not be permitted.

20.2.4 Structural control in limit state of serviceability

20.2.4.1 Building as a whole, including structural and non-structural elements shall be designed such that when subjected to earthquake lateral loads in limit state of serviceability, no damages occur to the elements, as much as possible, and also no restrictions are imposed at service time. Therefore, limitation of lateral displacements induced by earthquake loads to an extent appropriate for structural and non-structural elements of conventional buildings is considered adequate.

20.2.4.2 For high importance buildings like hospitals and factories, in addition to limiting lateral displacements, special provisions shall be applied so as to mitigate damages incurred on equipment and machinery to a minimum.

20.2.5 Structural ductility limits

20.2.5.1 Earthquake lateral-load resisting elements shall be designed for one of the three ductility limits defined in 20.2.5.2 through 20.2.5.4. Design criteria for each of these limits are described in 20.3 through 20.5.

20.2.5.2 Low ductility limit: This limit is appropriate for structures which are not expected to undergo large deformation such that special provisions to ensure their safety against seismic recurring and cyclic loads would suffice.

20.2.5.3 Moderate ductility limit: This limit is required for structures where structural response to seismic loads extends into the nonlinear region; in that, structural sections shall be so designed as to sustain adequate safety against brittle failure.

20.2.5.4 High ductility limit: This limit is required for structures in which, members shall have high capacity to absorb and dissipate energy so that in case of approaching mechanism, general stability and solidarity of the structure is preserved and adequate assurance in this respect is provided.

20.2.5.5 Structures in which ductility limits are provided more intensely, may be designed for lesser earthquake lateral loads. The criteria to decrease these loads are prescribed in the Code of Practice for Seismic Resistant Design of Buildings (Standard 2800).

❒ 20.3 Principles of low ductility structures

20.3.1 The ratio of tension reinforcement at any section of flexural members shall be not less than $\frac{1.4}{f_y}$ and $\frac{0.25\sqrt{f_c}}{f_y}$, nor greater that 0.025.

20.3.2 For members subjected to flexure and compression, ratio of longitudinal reinforcement shall not be taken less than one percent and greater than six percent. Maximum reinforcement limit at the splices shall also be observed. When longitudinal bar is of S400 type steel, ratio of reinforcement outside splices is limited to 4.5 percent maximum.

20.3.3 For members subjected to flexure and compression, center-to-center spacing of longitudinal reinforcement shall not be greater than 200 mm.

20.3.4 For beam-column joints along the largest depth of the beam or slab, ending in the joint, transverse reinforcement at least equal to the following value shall be incorporated in the direction perpendicular to longitudinal bars in the column:

$$A_v = 0.35\frac{bs}{f_y} \qquad (20\text{-}1)$$

In this equation b is the greater dimension of rectangular shaped section of the column or cross-sectional diameter of circular-shaped section of the column.

❐ 20.4 Special criteria for structures with moderate ductility

20.4.1 Frame members subjected to flexure $(N_u \le \phi_c f_c A_g)$

20.4.1.1 Geometric constraints

20.4.1.1.1 The following geometric constraints shall be observed for flexural frame members:

a) Effective depth of the members shall not exceed one-fourth its clear span.
b) Section width shall not be less than one-fourth its depth.
c) Section width shall not be taken:
   - More than the width of the supporting column, measured on a plane perpendicular to the longitudinal axis of the flexural member, plus three-fourths the depth of the flexural member on each side of the column.
   - More than the width of the supporting column, plus one-fourth the other cross-sectional dimension of the column on each side of it.
   - Less than 250 mm.

20.4.1.1.2 Eccentricity of each flexural member relative to the column with which it forms a frame; that is, the distance between geometric axes of two members shall not exceed one-fourth the cross-sectional width of the column.

20.4.1.2 Longitudinal and transversal reinforcement

20.4.1.2.1 Reinforcement ratio at any section of a flexural member, for top as well as for bottom reinforcement shall not be less that $\frac{1.4}{f_y}$ and $\frac{0.25\sqrt{f_c}}{f_y}$; also tensile reinforcement ratio shall not exceed 0.025. At least two bars of equal 12 mm diameter shall be provided simultaneously and continuously both at top and at bottom of the member.

20.4.1.2.2 At the supports of the flexural member and at any section where possibility of plastic hinge formation exists, compression reinforcement equal to one-third the amount of tensile reinforcement shall be provided at that section.

20.4.1.2.3 At least one-fifth of the top reinforcement at the support section of the flexural member, at any end with greater amount of bars, shall be continuous throughout the length of beam at top and at bottom.

20.4.1.2.4 Unless shear design indicates the need for more reinforcement, hoops shall be used along the following critical segments of the flexural members in accordance with 20.4.1.2.5:

a) Over a length equal to twice the member depth, measured from the face of the supporting member toward midspan, at both ends of the flexural member.

b) Over a length equal to twice the member depth on both sides of a section where plastic hinge is likely to occur in connection with inelastic lateral displacements of the frame.

c) Over a length in which, provision of flexural capacity at the section requires compression reinforcement.

20.4.1.2.5 Stirrups and their spacing shall have the following conditions:

a) Diameter of hoops shall not be less than 6 mm.

b) Maximum spacing of the hoops shall not exceed one-fourth the effective depth of the section, eight times the diameter of the smallest longitudinal bars, 24 times the diameter of the hoop bars, and 300 mm.

c) The first hoop shall be located not more than 50 mm from the face of a supporting member.

20.4.1.2.6 In segments of the flexural member length not hooped in accordance with 20.4.1.2.5, tie spacing shall not exceed one-half the effective depth of the section.

20.4.2 Frame members subjected to flexure and compression-columns *($N_u \geq \phi_c f_c A_g$)*

20.4.2.1 Geometric constraints

20.4.2.1.1 The following geometric constraints shall apply to columns:

a) Section width shall not be less than three-tenths of its other dimension and not less than 250 mm.

b) Clear span-to-width ratio of a column section shall not exceed 25.

20.4.2.2 Longitudinal and transversal reinforcement

20.4.2.2.1 Longitudinal reinforcement ratio in columns shall not be less than one percent and not greater than six percent. Maximum reinforcement limit at the splices shall also be observed. When longitudinal bar is of S400 type steel, reinforcement ratio outside splices is limited to 4.5 percent maximum.

20.4.2.2.2 Center-to-center spacing of longitudinal reinforcement shall not exceed 200 mm.

20.4.2.2.3 Unless shear design indicates the need for more reinforcement, transverse reinforcement in amount specified in 20.4.2.2.2 shall be provided over a length ℓ_o from both ends of column. The length ℓ_o, critical zone, measured from of lateral members joint face shall not be less than the following values:

a) One-sixth the clear height of the column.

b) Greater dimension of rectangular-shaped section, or diameter of circular-shaped section of the column.

c) 450 mm.

20.4.2.2.4 Required transverse reinforcement over the length ℓ_o shall have a diameter at least 8 mm, and their spacing is determined from 11.9.3, when used as spiral. Transverse reinforcement spacing, when used as hoops, shall be less than the following values:

a) eight times the diameter of the smallest longitudinal bar.
b) 24 times the diameter of the hoop bars.
c) one-half the smallest cross-sectional dimension of the column.
d) 250 mm.

Distance between the first hoop and beam-column joint face shall not exceed one-half the above values.

20.4.2.2.5 For segments of the column length not including the length ℓ_o, transverse reinforcement segments criteria is similar to those of usual columns.

20.4.2.2.6 Columns that support reactions from stiff members such as reinforced concrete walls shall be provided with transverse reinforcement over their full height in accordance with 20.4.2.2.4. In addition, transverse bars shall extend for at least the development length along part of the longitudinal reinforcement of the column that extends through the wall. The criteria for transverse reinforcement extension into the wall shall also apply to columns that terminate on a wall.

20.4.2.2.7 At the column-footing joint, longitudinal reinforcement extended into the footing shall be strengthened with transverse reinforcement for a length at least equal to 300mm.

20.4.3 Structural walls, diaphragms and trusses

20.4.3.1 For structural walls, diaphragms and trusses, requirements of 20.5.3.1 through 20.5.3.3 in connection with high ductility structures shall be observed considering exceptions of 20.4.3.2 and 20.4.3.3.

20.4.3.2 Wherever necessary, instead of special transverse reinforcement when required by 20.5.3.1 through 20.5.3.3, reinforcement shall be done in accordance with requirements of 20.4.2.2.4.

20.4.3.3 Requirements of 20.5.3.2.6 need not be observed for reinforcement anchorage and splicing. Splicing and anchorage shall be carried out in accordance with requirements of chapter 18.

20.4.4 Beam-column joints in frames

20.4.4.1 For beam-column joints, over the depth of the deepest beam or slab that terminates in the joint, transverse reinforcement at least equal to the following values shall be provided in the direction perpendicular to the longitudinal reinforcement of the column:

a) Cross-sectional area of transverse reinforcement shall not be less than the value calculated by the Eq. (20-1).

b) Amount of transverse reinforcement shall not be less than two-thirds the transverse reinforcement available in the ℓ_o zone of the column, according to 20.4.2.2.4. Spacing of these reinforcement layers shall not exceed 1.5 times that of corresponding layers in the same ℓ_o zone.

20.4.5 Principles of shear design in frame members

20.4.5.1 For frame members subjected to flexure or to combination of compression and flexure, ultimate limit state of strength shall be based on Eq. (12.1). The value of V_u in the equation is equal to greater of the following two values:

a) Shear forces generated in the member, considering static equilibrium of gravity loads, if any, and flexural moments in its and sections. Flexural capacity of plastic hinges, positive or negative, shall be considered together with nominal flexural moment for strength M_n at the section. In determining these moments, the most adverse ultimate axial force in the member, which develops greatest flexural moment shall be incorporated. Directions of these moments shall be chosen so that maximum force is induced in the member.
b) Shear force due to ultimate reactions of gravity loads and earthquake-induced lateral forces, assuming that earthquake forces effecting the structure are two times the values determined in the Code of Practice for Earthquake Resistant Design of Buildings (Standard 2800).

❒ 20.5 Principles of high ductility structures

20.5.1 Frame members subjected to flexure ($N_u \leq 0.15\phi_c f_c A_g$)

20.5.1.1 Geometric constraints

20.5.1.1.1 The following geometric constraints shall be observed for flexural frame members:

a) Effective depth of the member shall not exceed one-fourth its clear span.
b) Section width shall be less than three-tenths its depth.
c) Section width shall not be taken:
   - More than the width of the supporting column, measured on a plane perpendicular to the longitudinal axis of the flexural member, plus three-fourths the depth of the flexural member on each side of column
   - More than the width of the supporting column, plus one-fourth the other cross-sectional dimension of the column on each side of it .
   - Less than 250 mm.

20.5.1.1.2 Eccentricity of each flexural member relative to the column with which it forms a frame; that is, the distance between geometric axes of the two members shall not be greater than one-half the width of the column section.

20.5.1.2 Longitudinal reinforcement

20.5.1.2.1 Reinforcement ratio at any section of a flexural member, for top as well as for bottom reinforcement shall not be less than $\frac{1.4}{f_y}$ and $\frac{0.25\sqrt{f_c}}{f_y}$; also tensile reinforcement ratio shall not exceed 0.025. At least two bars of equal 12 mm diameter shall be provided simultaneously and continuously both at top and at bottom of the member.

20.5.1.2.2 At the supports of the flexural member and at any section where possibility of plastic hinge formation exists, compression reinforcement equal to one-half the amount of tensile reinforcement shall be provided at that section.

20.5.1.2.3 At least one-quarter of the top reinforcement at the support sections of the flexural member, at any end with greater amount of bars, shall be continuous throughout length of the beam at top and at bottom.

20.5.1.2.4 For T-or L-shaped flexural members that are built integrally with slabs, the amount of reinforcement assumed for effective flexure at the face of columns, in addition to the bars in the beam web, is as follows:

a) For inner columns where the dimensions of transverse beam at the column connection is roughly equal to those of longitudinal flexural member: all bars located in a width of slab equal to four times its thickness on each side of the column.

b) For inner columns where there is no transverse beam, all bars located in a width of slab equal to 2.5 times its thickness on each side of the column.
c) For outer columns where transverse beam at column connection is roughly equal to dimensions of longitudinal flexural member that are required to be anchored, all bars located in a width of slab equal to 2 times its thickness on each side of the column.
d) For outer columns where there is no transverse beam, all bars located across the width of column.
e) In all cases, at least 75 percent of the top and the bottom reinforcement that provide the required flexural capacity shall extend through the column core and shall be anchored in it.

20.5.1.2.5 Application of lap splices in longitudinal flexural reinforcement is permitted only on conditions where hoop or spiral reinforcement is provided over the entire lap length. Spacing of the transverse reinforcement enclosing the lapped bars shall not exceed one-fourth the effective depth of the section or 100 mm.

20.5.1.2.6 Lap splices shall not be used in the following locations:
a) Within beam-column joints.
b) Within a distance twice the member depth from the face of the joint.
c) At locations where the possibility of plastic hinge formation caused by inelastic lateral displacement of the frame exists.

20.5.1.2.7 Welded or mechanical splices complying with 18.4.1.6 and 18.4.1.7 are permitted if reinforcement splice in each layer is provided alternately and the splice spacing in adjacent bars is less than 600 mm in the direction of member length.

20.5.1.3 Transverse reinforcement

20.5.1.3.1 Unless shear design indicates the need for more reinforcement, hoop type transverse reinforcement satisfying conditions of 20.5.1.3.2 shall be provided along the following critical segments of flexural members:

a) Over a length equal to twice the member depth, measured from the face of the supporting member toward midspan, at both ends of the flexural member.
b) Over a length equal to twice the member depth on both sides of a section where plastic hinge is likely to occur in connection with inelastic lateral displacements of the frame.
c) Over a length in which, provision of flexural capacity at the section requires compression reinforcement.

20.5.1.3.2 Hoops and their spacing shall have the following conditions:

a) Hoop diameter shall not be less than 8 mm.
b) Maximum spacing of the hoops shall not exceed one-fourth the effective depth of the section, eight times the diameter of the smallest longitudinal bar, 24 times the diameter of the hoop bars, and 300 mm.
c) The first hoop shall be located not more than 50 mm from the face of a supporting member.

20.5.1.3.3 In segments of the flexural member length where hoops are required in accordance with 20.5.1.3.1, longitudinal reinforcement on the perimeter of the section shall have lateral support conforming to 8.4.3.5.

20.5.1.3.4 In portions of the flexural member length where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance less than or equal to one-half the effective depth of the member section.

20.5.1.3.5 Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a U-shaped stirrup having seismic hook at both ends and closed by a crosstie, which forms a hoop with the first stirrup. Consecutive crossties engaging the same longitudinal bar shall have their 90deg hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the crossties are confined by a slab on only one side of the flexural frame member, the 90deg hooks of the crossties shall be placed on that side.

20.5.2 Frame members subjected to flexure and compression-column *($N_u \geq \phi_c f_c A_g$)*

20.5.2.1 Geometric constraints

20.5.2.1.1 The following geometric constraints shall apply to columns:

a) Section width shall not be less than four-tenths its other dimension and not less than 300 mm.

b) Clear span-to-width ratio of a column section for columns subjected to existing flexural moments and bent at both ends in two directions shall not exceed 16, but for cantilever columns it shall not exceed 10.

20.5.2.2 Longitudinal reinforcement

20.5.2.2.1 Longitudinal reinforcement ratio in columns shall not be less than one percent and not greater than six percent. Maximum reinforcement limit at the splice shall also be observed. When longitudinal bar is of S400 type steel, reinforcement ratio outside splices is limited to 4.5 percent maximum.

20.5.2.2.2 Center-to-center spacing of longitudinal reinforcement shall not exceed 200 mm.

20.5.2.2.3 Except at beam-column joints, lap splices is permitted to be applied throughout the length of longitudinal reinforcement, provided the lap length of such splices is 1.3 times that of tension splices. When splices are outside the middle half of a column, special transverse reinforcement shall be placed throughout the splice length, in accordance with 20.5.2.3.2.

20.5.2.2.4 Application of lap splice in longitudinal reinforcement is permitted only at the middle half of column. Lap length of such splices shall be applied for tension splices.

20.5.2.2.5 When at any section of the column, longitudinal bars are spliced alternately at the maximum, splice location may be at any segment of the column length except at beam-column joints. As such, lap length in these splices need not exceed that of tension splices. Application of special transverse reinforcement in accordance with requirements of 20.5.2.3.2 is mandatory for the splices located outside middle half of the column.

20.5.2.2.6 Welded or mechanical splices complying with 18.4.1.6 and 18.4.1.7 are permitted if reinforcement splice in each layer is provided alternately and the splice spacing in adjacent bars is less than 600 mm in the direction of member length.

20.5.2.3 Transverse reinforcement

20.5.2.3.1 Unless shear design indicates the need for more reinforcement, special transverse reinforcement as per requirements of 20.5.2.3.2 through 20.5.2.3.6 shall be provided over a length ℓ_o referred to as “critical zone”, measured from segments at both ends of the column. The length ℓ_o measured from the face of lateral members joint shall not be less than the following values:

a) One-sixth the clear height of column.

b) Greater dimension of the rectangular-shaped section, or diameter of circular-shaped section of the column.
c) 450 mm.

20.5.2.3.2 The amount of transverse reinforcement needed in the critical zone is determined based on the following criteria:
a) For circular section columns, the volumetric ratio of spiral or circular hoop reinforcement ρ_s shall not be less than the following two values:

$$\rho_s = 0.12 \frac{f_c}{f_{yh}} \qquad (20\text{-}2)$$

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f_c}{f_{yh}} \qquad (20\text{-}3)$$

b) For rectangular section columns, the total cross-sectional area of special hoop reinforcement in each extension A_{sh} shall not be less than the following two values:

$$A_{sh} = 0.3(s.h_c \frac{f_c}{f_{yh}})(\frac{A_g}{A_{ch}} - 1) \qquad (20\text{-}4)$$

$$A_{sh} = 0.09 s.h_c \frac{f_c}{f_{yh}} \qquad (20\text{-}5)$$

20.5.2.3.3 In columns where design strength of column core by itself satisfies the requirements of loading combinations including earthquake effect, Eqs. (20-3) and (20-4) need not be satisfied.

20.5.2.3.4 Transverse reinforcement diameter in critical zone shall not be less than 8 mm, and the spacing of such reinforcement curtains shall not exceed the following values:

a) One-quarter of the smallest cross-sectional dimension of the column
b) Eight times the smallest diameter of longitudinal bar
c) 125 mm

20.5.2.3.5 Transverse reinforcement for critical zone may be provided by single integrated
hoops or by overlapping multi-piece hoops. Crossties with a 90deg hook on one end and of the same bar size and spacing as the hoops shall also be permitted.

20.5.2.3.6 Crossties or legs of overlapping hoops at any section shall not be spaced more than 350 mm on center in the direction perpendicular to the longitudinal axis of the column.

20.5.2.3.7 In columns where plastic hinge due to inelastic lateral displacement of the frame is likely to occur in sections other than end sections, a length ℓ_o on each side of the section referred to as critical zone shall be reinforced with special transverse bars.

20.5.2.3.8 Columns that support reactions from stiff members such as reinforced concrete walls shall be provided with special transverse reinforcement over their full height. In addition, transverse bars shall extend for at least the development length along part of the longitudinal reinforcement of the column that extends through the wall. The criteria for special transverse reinforcement extension into the wall shall also apply to columns that terminate on a wall.

20.5.2.3.9 In columns where portion of their height is engaged with a concrete wall, special reinforcement shall be placed in the non-engaged free part of the column.

20.5.2.3.10 At the column-footing, connection longitudinal reinforcement extended into the footing shall be strengthened with transverse reinforcement for a length at least equal to 300 mm.

20.5.2.3.11 In portions of the column length not treated with special transverse reinforcement, transverse reinforcement as spiral or special hoop with a diameter at least 8 mm shall be used. Spacing of such reinforcement layers shall be based on shear design needs, but the spacing shall not exceed one-half the smallest rectangular-shaped cross-sectional dimension of the column, one-half the dimension of circular-shaped section of the column, six times the diameter of longitudinal bar, and/or 200 mm.

20.5.2.4 Minimum flexural strength of columns

20.5.2.4.1 In all beam-column joints, except for cases stated in 20.5.2.4.2 and 20.5.2.4.3, column flexural moments for strength shall satisfy the following equation:

$$\sum M_e \geq 1.2 \sum M_g \qquad (20\text{-}6)$$

$\sum M_e$ = Sum of flexural moments corresponding to flexural moments for strength of the columns at top and at bottom of the joint, calculated at the center of joint. Column flexural moments for strength shall be calculated for the most adverse axial loading case on the columns, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

$\sum M_g$ = Sum of flexural moments corresponding to flexural moments for strength of beams on both sides of the joint, calculated at the center of joint.

Flexural moments in the Eq. (20-6) shall be summed up such that the column moments oppose the beam moments. Eq. (20-6) shall be satisfied for beam flexural moments acting on both directions in the vertical plane of the frame considered.

20.5.2.4–2 When the number of columns in a frame of an story is greater than four, one column out of each four is permitted not to satisfy the Eq. (20-6).

20.5.2.4–3 Columns in one and two story frames and also columns of the last story in multi-story frames may be permitted not to satisfy Eq. (20-6), but they must satisfy requirements of 20.5.2.4.5. Such columns shall not be subject to requirements of 20.5.2.4.5.

20.5.2.4–4 If a column does not satisfy the Eq. (20-6), it shall be provided with transverse reinforcement as specified in 20.5.2.3.2 through 20.5.2.3.6 over their full height.

20.5.2.4–5 If a column does not comply with requirements of 20.5.2.4.1, any positive contribution to the lateral strength and stiffness of the structure against lateral loads such as earthquake shall be ignored. The column shall however satisfy requirements of 20.5.6.

20.5.3 Structural walls, diaphragms and trusses

20.5.3.1 Geometric constraints

20.5.3.1.1 The following geometric constraints shall be applied to structural walls:

a) Thickness of the wall shall not be less than 150 mm.

b) For the walls where boundary elements are used in accordance with 20.5.3.3, width of the boundary element shall not be less than 300 mm.

20.5.3.1.2 Large openings shall be prohibited as much as possible within structural walls. In cases where openings are inevitable, their geometric position shall be considered such that the wall can act as coupled walls. Otherwise, the effect of opening on the wall function shall be investigated with the help of careful analysis or proper tests.

20.5.3.1.3 For diaphragms on which there exists large openings, shape and location of opening shall not have determinate effect on lateral stiffness of the diaphragm. Diaphragms behavior shall however comply with analysis assumptions in connection with their level of stiffness.

20.5.3.1.4 In designing walls with T and U-shaped sections, effective flange width, measured from each face of the web and used in calculations shall not be taken greater than the following values:

a) one-half the distance to an adjacent wall web.

b) 10 percent of the total wall height.

20.5.3.1.5 Cast in place reinforced concrete diaphragms, or concrete topping slabs over steel beams, or precast reinforced concrete members acting as a combined unit, serving as diaphragms used to transmit and distribute earthquake forces shall not be less than 50 mm thick.

20.5.3.1.6 Reinforced concrete slabs cast in place on precast floors shall be permitted to be used as structural diaphragm provided the topping slab connections are proportioned and detailed to provide for a complete transfer of forces to chords, ties, collectors, and lateral-force-resisting systems. The surfaces of the precast concrete members at the junction with the in-place reinforced concrete slab shall be clean, free of laitance, and intentionally roughened.

20.5.3.2 Vertical and horizontal reinforcement

20.5.3.2.1 Reinforcement ratio for structural walls shall not be less than 0.25 percent in none of the two vertical and horizontal directions, unless the ultimate shear force at wall section is less than 0.5 $A_{cv}v_c$. Minimum reinforcement required for the wall shall be observed in accordance with requirements of 16.4 from chapter sixteen.

20.5.3.2.2 Vertical reinforcement ratio in no segment of the wall height shall exceed 4 percent.

20.5.3.2.3 Center-to-center spacing of reinforcement shall not exceed 350 mm in none of the two vertical and horizontal directions. The spacing of vertical reinforcement in boundary elements shall not be taken greater than 200 mm.

20.5.3.2.4 At least two curtains of reinforcement shall be used in a wall if the ultimate shear force at its section is greater than $A_{cv}v_c$.

20.5.3.2.5 Structural truss elements, struts, ties, and force-collector elements with compressive stresses exceeding $0.2f_c$ shall have special transverse reinforcement in accordance with 20.5.2.3.2 through 20.5.2.3.6, over full length of the element. The special transverse reinforcement may be discontinued at segments of the member length where the calculated compressive stress of concrete is less than $0.25\phi_c f_c$. Compressive stress of the member is calculated for the ultimate forces, assuming linear distribution of stress at the section and based on specifications of the uncracked section.

20.5.3.2.6 All continuous reinforcement in structural walls, diaphragms, trusses, struts, ties, and force-collector elements shall be anchored or spliced in accordance with the provisions for tension reinforcement as specified in 20.5.4.3.

20.5.3.3 Boundary elements in structural walls and in diaphragms

20.5.3.3.1 At edges and around openings of structural walls and diaphragms where compressive stress of concrete at the extreme compression fiber of the section, corresponding to ultimate forces including earthquake effects exceeds $0.2f_c$, boundary elements shall be provided in accordance with 20.5.3.3.2 through 20.5.3.3.4 unless special transverse reinforcement is provided over full length of the wall or diaphragm. Boundary elements may be discontinued at segments where concrete compressive stress is less than $0.15\ f_c$. Compressive stress of concrete is calculated assuming linear distribution of stress at the wall section and based on specifications of the uncracked section.

20.5.3.3.2 Boundary elements of structural wall in ultimate limit state of strength shall be designed for sum of vertical loads exerted on the wall, including loads of the elements in connection with wall, weight of wall and the axial force due to overturning moment of earthquake lateral forces.

20.5.3.3.3 Boundary elements of structural diaphragms in ultimate limit state of strength shall be designed for sum of axial forces acting in the plane of the diaphragm and the axial force obtained from dividing the effective flexural moment at the diaphragm section by the distance between the boundary elements of the diaphragm at that section.

20.5.3.3.4 Boundary elements shall be treated with special transverse reinforcement over their full length, in accordance with requirements of 20.5.2.3.2 through 20.5.2.3.6.

20.5.3.3.5 In walls with boundary elements, horizontal reinforcement in the wall web shall be anchored to develop tensile stress in the range of specified yield strength within the confined core of the boundary element.

20.5.3.3.6 Horizontal reinforcement terminating at the edges of structural walls without boundary element shall have a standard hook engaging the edge reinforcement. Otherwise the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement. When ultimate shear force at the wall section is less than $0.5A_{cv}v_c$, requirement of the current clause need not be observed.

20.5.3.4 Coupling beams in coupler walls

20.5.3.4.1 Coupling beams in coupler walls where ultimate shear force is greater than $2A_{cv}v_c$ and where clear span-to-depth ratio at the section is less than 3 shall be reinforced in accordance with requirements of 20.5.3.4.2 and 20.5.3.4.3. Otherwise reinforcement in such beams shall be done according to requirements of flexural members. Width of the beams shall not be taken less than 200 mm.

20.5.3.4.2 Shear strength of coupling beams shall be totally provided by intersected diagonal reinforcement placed symmetrically along the entire length of the beam and shall extend into the walls on both sides of the beams for a length equal to 1.5 times the development length of reinforcement. Cross-sectional area of diagonal reinforcement in each of the intersecting legs is calculated by the following equation:

$$A_{vd} = \frac{V_u}{2f_y \sin\alpha} \qquad (20\text{-}7)$$

In this equation α is the angle between diagonal reinforcement and the longitudinal axis of the beam.

20.5.3.4.3 Diagonal reinforcement shall be enclosed in spiral or hoop shape transverse reinforcement with a diameter at least 8 mm. Transverse reinforcement spacing shall at most be equal to the smallest of the following three values:

a) Eight times the diameter of the smallest longitudinal bar.
b) 24 times the diameter of the hoop or spiral bars.
c) 125 mm.

20.5.3.4.4 Flexural strength provided by diagonal reinforcement may be incorporated in calculation of flexural capacity of the coupling beams.

20.5.3.5 Construction joints

20.5.3.5.1 All construction joints in structural walls and diaphragms shall satisfy the requirements of 9.8. Roughness of the contact surfaces shall conform to 12.14.3.5. Shear design requirements of construction joints shall be in accordance with 12.17.3.3.

20.5.4 Beam-column joints in frames

20.5.4.1 General design principles

20.5.4.1.1 Shear design for beam-column joints in frames shall be based on the Eq. (12-1). V_u and V_r in this equation shall conform to the requirements of 20.5.4.1.2 and 20.5.4.1.3.

20.5.4.1.2 Ultimate shear force effective on the joint, V_u, shall be calculated based on maximum tensile force that may develop in tension reinforcement of the beams on both sides of the joint and also based on the shear that may develop in the columns at the top and at the bottom of the joint. In determining these values it is assumed that plastic hinges with flexural capacities, positive or negative, equal to probable flexural moment for strength, M_{pr}, at sections on the face of joint are formed in the beams on both sides of the joint. Directions of these moments shall be chosen so that maximum shear is induced in the joint.

20.5.4.1.3 The ultimate shear strength of the joint, V_r, may be taken equal to the following values at the maximum, on the condition that requirements of 20.5.4.2 are satisfied:

a) For joints confined on all four faces $12A_Jv_c$

b) For joints confined on three faces or on two opposite faces $9A_Jv_c$

c) For other joints $7.5A_Jv_c$

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member.

20.5.4.2 Reinforcement placement

20.5.4.2.1 In all joints except those specified in 20.5.4.2.2, special transverse reinforcement shall be used in accordance with requirements of 20.5.2.3.2 through 20.5.2.3.6.

20.5.4.2.2 Within the depth of the shallowest framing member (beam), special transverse reinforcement equal to one-half the amount required by 20.5.4.2.1 shall be provided where members frame into all four sides of the joint and where member width is not less than three-fourths the column width. Transverse reinforcement spacing at these joints may be permitted to be increased to 150 mm.

20.5.4.2.3 Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to 20.5.4.3 and in compression according to the requirements of chapter 18.

20.5.4.2.4 Beams in which longitudinal reinforcement does not pass through the confined core of the joining column shall be treated with special transverse reinforcement along the length of longitudinal reinforcement outside the column core, if such confinement is not provided by another beam framing into the joint.

20.5.4.2.5 The diameter of longitudinal beam reinforcement extending through a beam-column joint shall not exceed 5 percent of the joint dimension in the extension of longitudinal reinforcement.

20.5.4.3 Development length of bars in tension

20.5.4.3.1 Development length of reinforcement bars with a standard 90 deg hook, ℓ_{dh}, shall be based on the Eq. (18-2) incorporating equivalent bond strength of concrete equal to $2f_{bd}$, Eq. (18-2). Development length of hook shall not be less than: 8 times the reinforcement diameter and 150 mm.

20.5.4.3.2 Hooks shall be anchored in the confined column core, or in boundary elements of walls.

20.5.4.3.3 Development length of straight bars, ℓ_d, shall not be less than 2.5 times the development length of hooked bars for bottom reinforcement, nor less than 3.5 times the development length of hooked bars for top reinforcement, according to definition in 18.2.2.1(a).

20.5.4.3.4 Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary element. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

20.5.5 Principles of shear design

20.5.5.1 Frame members subjected to flexure and combination of flexure and compression

20.5.5.1.1 For frame members subjected to flexure and to the combination of flexure and compression, shear control in ultimate limit state of strength shall be based on the Eq. (12-1). V_u and V_r shall be calculated in accordance with requirements of 20.5.5.1.2 through 20.5.5.1.4.

20.5.5.1.2 Shear forces generated in the flexural member shall be calculated by considering static equilibrium of gravity loads and flexural moments in its end sections, assuming that plastic hinges are formed in those sections. Flexural capacity of plastic hinges, positive or negative, shall be taken equal to the probable flexural moment for strength M_{pr} at the section. Directions of these moments shall be chosen so that maximum shear is induced in the member considered.

20.5.5.1.3 The ultimate shear force, V_u, in members subjected to compression and flexure shall be taken equal to the smallest of the following two values, but the force shall in no conditions be less than the shear force obtained from analysis of the structure under ultimate gravity loads and earthquake lateral forces:

a) Shear force generated in the member subjected to statical forces exerted on it includes gravity loads, if any, and flexural moments in its end sections, assuming plastic hinge is formed in those sections. Flexural capacity of plastic hinges, positive or negative, shall be taken equal to the probable flexural moment for strength M_{pr} at the section. In determining these moments, the most adverse ultimate axial force in the member, which develops maximum flexural moment shall be incorporated. Directions of these moments shall be chosen so that maximum shear is induced in the member considered.

b) Shear force induced in the member, assuming plastic hinges with specifications stated in 20.5.5.1.2 are formed in the beams connected to both ends of the member at the sections adjacent to the joints. Directions of these moments are chosen so that maximum shear is induced in the member considered.

20.5.5.1.4 Ultimate shear strength of prismatic members, V_r, shall be calculated based on the Eq. (12-2). For frame members in which compressive axial force is less than $0.75\phi_c f_c A_g$, and earthquake induced shear force conforming to 20.5.5.1.3 for critical regions of beams and to 20.5.2.3.1 for ℓ_o regions of the columns exceeds one-half the design shear force V_u, the shear strength of concrete V_c is taken equal to zero in these regions. The earthquake induced shear force is defined as the shear force induced in the member due to difference in flexural moments of the plastic hinges formed at the ends of the member considered, in accordance with requirements of 20.5.5.1.2.

20.5.5.1.5 In certain segments of the member, as described in 20.5.1.3 and 20.5.2.3 and 20.5.4.2, stirrups used for shear resistance shall be of hoop type.

20.5.5.2 Structural walls and diaphragms

20.5.5.2.1 For structural walls and diaphragms, ultimate limit state of strength in shear shall be based on the following equation:

$$V_u \le \phi_n V_r \tag{20-8}$$

V_u is the ultimate shear force in the section considered, which is obtained from analysis of the structure subjected to gravity and earthquake induced ultimate loads, and V_r is the ultimate shear strength at the section considered, which is calculated in accordance with 20.5.5.2.2. ϕ_n is the strength modification factor taken equal to 0.7 for these members.

20.5.5.2.2 Ultimate shear strength of the section, V_r, is calculated by the following equation:

$$V_r = A_{cv}(\alpha_c v_c + \phi_s \rho_n f_y) \tag{20-9}$$

Where α_c is a coefficient taken as follows:

a) For walls and diaphragms where the ratio of $\frac{h_w}{\ell_w}$ is greater than or equal to 2 $\alpha_c = 1$.

b) For walls and diaphragms where the ratio of $\frac{h_w}{\ell_w}$ is greater than or equal to 1.5 $\alpha_c = 1.5$.

c) For walls and diaphragms where the ratio of $\frac{h_w}{\ell_w}$ is between 1.5 and 2, the coefficient α_c is determined from linear interpolation of the above values.

20.5.5.2.3 In determining ultimate shear strength of the section in segments of wall or a diaphragm, the value of the coefficient α_c shall be calculated for the largest $\frac{h_w}{\ell_w}$ in the entire wall or diaphragm and in the segment considered.

20.5.5.2.4 Walls and diaphragms shall have distributed shear reinforcement to provide shear resistance in two orthogonal directions in the plane of the wall or the diaphragm. When the ratio $\frac{h_w}{\ell_w}$ is less than 2, vertical reinforcement ratio, ρ_v , shall not be less than horizontal reinforcement ratio ρ_n .

20.5.5.2.5 Ultimate shear strength of the section, V_r, for walls composed of a number of wall piers commonly bearing a unique lateral force shall not be taken greater than $4A_{cv}v_c$. In such walls, ultimate shear strength of the section for any one of the individual wall piers shall not be assumed to exceed $5A_{cp}v_c$, where A_{cp} is the cross-sectional area of each pier and A_{cv} is the total cross-sectional area of the piers.

20-5.5.2.6 Ultimate shear strength of horizontal wall segments corresponding to coupling beams in coupler walls shall not be assumed to exceed $5A_{cp}v_c$, where A_{cp} is the cross-sectional area of horizontal wall segment.

20.5.6 Frame members not designed to resist earthquake

20.5.6.1 Frame members assumed not to contribute to lateral resistance against earthquake forces shall be designed according to requirements of 20.5.6.1.1 and 20.5.6.1.2 depending on the magnitude of the flexural moment in those members when subjected to lateral displacement equal to twice that induced if subjected to ultimate earthquake load in the structure.

20.5.6.1.1 If the flexural moment induced in the member exceeds flexural moments of strength for that member M_r, longitudinal reinforcement requirements of 20.5.1.2.1 for flexural members, and transverse reinforcement requirements of 20.5.2.3 for members subjected to combination of compression and flexure shall be applied. In addition, all these members shall be designed for shear in accordance with requirements of 20.5.5.

20.5.6.1.2 If the flexural moment induced in the member is less than the flexural moment of strength for the member M_r, longitudinal reinforcement requirements of 20.5.1.2.1 for flexural members shall be applied.

20.5.6.2 All members subjected to combination of compression and flexure, where transverse reinforcement requirements of 20.5.2.3 are not satisfied shall be reinforced in accordance with requirements of 20.5.6.2.1 through 20.5.6.2.3.

20.5.6.2.1 Tie bars have hooks with an angle of not less than 135deg and a straight length at least six times the diameter of the stirrup or 60 mm. Application of crossties as defined in this chapter is permitted.

20.5.6.2.2 Maximum spacing between transverse reinforcement curtains, over a length ℓ_o, measured from both ends of a member as defined in 20.5.2.3.1 shall not exceed the following values:

a) Eight times the diameter of the smallest longitudinal bar
b) 24 times the diameter of tie bars
c) one-half the smallest cross-sectional dimension of the frame member

The first tie bar shall not be spaced more than one-half the above values from the face of the member-beam joint.

20.5.6.2.3 In a segment of the member length not including ℓ_o, transverse reinforcement requirements are similar to those of regular columns.

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