Amendments as on July 31, 2017

Following amendments (highlighted in yellow) are made in Table 3.3, Clauses G.2.3, G.2.4, G.2.5, G.3.2 (fully replaced), G.3.3 (newly added) and G.3.4 (shifted from existing G.3.3) in AERB Safety Standard on Design of concrete structures important to safety of Nuclear Facilities [AERB/SS/CSE-1(2001)]

TABLE 3.3: EXPRESSIONS OF *R* (f_k) FOR LIMIT STATE OF STRENGTH

Material	Limit States	$R(f_k)$
Concrete	Direct compression	$0.6 f_{ck}$
	Flexure compression	0.67
	Direct tension	0.35
	Flexure tension	$0.55(f_{ck})^{½}$
	Shear	Ref. cl. 3.6.2
	Bond(1):Deformed bars in tension	$0.6(f_{ck})^{½}$
	:Plain bars in tension	$0.4(f_{ck})^{½}$
Steel	All limit states	f_y

Note: (1) For bars in compression, these values should be increased by 25%.

APPENDIX-G

TEST FOR LIQUID-RETAINING STRUCTURES

G.2 Test Procedure

G.2.3 Test for ingress of ground water will be conducted first by allowing water table to rise slowly to a stable level by stopping dewatering system. All leakage spots and wet surfaces on the inner face of the wall and raft surfaces will be marked accurately. Water table outside the pool will be lowered by starting dewatering system and corrective treatment such as various types of injection grouting, surface coatings, repairs to the concrete etc. carried out. After satisfactory completion of the corrective measures, water level outside the pool will be allowed to rise again. Corrective treatment can be undertaken in presence of water, if performance of repair material in such condition is established. For grouts made of cement or similar material which cannot perform in presence of water, corrective treatment should be taken up only after emptying.

G.2.4 If any wet surfaces persist the procedure will be repeated until the acceptance criteria is satisfied.

Test for Egress of Stored Water

G.2.5 The test will be conducted by filling the storage pool with water. The filling will be in stages to properly identify the leakage spots and leakage paths if any. All wet patches and leak spots on external surfaces will be properly demarcated. After emptying the pool, corrective treatments such as various types of injection grouting, surface coating, repairs to concrete etc. shall be completed. Corrective treatment can be undertaken in presence of water, if performance of repair material in such conditions is established. For grouts made of cement or similar material which cannot perform in presence of water, corrective treatment should be taken up only after emptying. The pool will be filled again and external surfaces observed. In case wet patches appear, the above procedure will be repeated till the acceptance criteria is satisfied.

G.3 Acceptance Criteria

G.3.2 For water retaining structures following tank-in-tank concepts, the procedure of filling the tank and repairing shall be repeated for leak tightness test of stored water until total area of wet patches is less than 0.1% of surface area of each wall. Even in case where acceptance criteria is met, large single patch or multiple patches in a localized area should be further repaired.

G.3.3 For other water retaining structures 'The procedure of filling the tank and repairing shall be repeated both for test for ingress of ground water (where required) and leak tightness test for stored water until no wet patches appear on the surface opposite to liquid facing'.

G.3.4 A drop in water level at the rate of 6 mm or 12 mm for 24 hours may be allowed as evaporation losses for covered or open pool condition respectively as the case may be.

AERB SAFETY STANDARD NO. AERB/SS/CSE-1

DESIGN OF CONCRETE STRUCTURES IMPORTANT TO SAFETY OF NUCLEAR FACILITIES

Approved by the Board....October 5, 2001
(Amended in July 2017)

This document is subject to review, after a period of one year from the date of issue, based on the feedback received.

Atomic Energy Regulatory Board
Mumbai 400 094

Price:

Orders for this Standard should be addressed to:

Administrative Officer
Atomic Energy Regulatory Board
Niyamak Bhavan
Anushaktinagar
Mumbai - 400 094.

FOREWORD

Safety of public, occupational workers and the protection of environment should be assured while activities for economic and social progress are pursued. These activities include the establishment and utilisation of nuclear facilities and use of radioactive sources and have to be carried out in accordance with relevant provisions in the Atomic Energy Act, 1962.

Assuring high safety standards has been of prime importance since inception of the nuclear power programme in the country. Recognising this aspect, the Government of India constituted the Atomic Energy Regulatory Board (AERB) in November 1983, vide statutory order No. 4772 notified in the Gazette of India dated 13.12.1983. The Board has been entrusted with the responsibility of laying down safety standards and framing rules and regulations in respect of regulatory and safety functions envisaged under the Atomic Energy Act of 1962. Under its programme of developing safety codes and guides, AERB had issued four codes of practice in the area of nuclear safety covering the following topics:

Safety in Nuclear Power Plant Siting

Safety in Nuclear Power Plant Design

Safety in Nuclear Power Plant Operation

Quality Assurance for Safety in Nuclear Power Plants

Civil engineering structures in nuclear installations form an important feature having implications to safety performance of these installations. The objective and minimum requirements for the design of civil engineering buildings/structures to be fulfilled to provide adequate assurance for safety of nuclear installations in India (such as pressurised heavy water reactor and related systems) are specified in the Safety Standard for Civil Engineering Structures Important to Safety of Nuclear Facilities (AERB/SS/CSE). This standard is written by AERB to specify guidelines for implementation of the above civil engineering safety standard in the design of concrete structures important to safety.

This standard may be revised as and when necessary in the light of experience as well as developments in the field. The appendices included in the document are an integral part of the document, whereas the annexure, footnotes, and bibliography are to provide information that might be helpful to the user.

Emphasis in the codes, standards, guides and manuals is on protection of site personnel and public from undue radiological hazard. However, for aspects not covered in these documents, applicable and acceptable national and international codes and standards shall be followed. In particular, industrial safety shall be assured through good engineering practices and by complying with the Factories Act, 1948 as amended in 1987 and Atomic Energy (Factories) Rules, 1996.

This safety standard on civil and structural engineering (CSE) has been prepared by the professionals from AERB, BARC, NPC, DCL, TCE and BIS. In its preparation, the relevant national and international documents (mentioned in the "Bibliography" section of this standard) have been extensively used. It has been reviewed by experts and amended by Advisory Committees before issue. AERB wishes to thank all individuals and organisations who have contributed in the preparation, review and amendment of the safety standard. The list of persons who have participated in committee meetings, along with their affiliation, is included for information.

Suhas P. Sukhatme

(Suhas P. Sukhatme)
Chairman, AERB

DEFINITIONS

Acceptable Limits

Limits acceptable to Regulatory Body for accident condition or potential exposure.

Accident conditions

Substantial deviations from operational states which could lead to release of unacceptable quantities of radioactive materials. They are more severe than anticipated operational occurrences and include design basis accidents and severe accidents.

Admixture

Material other than water, aggregate or cement, used as an ingredient of concrete and added to concrete before, during or subsequent to its mixing to modify its properties.

Aggregate

Granular material, such as sand, gravel, crushed stone and iron blast-furnace slag, used with a cementing medium to form a hydraulic-cement concrete or mortar.

Anchorage (Prestressing)

In post-tensioning, a device used to anchor tendon to concrete member, in pretensioning, a device used to anchor tendon during hardening of concrete, a means by which force is transferred to the concrete.

Anchor Head

A nut, washer, plate, stud or bolt head or other steel component used to transmit anchor loads to the concrete by bearing.

Approval

A type of regulatory consent issued by the Regulatory Body to a proposal.

Atomic Energy Regulatory Board (AERB)

A national authority designated by Government of India having the legal authority for issuing regulatory consent for various activities related to the nuclear facility and to perform safety and regulatory functions including enforcement for the protection of the public and operating personnel against radiation.

Attachment

An attachment is an element in contact with or connected to the inside or outside of a component. It may have either a pressure retaining or non-pressure retaining function.

Base Temperature/Stress Free temperature

Temperature at which it is assumed that the material is free of thermal stresses.

Characteristic Strength of Materials

The value of strength of the material, below which not more than 5 percent of the test results are expected to fall.

Coarse Aggregate

The aggregate particles retained on a 4.75 mm IS sieve.

Commissioning

The process during which structures, systems and components of a nuclear and radiation facility, having been constructed, are made functional and verified to be in accordance with design specifications and to have met the performance criteria.

Competent Authority

Any official or authority appointed, approved or recognised by the Government of India for the pupose of the rules promulgated under the Atomic Energy Act 1962.

Construction

The process of manufacturing, testing and assembling the components of a nuclear or radiation facility, the erection of civil works and structures, the installation of components and equipment and the performace of associated tests.

Decommissioning

The process by which a nuclear or radiation facility is finally taken out of operation in a manner that provides adequate protection to the health and safety of the workers, the public and of the environment.

Design

The process and the results of developing the concept, detailed plans, supporting calculations and specifications for a nuclear or radiation facility.

Design Basis Accident (DBA)

Design basis accidents are a set of postulated accidents which are analysed to arrive at conservative limits on pressure, temperature and other parameters which are then used to set specifications that must be met by plant structures, systems and components, and fission product barriers.

Design Inputs

Those criteria, parameters, bases or other requirements upon which detailed final design is based.

Design Outputs

Documents, such as design reports, drawings, specifications, that define technical requirements necessary for manufacture, installation and operation of structures, systems and components.

Disposition

An act to determine how a departure from a specified requirement is to be handled or settled.

Documentation

Recorded or pictorial information describing, defining, specifying, reporting or certifying activities, requirements, procedures or results.

Earthquake

Vibration of earth caused by the passage of seismic waves radiating from the source of elastic energy.

Embedded Part

Any structural member, plates, angle, channel, pipe sleeve or other section anchored to a concrete structure through direct bond or other anchors.

Embedment

Embedment is that portion of the component in contact with the concrete or grout used to transmit applied loads to the concrete structure through direct bond or other anchor. The embedment may be fabricated lugs, bolts, reinforcing bars, shear connectors, expansion anchors, inserts or any combination thereof.

Examination

An element of inspection consisting of investigation of materials, components, supplies or services to determine conformance with those specified requirements which can be determined by such investigation.

Fine Aggregate (Sand)

The portion of aggregate passing a 4.75 mm IS sieve.

Green Concrete

Concrete that may or may not have attained initial set, but has not yet gained appreciable strength.

Inspection

Quality control actions which by means of examination, observation or measurement determine the conformance of materials, parts, components, systems, structures as well as processes and procedures with predetermined quality requirements.

Item

A general term covering structures, systems, components, parts or materials.

Items Important to Safety

The items which comprise:

(1) those structures, systems, equipment and components whose malfunction or failure could lead to undue radiological consequences at plant or off-site;

(2) those structures, systems and components which prevent Anticipated Operational Occurrences from leading to Accident Conditions; and

(3) those features which are provided to mitigate the consequences of malfunction or failure of structures, systems or components.

Main Structural Members

The structural members, which are primarily responsible to withstand, carry and distribute the applied load.

Maintenance

Organised activities covering all preventive and remedial measures, both administrative and technical, necessary to ensure that all structures, systems and components are capable of performing as intended for safe operation of plant.

Non-conformance

A deficiency in characteristics, documentation or procedure which renders the quality of an item unacceptable or indeterminate.

Normal Operation

Operation of a plant or equipment within specified operational limits and conditions. In case of Nuclear Power Plant this includes start-up, power operation, shutting down, shut-down state, maintenance, testing and refuelling.

Nuclear Power Plant (NPP)

A nuclear reactor or a group of reactors together with all associated structures, systems and components necessary for safe generation of electricity.

Nuclear Facility

All nuclear fuel cycle and associated installations encompassing the activities covering from the front end to the back end of nuclear fuel cycle processes and also the associated industrial facilities such as heavy water plants, beryllium extraction plants, zirconium plant, etc..

Objective Evidence

Term used in context of Quality Assurance, qualitative or quantitative information, record or statement of fact, pertaining to the quality of an item or service, which is based on observation, measurement or test and which can be verified.

Operating Basis Earthquake (OBE)

The “Operating Basis Earthquake” (OBE) is that earthquake which, considering the regional and local geology and seismology and specific characteristics of local sub-surface material, could reasonably be expected to affect the plant site during the operating life of the plant; it is that earthquake which produces the vibratory ground motion for which the features of Nuclear Power Plant (NPP) necessary for continued safe operation are designed to remain functional.

Operation

All activities following commissioning and before decommissioning performed to achieve, in a safe manner, the purpose for which a nuclear or radiation facility was constructed, including maintenance.

Physical Separation

A means of ensuring independence of an equipment through separation by geometry (distance, orientation, etc.), appropriate barriers or a combination of both.

Postulated Initiating Events (PIEs)

Identified event that lead to anticipated operational occurrence and accident conditions, and their consequential failure effects.

Primary Stress

Primary stress is any normal stress or shear stress developed by an imposed loading which is necessary to satisfy the laws of equilibrium of external and internal forces and moments. The basic characteristic of a primary stress is that it is not self limiting.

Prescribed Limits

Limits established or accepted by the Regulatory Body.

Qualified Person

A person who having complied with specific requirements and met certain conditions, has been officially designated to discharge specific duties and responsibilities. (e.g., Reactor Physicist, Station Chemist and Maintenance Personal of Nuclear Power Plants are qualified persons).

Quality

The totality of features and characteristics of a product or service that bear on its ability to satisfy a defined requirement.

Quality Assurance

Planned and systematic actions necessary to provide adequate confidence that an item or facility will perform satisfactorily in service as per design specifications.

Quality Control

Quality assurance actions, which provide a means to control and measure the characteristics of an item, process or facility in accordance with established requirements.

Records

Documents which furnish objective evidence of the quality of items or activities affecting quality. It also includes logging of events and other measurements.

Reliability

The probability that a device, system or facility will perform its intended function satisfactorily under stated operating conditions.

Repair

The process of restoring a non-conforming item to a condition such that the capability of this item to function reliably and safely is unimpaired, even though that item still may not conform to the prior specification.

Responsible Organisation

The organisation having overall responsibility for siting, design, construction, commissioning, operation and decommissioning of a facility.

Rework

The process by which a non-conforming item is made to conform to a prior specified requirement by completion, remachining, reassembling or other corrective means.

Safe Shutdown Earthquake (SSE)

The “Safe Shutdown Earthquake” is that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local sub-surface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems and components are designed to remain functional. These structures, systems, and components are those which are necessary to assure:

(a) the integrity of the reactor coolant pressure boundary; or

(b) the capability to shut down the reactor and maintain it in a safe shutdown condition; or

(c) the capability to prevent the accident or to mitigate the consequences of accidents which could result in potential off-site exposure higher than the limits specified by the Regulatory Body; or

(d) the capacity to remove residual heat.

Safety

Protection of all persons from undue radiological hazards.

Safety Limits

Limits upon process variables within which the operation of the facility has been shown to be safe.

Safety System (Safety Critical System)

Systems important to safety, provided to assure, under anticipated operational occurrences and accident conditions, the safe shutdown of the reactor (shut down system) and the heat removal from the core (emergency core cooling system), and containment of any radioactivity (containment isolation system).

Secondary Stress

Secondary stress is a normal stress or shear stress developed by the constraint of adjacent material or by self-constraint of the structure. The basic characteristic of a secondary stress is that it is self-limiting.

Services

The performance by a supplier of activities such as design, fabrication, installation, inspection, non-destructive examination, repair and/or maintenance.

Site

The area containing the facility defined by a boundary and under effective control of facility management.

Site Personnel

All persons working on the site, either permanently or temporarily.

Siting

The process of selecting a suitable site for a facility including appropriate assessment and definition of the related design bases.

Specification

A written statement of requirements to be satisfied by a product, a service, a material or a process, indicating the procedure by means of which it may be determined whether the specified requirements are satisfied.

Structure

The assembly of elements which supports/houses the plants, equipment and systems.

Supplier Evaluation

An appraisal to determine whether or not a management system is capable of producing a product or service of a stated quality and generating evidence that supports decisions on acceptability.

Surveillance

All planned activities viz monitoring, verifying, checking including in-service inspection, functional testing, calibration and performance testing performed to ensure compliance with specifications established in a facility.

Testing

The determination or verification of the capability of an item to meet specified requirements by subjecting the item to a set of physical, chemical, environmental or operational conditions.

Verification

The act of reviewing, inspecting, testing, checking, auditing, or otherwise determining and documenting whether items, processes, services or documents conform to specified requirements.

SYMBOLS

Unless specified otherwise, the following symbols apply to this standard and may not necessarily conform to the symbols adopted elsewhere for national and international use. Unless specified otherwise, SI units (millimeter for linear dimension and newton for force) are adopted.

A_{cv}	Area of concrete section resisting shear.
A_g	Gross cross-sectional area of concrete members.
A_p	Area of prestressing reinforcement.
A_{sc}	Area of compressive reinforcement.
A_{st}	Area of tensile reinforcement.
$A_{s\min}$	Minimum reinforcement for massive concrete element.
A_t	Area of structural steel shape, pipe or tubing in a composite section.
A_{tc}	Effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that of reinforcement, divided by the number of bars. When flexural reinforcement consists of different bar/wire sizes, the number of bars/wires shall be computed as the total area of reinforcement divided by the area of the largest bar used.
a	Air content in the concrete (in %).
b	Width of compressive flange of a rectangular beam.
b_0	Perimeter of critical section for slab and footing.
b_v	Width of cross-section at contact surface being investigated for horizontal shear.
b_w	Width of web of flanged beam.
C	Creep coefficient.

c	Clear cover
CC	Cement content, (kg/m^3)
DL	Dead loads and others.
d	Effective depth of flexural member, or distance from extreme compression fibre to centroid of tension, reinforcement.
d_c	Distance from extreme compression fibre to centroid of compression reinforcement.
E_c	Short-term static modulus of elasticity of concrete.
E_0	Loads generated by the operating basis earthquake.
E_s	Modulus of elasticity of steel.
E_{ss}	Loads generated by the safe shutdown earthquake.
F	Load resulting from the application of prestress.
FF	Loads resulting from Design Basis Flood.
F_{di}	i^{th} design load combination.
F_{ij}	j^{th} characteristic load for i^{th} load combination.
F_h	Hydrostatic load due to internal flooding.
f_k	Characteristic strength of materials (general representation).
f_{ba}	Allowable bending compressive stress in concrete.
f_{ca}	Allowable direct compressive stress in concrete.
f_{bc}	Calculated bending compressive stress in concrete.
f_{cc}	Calculated direct compressive stress in concrete.
f_{ck}	Characteristic compressive strength of 150 mm concrete cube at 28 days.
f_{ci}	Characteristic strength of concrete at i^{th} day after casting.

f_{cp}	Compressive stress at centroidal axis due to prestress.
f_{cr}	Modulus of rupture of concrete.
f_d	Design strength of material.
f_p	Characteristic strength of prestressing steel.
f_s	Stress in reinforcement.
f_y	Characteristic strength of reinforcing steel, corresponding to 0.2% proof stress for high strength deformed bars or yield stress in case of mild steel.
H	Lateral earth pressure.
H_d	Relative humidity (in %).
h	Overall depth or thickness of member.
I	Moment of inertia of section.
I_b	Moment of inertia of beam about centroidal axis of gross section.
I_c	Moment of inertia of gross section of column.
I_{cr}	Moment of inertia of cracked section transformed to concrete.
I_e	Effective moment of inertia for computation of deflection.
I_g	Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement.
I_{se}	Moment of inertia of reinforcement about centroidal axis of member cross-section.
I_t	Moment of inertia of a structural steel shape, pipe or tubing in a composite member cross section.
k	Effective length (slenderness ratio) or height factor.
LL	Live loads and others.
L_c	Vertical distance between supports.

L_x, L_y Length of the shorter span and larger span in two-way slab panels respectively.

l_c Unsupported length of column.

l_d Development length.

MA Load and other effect of aircraft impact.

ME Load and other effect of missiles due to external events other than those related to wind or tornado, explosions in transportation systems, disintegration of turbine and other components.

MI Load effects due to internal missiles.

MT Load and other effect of missiles, wind and overpressure generated from explosions in transportation systems, on land, water or in air.

Mt Loading effect of turbine missile.

M_u Factored moment.

M_{ns} Factored end moment on a compression member due to load that result in no appreciable lateral deflection; calculated by conventional elastic frame analysis.

M_{cr} Cracking moment.

M_s Factored end moments on a compression member which result in appreciable lateral deflection; calculated by conventional frame analysis.

M_1 Value of smaller factored end moment.

M_2 Value of the larger factored end moment.

P_a Maximum differential pressure load generated by a postulated design basis accident.

P_r Axial tensile strength of concrete.

P_t Pressure during the structural integrity and leak rate tests.

P_u Factored axial load normal to the cross section.

P_v	Pressure load resulting from pressure variation either inside or outside the containment
p	Percentage of tension steel.
p_c	Percentage of compression steel.
R_a	Pipe and equipment reactions generated by a postulated accident used as a design basis and including R_0.
R_0	Pipe and equipment reactions during normal operating or shutdown conditions, based on the most critical transient or steady state condition, excluding dead load and earthquake reactions.
T	Time-dependent factor for sustained load.
T_a	Thermal effects and loads generated by a postulated design basis accident and including T_0.
T_0	Thermal effects and loads during normal operating or shutdown conditions.
T_t	Thermal effects during test.
T_u	Factored torsional moment.
V_u	Factored shear force.
V_c	Shear resistance provided by concrete.
V_{ci}	Shear resistance provided by concrete when diagonal cracking results from combined shear and moment.
V_{uh}	Factored horizontal shear strength of composite section.
V_s	Shear strength offered by shear reinforcement.
WC	Loads generated by severe wind.
w_c	Crack width.

w — Value of uniformly distributed load per unit length of beam and per unit area of slab.

W_t — Loading effects due to missile generated by extreme wind.

x — Shorter overall dimension of rectangular part of cross-section.

x_u — Depth of neutral axis from the extreme compressive fibre.

y — Larger overall dimension of rectangular part of cross-section.

Y_j — Jet impingement load on a structure generated by a postulated accident used as a design basis.

Y_m — Missile impact load on a structure, such as pipe whipping generated by or during a postulated accident used as a design basis.

Y_r — Loads on the structure generated by the reaction of broken high-energy pipe during a postulated accident used as a design basis.

y_t — Distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension.

$_x$, $_y$ — Bending moment coefficients in two-way slabs.

Ratio of the long side to short side of the footing, two-way slab.

$_d$ — Ratio of maximum factored dead load moment to maximum factored total load moment, always positive

$_{fij}$ — Partial factor of safety (load factor) for j^{th} characteristic load in i^{th} load combination.

$_m$ — Partial factor of safety appropriate to materials (materials factor).

$_c$ — Partial factor of safety for concrete.

$_s$ — Partial factor of safety for reinforcing steel.

$_b$ — Moment magnification factor for frames, braced against sidesway to reflect effects of member curvature between ends of compression members.

$_{s}$	Moment magnification factor for frames not braced against sidesway to reflect lateral drift resulting from lateral and gravity loads.
$_{cc}$	Creep strain.
$_{c}$	Concrete strain
$_{i}$	Instantaneous strain.
$_{s}$	Steel strain
$_{sh}$	Shrinkage strain.
	Stress.
U	Load combination factor.
$_{c}$	Factor to calculate long time deflection.
	Diameter of reinforcing bars; Factor of safety for structural resistance (resistance factor).
T	Temperature gradient.

CONTENTS

1. INTRODUCTION

1.1 General

1.1.1 The requirements for design and detailing of concrete structures important to safety of Nuclear Power Plants (NPPs) are in some respects different from those of normal conventional structures. The Indian Standard (IS) codes do not cover the special features related to safety demand of NPP structures. Keeping this in view, generally, the design standards for concrete structures important to safety, used in other countries, e.g. USA, Canada, France, etc. were adapted along with IS codes and IAEA documents in the engineering of previous Indian NPPs. One vital question that arose was how to assure the compatibility of the Indian material used in construction with the design specification of codes used by other countries. The second important point was the integration of Indian practice of design and construction with the codal requirements of other countries. In 1990, the AERB therefore decided to prepare its own document for design/detailing and construction of concrete structures important to safety of Indian NPPs.

The AERB therefore constituted a Code Committee for Civil and Structural Engineering (CCCSE) which was assigned to prepare design standards, guides and manuals considering information on design specification of NPP structures acceptable to the regulatory authorities of other countries. In the preparation of this document, codal provisions for the engineering of structures important to safety of NPP used in other countries and acceptable to their regulatory authorities were examined with respect to the need felt in India.

The present document may be viewed as the Indian design standard for concrete structures important to safety of nuclear facilities. To prepare this standard, CCCSE derived assistance from the following codes/standards:

(1) ACI (1985), Code Requirements for Nuclear Safety-Related Concrete Structures (ACI349-85) and Commentary (ACI349-90) and Commentary (ACI349R-85), American Concrete Institute, Redford Station, Detroit, USA.

(2) BIS (1978), Indian Standard Code of Practice for Concrete Structures for Plain and Reinforced Concrete (Third Revision), IS:456-1978, Bureau of Indian Standards, New Delhi.

(3) BIS (1981), Indian Standard Code of Practice for Prestressed Concrete (First Revision), IS:1343-1980, Bureau of Indian Standards, New Delhi.

(4) BSI (1985), British Standard, Structural Use of Concrete, Part-I, Code of Practice for Design and Construction, BS8110, Part-I, British Standard Institution, U.K.

(5) BSI (1987), British Standard Code of Practice for Design of Concrete Structures for Retaining Aqueous Liquids, BS8007:1987, British Standard Institution, U.K.

(6) CSA (1984), National Standard of Canada, Design of Concrete Structures for Buildings, CAN3-A23.3-M84, Canadian Standards Association, Toronto, Ontario, Canada.

(7) EDF-AFCEN (1989), Design and Construction Rules for Civil Works of PWR Nuclear Islands, RCC-G, Volume-1 Design, Electricite De France, Paris, France.

(8) JSCE (1986), Standard Specification for Design and Construction of Concrete Structures, Part-1 (Design), First Edition, SP-1, Japan Soc. of Civil Engineers, Tokyo, Japan.

(9) SANZ (1982), New Zealand Standard Code of Practice for the Design of Concrete Structures, NZS3101, Part-I, Standards Association of New Zealand, Wellington, New Zealand.

(10) SANZ (1982), Commentary on Code of Practice for the Design of Concrete Structures, NZS3101, Part-II, Standards Association of New Zealand, Wellington, New Zealand.

1.1.2 The special functional and safety requirements of reinforced and prestressed concrete structures important to safety of Nuclear Power Plants (NPPs) calls for design criteria over and above conventional structures. The Safety Standard for Civil Engineering Structures important to safety of Nuclear Facilities, (AERB/SS/CSE) describes the philosophy, safety design approach, and design requirements of civil engineering structures important to safety. The present standard aims at stipulating the design requirements of concrete structures important to safety of nuclear facilities in line with the stipulations contained in AERB/SS/CSE. Pressurised concrete reactor vessels and containment structures are excluded from the scope of this standard, unless specified otherwise. The design requirements for concrete containment structures are covered in the standard on Design of Nuclear Power Plant Containment Structures, (AERB/SS/CSE-3).

1.2 Safety Design Basis

1.2.1 The goal of nuclear safety, also important for safe design, is to protect site personnel, public, and environment by establishing and maintaining an effective safeguard against radiological hazards.

1.2.2 The structural system of concrete structures important to safety shall be so designed as to serve the needs of safety functions in two ways. It supports, houses and provides controlled environment for safe operation of plants, systems and equipment such that no fault can occur due to the effects of site/ plant-specific Postulated Initiating Events (PIEs) which might otherwise have caused release of radioactivity. Secondly, given a condition, the radioactivity beyond structural boundary of the building is within prescribed limits under normal operating conditions and within acceptable limits during and following accident conditions.

1.2.3 Design bases of concrete structures important to safety shall specify the necessary capabilities of plant structures to cope with a specified range of operational states to maintain the prescribed limit. The bases shall also specify similar capabilities with respect to accident conditions to maintain an acceptable limit.

1.2.4 Design criteria for a building/structure shall be derived from relevant safety functions of that building/structure (refer Section 2.3 of AERB/SS/CSE).

Design criteria of concrete structures important to safety are divided into the following categories:

(a) radiological protection;

(b) serviceability; and

(c) structural strength.

In addition, leak tightness of the liquid-retaining structures shall be considered.

1.2.5 Concrete structures shall be so designed for design loading conditions and so constructed that they are structurally safe, stable and capable of performing the required safety functions throughout the planned life and until such time as may be required after decommissioning the plant.

1.2.6 All concrete structures important to safety shall be classified depending on the required safety functions to be performed by structures satisfying the provisions of Sections 2.3 and 2.4 of AERB/SS/CSE. The design conditions and respective load combinations are specified on the basis of their classifications. The summary of classification of civil engineering structures and the corresponding design conditions with load combinations are given in Table 1.1.

1.3 Scope

1.3.1 This standard describes the acceptable design stipulations for design class DC3 concrete structures (ref. cl. 2.4.10 of AERB/SS/CSE) which include the internal structure of the reactor building and other buildings and structures important to safety.

1.3.2 This standard covers:

(a) general design requirements;

(b) limit state design method of RCC and prestressed concrete structures;

(c) allowable stress method of design;

(d) special design requirements;

(e) detailing;

(f) design and testing of water-retaining structures;

(g) precast and composite construction;

(h) construction requirements; and

(i) strength assessment of existing structures or its elements.

1.3.3 This standard does not cover the following unless specified otherwise:

(a) pressurised concrete reactor vessel;

(b) reactor containment structures;

(c) design criteria with respect to radiation shielding;

(d) irradiation effects on concrete structures; and

(e) design of structures not important to safety.

1.3.4 The data and relationships given in this document are only for the purpose of guidance.

1.3.5 Safety assessment of other structures outside the plant area or site area such as dams, slopes, etc. whose performance influence the safety of the plant shall be carried out following the stipulations of this standard.

1.3.6 This standard is applicable for nuclear power plant. However, other nuclear installations, the failure of which may cause unacceptable radiological hazard in public domain may also find the provisions of this standard useful.

TABLE 1.1: SUMMARY OF CLASSIFICATIONS, DESIGN CONDITIONS AND LOAD COMBINATIONS FOR CIVIL ENGINEERING STRUCTURES

Design Class	Safety Class	Seismic Category	Quality Requirement	Design[1,7] Conditions	Load[2] Combinations
DC1	1	1	Chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
				Abnormal	LC3, LC4, LC5, LC6
DC2	2	1	Chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
				Abnormal	LC3, LC4, LC5, LC6
DC3	2,3	1	Chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
				Abnormal	LC3, LC4, LC5, LC6[3]
	3[4]	2	Chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
				Abnormal	LC4, LC5
	4[5]	2	Chapter-9 of AERB/SS/CSE	Normal	LC1, LC2
				Abnormal	LC4
DC4	NNS[6]	3			

Note: (1) Ref. cl. 3.2.2 of AERB/SS/CSE or cl. 2.1.3 of this standard,

(2) Ref. cl. 3.5.5 of AERB/SS/CSE,

(3) This load combination is applicable only for internal structures of reactor building,

(4) Structures which do not perform the safety functions associated with supporting the core cooling systems and other systems related to safe shutdown of reactor or prevent/mitigate the consequences of accident which could result in potential off-site exposure comparable to relevent AERB guidelines. This class also includes structures of nuclear facilities with limited radioactive inventory whose functioning shall be maintained in the event of the design basis ground motion,

(5) Structures of nuclear facilities with limited radioactive inventory whose loss of function may be permitted but should be designed against collapse in the event of design basis ground motion. Lower safety level for load combinations LC2 and LC4 (smaller load factors for limit state design or plastic design and higher allowable stresses in allowable stress design method) is allowed for these structures,

(6) Non-nuclear services not important to safety should meet the design requirements as per relevant national standard engineering practices,

(7) Design requirements should be as per the relevant AERB standards/guides (ref. 3.5 of AERB/SS/CSE).

Use of the Standard

1.3.7 This standard shall be used in conjunction with the Safety Standard for Civil Engineering Structures Important to Safety of Nuclear Facilities (AERB/SS/CSE) and other relevant AERB safety codes and guides, Code of Practice for Plain and Reinforced concrete (IS:456), Code of Practice for Prestressed Concrete Structures (IS:1343) and other relevant Indian standards published by Bureau of Indian Standards (BIS). The provision in this standard will have precedence over corresponding provisions contained in the relevant IS codes and standards.

1.4 Quality Assurance

1.4.1 Complete quality assurance programme for design and construction of concrete structures important to safety shall be developed in line with chapter-9 of AERB/SS/CSE and highlighting the documentation as per 1.4.2.

1.4.2 Design Documentation

(a) The analysis and design of the structure shall be well documented and prepared and shall include reasonableness and basis of all assumptions made in the analysis and design methods. The source of input data used in calculations should be adequately referenced so that all data is readily traceable. The design document should include analysis and design assumptions, sketches, input data, output results and other item as may be deemed fit,

(b) The design shall be independently checked and certified by competent persons as per safety importance of component/system,

(c) Model tests, if performed, should be suitably documented,

(d) Structural drawings, details, and specifications for all design class DC3 structures, shall be approved and signed by the authorised engineer and retained by the owner, as permanent record.

1.5 Approval of Special Design and Construction Techniques

1.5.1 For novel or unproven methods of analysis, design and construction or for use of special construction materials (not covered in this standard), prior approval shall be obtained from AERB after ensuring comparable safety.

1.6 Structure of the Document

1.6.1 This standard comprises 8 chapters, 8 appendices and 1 annexure. Each chapter is divided into Sections and further subdivided into paragraphs or clauses.

1.6.2 Chapter-2 : General design requirements

Chapter-3 : Limit state design of non-prestressed reinforced concrete structures

Chapter-4 : Allowable stress design method

Chapter-5 : Special design requirements of structural elements

Chapter-6 : Prestressed concrete structures

Chapter-7 : Liquid-retaining structures

Chapter-8 : Precast and composite construction

1.6.3 Appendix-A: Thermal considerations

Appendix-B: Estimation of creep and shrinkage

Appendix-C: Estimation of crack width

Appendix-D: Effective length of columns

Appendix-E: Detailing of reinforcements

Appendix-F: Requirements for construction and expansion joints, embedded pipes and parts

Appendix-G: Test for liquid-retaining structures

Appendix-H: Strength evaluation of existing structures

1.6.4 Annexure-I: Consideration of impulsive and impactive effects

2. GENERAL DESIGN REQUIREMENTS

2.1 General Requirements

2.1.1 The design class DC3 structures may be of different safety classifications. The same level of safety in terms of load factors and strength factors, or factor of safety is provided for safety classes 2, 3 and 4 concrete structures in this standard for design excepting the provision of clauses 3.4.11 and 4.2.6. However, a variable level of safety in designing these class DC3 concrete structures depending on safety classification is acceptable provided the safety equation of Annexure-I of AERB/SS/CSE is satisfied.

2.1.2 All structural elements should be cast-in-situ except elements like hatch covers and lintels where precast construction is permissible. However, precast construction of other structural elements is acceptable if provision of section 1.5 is complied with.

Design Methods

2.1.3 Only limit state and allowable stress design methods are acceptable unless other methods are acceptable to the regulatory body. The structure shall be designed to withstand safely all loads (due to normal and abnormal condition) liable to act on it through out its life for the following design conditions (ref. cl. 3.2.2 of AERB/SS/CSE):

(a) Normal Design Condition which includes load combinations LC1 and LC2, i.e. normal and severe environmental load combination respectively,

(b) Abnormal Design Conditions which include load combinations LC3, LC4, LC5 and LC6, i.e. extreme environmental, abnormal, abnormal-severe environmental, and abnormal extreme environmental load combinations respectively.

2.1.4 For both limit state and allowable stress design methods, the design shall satisfy the strength and serviceability (such as limitations on stress, deflection, cracking, etc) requirements specified for design conditions given in cl. 2.1.3.

2.1.5 Design requirements for reinforced and prestressed concrete structures shall comply with the following clauses in addition to IS:456, IS:1343 and other relevant IS Codes. The provision of this standard will have precedence over the corresponding provision of IS standard.

Impulsive and Impactive Effects

2.1.6 Information regarding design consideration of impulsive and impactive effects is given in Annexure-I.

Strength Evaluation of Existing Structures

2.1.7 Strength of existing structures should be evaluated using the methodology described in Appendix-H.

2.2 Loading

2.2.1 Structures shall be designed to resist all applicable loads described in Appendix-A of AERB/SS/CSE.

Live Load Pattern

2.2.2 Live load should be considered to be applied as to produce the worst effect on the member.

2.2.3 Reduction in magnitude of live load is not permissible except in calculating inertia masses due to live load for seismic analysis.

2.3 Materials

2.3.1 Use of light weight concrete is not allowed in structural elements of NPP.

2.3.2 All materials specified in design and used in construction of concrete structures important to safety shall comply the requirement of AERB safety guide Material of Construction for Civil Engineering Structures Important to Safety (AERB/SG/CSE-4).

2.3.3 Grade of steel reinforcement shall not be more than Fe 415.

2.3.4 The minimum grades of concrete shall be:

Site	Reinforced concrete structures	Prestressed concrete structures
Inland site	M25	M35
Coastal site	M30	M40

In no case, the grade of concrete for any structure or part of structures shall be leaner than that specified in IS: 456/IS:1343 for different types of exposure conditions.

Creep and Shrinkage

2.3.5 Recommended methodology to determine the creep coefficient and shrinkage strains are given in Appendix-B.

Coefficient of Thermal Expansion

2.3.6 In the absence of actual data, the coefficient of thermal expansion for concrete may be taken as per values specified in IS:456 for ordinary concrete. For heavy concrete the same may be taken as 1.1 x 10^{-5} per ^{o}C.

Modulus of Elasticity

2.3.7 Modulus of elasticity is normally related to compressive strength of concrete. In the absence of test data, the modulus of elasticity for structural concrete may be taken as per the value specified in IS:456.

For heavy concrete made with naturally occuring aggregates, similar values as for normal concrete, as given above, may be used.

Modulus of Rupture

2.3.8 Flexural and split tensile strengths shall be obtained as described in IS:516 and IS:5816 respectively. In the absence of test data, modulus of rupture may be estimated using following expression:

$$f_{cr} = 0.7 \quad f_{ck} \quad (N/mm^2)$$

where f_{ck} is the characteristic compressive strength of concrete in N/mm^2.

Poisson's Ratio

2.3.9 In the absence of test data, Poisson's ratio of structural concrete may be taken as 0.20.

Age Factor

2.3.10 The strength of concrete increases with its age. In the case of retrofitting analysis of existing structures the increase in concrete strength can be considered when structures satisfy the criteria given in AERB/SS/CSE. In the design of new structures for strength, the increase in strength due to age need not be considered. However, in calculating deflection the effect of age is to be considered.

Durability

2.3.11 All provisions specified in IS:456 with respect to durability of concrete structures shall be adhered to.

2.3.12 Durability of concrete in a structure is its resistance to the deteriorating effects of its environment. In particular, the concrete should be resistant to the action of chemicals, alternate wetting and drying, freezing and thawing cycles, etc..

2.3.13 Air entraining agents should be used in freezing and thawing exposures. Resistance to chemical attack can be improved by using good quality concrete with types of cement that can improve chemical resistance and provide a smooth surface finish. Care should also be taken to provide adequate cover for reinforcement and to use fittings and embedded items that do not corrode and cause damage to the concrete. Filling and patching of tie holes is necessary to ensure long-term durability of the concrete.

2.3.14 Improved workability, lower water-cementitious material ratio, corrosion resistance and increased resistance to sulphate attack may be derived from proper use of a good quality mineral admixture in the concrete mix. The use of a sulphate-resistant cement may also be considered when sulphate-resistant concrete is principally required.

2.3.15 Adequate provisions should be made to avoid damage due to floods, rain, snow, and freezing and thawing. In some cases, adequate durability can be obtained only by use of special protective barriers. Structures subject to movement of liquids should be resistant to erosion.

2.4 Methods of Analysis

2.4.1 The general requirements of structural analysis are given in cl. 3.5.1 and 3.5.2 of AERB/SS/CSE. For response analysis corresponding to limit state method of design, Section 3.3 may be referred.

Redistribution of Negative Moments in Reinforced Concrete Beams

2.4.2 (a) Negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement may each be increased or decreased by not more than 10%, except where approximate values for moments are used,

(b) Modified negative moments shall be used for calculating moments at sections within the spans.

Span Length

2.4.3 Span length of members, not built integrally with supports, is to be considered as clear span plus depth of member but need not exceed distance between centres of supports.

2.4.4 In simplified analysis of frames or continuous construction for determination of moments, span length is to be taken as the centre-to-centre distance of supports. However, for beams built integrally with supports, moments at faces of supports may be used for design.

Stiffness

2.4.5 Reasonable and consistent assumptions should be adopted throughout the analysis for computing stiffnesses of members.

2.4.6 Flexural stiffness of members may be based on the moment of inertia of the section determined on the basis of any one of the following assumptions:

(a) Gross Section : the overall cross-section of the member ignoring reinforcement;

(b) Transformed section : the concrete cross-section plus the area of reinforcement transformed on the basis of modular ratio;

(c) Cracked section : the stiffness parameters shall be based on moment curvature diagrams.

The assumption made in calculating stiffness shall be consistently followed.

2.5 Design Considerations Based on Structural Behaviour

Critical Sections for Moment and Shear

2.5.1 For beams built integrally with supports, moments at faces of supports may be considered for design.

2.5.2 For non-prestressed members, sections located less than a distance *d* from face of support may be designed for the same shear force as that computed at a distance *d*.

2.5.3 For prestressed members, sections located less than a distance *h*/2 from face of support may be designed for the same shear force as that computed at a distance *h*/2.

2.5.4 Maximum factored shear force at supports may be computed as stated above only when both the following conditions are satisfied,

(a) support reaction, in direction of applied shear, introduces compression into the end regions of member,

(b) no concentrated load occurs between face of support and location of critical sections defined above.

2.5.5 In the analysis for determining design moments and axial forces, special attention should be given to loading patterns which will yield the critical design condition.

Columns

2.5.6 In frames or continuous construction, consideration should be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

T-Beam and L-Beam

2.5.7 In T-beam construction, the flange and web shall either be built integrally or otherwise effectively bonded together. The flange width of T-beam or L-beam should be calculated as per IS:456 and should satisfy the following:

(a) For the beam to be effective as a T-beam, flange width beyond the web on either side shall not be taken exceeding 8 times the slab thickness,

(b) For the beam to be effective as L-beam, flange width beyond web shall not be taken exceeding 6 times the slab thickness.

Effect of Floor Finish

2.5.8 A floor finish shall not be included as part of structural member unless cast monolithically with the floor slab or designed in accordance with requirements of composite construction.

Stability of Structures

2.5.9 The requirement of AERB safety guide, Geotechnical Aspects for Buildings and Structures Important to Safety of Nuclear Facilities (AERB/SG/CSE-2) shall be satisfied to ensure overall stability of structures.

Fire Resistance of Structures

2.5.10 Adequate fire protection should be provided through the use of fire resistant construction. Any new construction material should be qualified for desirable fire resistance properties before its adoption in design and construction.

2.5.11 Protection of reinforcing steel in reinforced concrete construction is achieved by providing suitable concrete cover to the members as given in Appendix-E. When clear cover is required to be more than 40 mm for fire protection, wire mesh shall be provided.

2.5.12 Monolithic concrete floor finishes may be considered as part of required cover or total thickness from fire resistance considerations.

2.5.13 The minimum requirements of fire protection shall comply with cl. 3.8.2, 3.8.3, 3.8.4 of AERB/SS/CSE and with Code of Practice for Fire Safety of Buildings (general) : Materials and Details of Construction, (IS:1642).

2.6 Detailing

2.6.1 Design and detailing of construction/expansion joints should be in accordance with Appendix-F.

2.6.2 Detailing of reinforced concrete members shall be so developed that by virtue of it, maximum ductility is imparted in the structure. Appendix-E describes the suitable detailing of reinforced concrete structures.

3. LIMIT STATE DESIGN OF NON-PRESTRESSED REINFORCED CONCRETE STRUCTURES

3.1 General

3.1.1 The acceptable limit for safety, serviceability and other requirements is called a "Limit State". The provision of this standard aims at achieving the design with acceptable probabilities that the structure will not become unfit for the use for which it is intended, that is, it will not reach any of the limit states.

3.1.2 In the limit state design method the structure shall be designed to ensure an adequate safety and serviceability, stability, etc. for all relevant limit states. In general, the structure shall be designed on the basis of the most critical limit state and checked for other limit states.

Limit State of Strength

3.1.3 The limit state of strength of the structure or part of the structure corresponds to the state in which the structure or part thereof actually collapses or becomes unfit for usage. Separate limit states of failure in respect of strength in compression, flexure, shear, torsion or combined effects, are defined by different requirements of sectional capacities.

3.1.4 In the case of design for limit state of strength all structures and their parts shall be designed for the following limit state of strength, unless specified otherwise:

- flexure;
- axial force;
- axial force and flexure;
- shear;

- torsion; and
- shear and torsion.

Limit State of Serviceability

3.1.5 The limit state of serviceability of the structure or part of the structure corresponds to the state in which the structure or part thereof will have deflected or deformed or cracked to an extent that would prevent proper functioning of contained or nearby systems, jeopardise systems or other structures, or in any way impair the intended use of the structure or structural member.

3.1.6 In the case of design for limit state of serviceability, the following limit states shall be considered, unless specified otherwise:

- deflection; and
- cracking.

Limit State of Stability

3.1.7 The limit state of stability of structure corresponds to the following states of the structure:

(a) Rigid body movement of the overall structure under the action of destabilising force;

(b) Movement of the part of structures or whole structures due to elastic instability.

3.1.8 In the case of design for limit state of stability (rigid body movement of overall structures) the following limit states shall be considered, unless specified otherwise:

- overturning;
- flotation; and
- sliding.

Other Limit States

3.1.9 Depending on the design requirements, structures may be designed for any other relevant limit states (such as collapse mechanism, fatigue, etc.) in addition to those specified in cl. 3.1.3 to 3.1.8. In such cases all the additional limit states appropriate to that structure shall be considered.

3.1.10 In the case of design for other limit states, the design requirements of limit-state with respect to structural response shall be specified.

Characteristic Values of Strength and Load

3.1.11 Characteristic strength means that value of the strength of the material below which not more than 5% of the test results are expected to fall.

(a) The typical characteristic strength for different concrete grades should comply with AERB/SG/CSE-4;

(b) The characteristic strength of reinforcing steel and prestressing steel is the yield stress corresponding to 0.2% residual strain for high strength deformed bars and yield stress in case of mild steel as specified in the AERB/SG/CSE-4 or equivalent IS Code.

3.1.12 The characteristic load is the value of the load which has a 95% probability of not being exceeded during the life of a structure. In the absence of adequate data, the dead loads worked out on the basis of IS:875-part I and other loads on the basis of Code of Practice on Design for Safety in Pressurised Heavy Water Based Nuclear Power Plants (AERB/SC/D) and Code of Practice on Safety in Nuclear Power Plant Siting (AERB/SC/S) can be assumed as characteristic load.

Reinforcement Detailing

3.1.13 Reinforcement detailing shall be done following good engineering practices and the guidelines given in Appendix-E.

3.2 Requirement of Limit State Method of Design

3.2.1 The design shall satisfy the requirement of limit state by complying with the following criteria:

$$F_{sd} \leq \quad R_{sd}\,, \tag{3.1}$$

where,

F_{sd} = design value of structural response for a particular failure mode of a given limit state (ref. cl. 3.1.3 through 3.1.10),

R_{sd} = design value of resistance of members or the structure against failure mode of the given limit state considered in the design or the limiting value of design state under consideration, and

= resistance factor.

3.2.2 The structural response for i^{th} load combination is given by the following expression:

$$F_{sdi} = F_s\,(F_{di}) \tag{3.2}$$

where,

$F_s(F_{di})$ = Structural response determined by structural response analysis for i^{th} load combination F_{di}.

$$F_{di} = {}_{i} \qquad {}_{fij}\, F_{ij}\,, \tag{3.3}$$

where,

${}_{i}$ = load combination factor,

${}_{fij}$ = partial safety factor of load (load factor) for j^{th} individual load in i^{th} load combination, and

F_{ij} = j^{th} characteristics load for i^{th} load combination.

3.3 Structural Response Analysis for Limit States

3.3.1 Structures shall be analysed using appropriate method for factored load depending on the limit states under consideration.

3.3.2 (1) For structural analysis appropriate models are selected corresponding to geometry of structures, support conditions, loads and limit states to be examined;

(2) The structure may be analysed by assuming models such as slabs, beams, frames, etc. and their combination corresponding to their geometry.

3.3.3 (1) For structural analysis, loads may be modelled to give equivalent or conservative effects by simplifying load distribution or by replacing dynamic loads by static load;

(2) Thermal loading should be considered in the design using the methodology given in Appendix-A.

3.3.4 (1) Linear analysis shall be performed for structural response analysis for the design of new plant and except as permitted in cl. 8.3.2 of AERB/SS/CSE,

(2) For linear analysis, stiffness properties shall be calculated complying with requirements of cl. 2.4.5 and 2.4.6,

(3) In case of structural response analysis for limit states of serviceability, the following shall be considered:

(a) Irrespective of the assumption made in calculating stiffness for response analysis, the member forces associated with temperature changes and drying shrinkage may be obtained by taking into account the stiffness deterioration of members due to cracking;

(b) In computing the deflections and deformations of concrete structures the effects of stiffness deterioration due to cracks, drying shrinkage and creep, shall be taken into account.

3.3.5 To assess the safety of the existing structures, the strength can be evaluated corresponding to the collapse or partial collapse of the structure:

(1) Nonlinear structural analysis may be adopted provided the provisions of chapter-8 of AERB/SS/CSE are satisfied;

(2) Strength of existing structure by testing should be assessed in accordance with Appendix-H.

3.3.6 Member forces such as flexural moments, shear forces, axial force and torsional moment should be computed by appropriate analytical theories corresponding to limit states.

3.4 Design for Limit State of Strength

3.4.1 In designing structures or its parts for the limit states of strength as specified in Section 3.1, the provision of Indian standard Code of Practice for Plain and Reinforced Concrete (IS:456) should be adopted when no other guidelines are available. The methodology described in IS:456 is directly applicable to the design for limit state of strength corresponding to normal design condition. While designing for abnormal conditions, appropriate design charts, graphs, and equations should be used considering appropriate partial safety factor for materials.

3.4.2 Unless specified otherwise, the value of in equation (3.1), if considered in design shall be taken as follows:

= 1.0 for normal design condition; and

= 0.87 for abnormal design condition.

Loads for Limit State of Strength

3.4.3 Structures and structural members should be designed to have design strengths (determined according to the limit state of strength) at all sections at least equal to the required strengths calculated for factored loads and forces in such combinations as stipulated for loads in cl. 3.4.5 through 3.4.10 and as combined in accordance with the provisions specified in cl. 3.4.11 and 3.4.12.

3.4.4 (a) The loads as mentioned in this section should mean loads and/or related internal moments and forces due to effects of stipulated load described in AERB/SS/CSE. The effect of earthquake on structures, in accordance with AERB/SS/CSE, shall be considered in the design. For a detailed description of individual loadings given in the following clauses, Appendix-A of AERB/SS/CSE shall be referred.

(b) In the design, the effects created by application of loads and its interaction with the structure should be considered. The structure response due to effects arising from other forms of vibration, impact, differential settlements, creep, shrinkage and any type of transients should also be considered.

3.4.5 Normal Loads

DL dead load,

H lateral earth pressure,

LL live load,

P_v pressure loads resulting during normal operating condition,

P_t test pressure,

T_t thermal effects during test,

R_0 pipe and equipment reactions during normal operating or shutdown conditions,

T_0 thermal effects and loads during normal operating or shutdown conditions, and

F loads resulting from application of prestress.

3.4.6 Dynamic effects of live load should be considered in the analysis. In cases where a detailed dynamic analysis is performed for crane systems, elevators, or other moving machinery, the resulting load with dynamic amplification should be used. If such an analysis is not performed, the following increases over static effects should be used to account for the dynamic effects.

-	for supports of elevators	100 %
-	for cab operated travelling crane, support girders and their connections.	25 %
-	for pendant operated travelling crane, support girders and their connections.	10 %
-	for supports of light machinery, shaft, or motor driven equipment.	not less than 20 %
-	for supports of reciprocating machinery or power-driven units.	not less than 50 %.
-	for hanger supports	33 %

3.4.7 Crane Runway Horizontal Forces

In addition to the above, the horizontal forces on the crane runway should be considered in *LL*. These lateral forces on the crane runways due to the effects of moving crane trolleys should, if not otherwise specified, be 20% of the sum of the weights of the lifted load and that of the crane trolley, but exclusive of other parts of the crane. The longitudinal forces should, if not otherwise specified, be taken as 10% of the maximum wheel loads of crane applied at the top of the rail.

3.4.8 Severe Environmental Loads

E_0	loads generated by operating basis earthquake,
WC	loads generated by severe wind, and
FF	loads resulting from design basis flood.

3.4.9 Extreme Environmental Loads

E_{ss}	loads generated by safe shutdown earthquake, and
W_t	wind-induced missile load generated due to extreme wind.

3.4.10 Accidental Loads

F_h	hydrostatic load due to internal flooding,
MA	load and other effects of aircraft impact,
ME	load of missiles due to external events other than those related to wind or tornado, explosions in transportation systems, disintegration of turbine and other components,
MI	loading due to internal missiles,
MT	loading due to missiles, wind and overpressure generated from explosions in transportation systems, on land, water or in air,
Mt	loading effect of turbine missile,
P_a	maximum differential pressure load generated by postulated design basis accident
R_a	pipe and equipment reactions generated due to design basis accident,

T_a	thermal loads generated due to design basis accident.
Y_j	jet impingement load,
Y_m	impact load such as pipe whipping, and
Y_r	reaction due to broken high-energy pipe.

Load Combinations and Partial Factor of Safety for Loads (Load Factors)

3.4.11 (a) Unless specified otherwise load combinations [ref. equation (3.3)] given in Table 3.1 shall be used,

(b) Load combination No.18 of LC6 applies for internal structures of reactor building,

(c) For safety class 3 and design class DC3 structures which do not perform the safety functions associated with supporting the emergency core cooling systems and other systems related to safe shutdown of reactor or to prevent/mitigate the consequences of accident which could result in potential off-site exposure to relevant AERB guidelines, applicable load combinations do not include LC3 and LC6,

(d) For safety class 4 structures applicable load combinations are LC1 and LC2. The value of load combination factor $_i$ for load combination numbers 7 and 9 for load combination type LC2 is 0.75,

(e) Structural response should be evaluated both for short-term and long-term (considering the creep effect) for all applicable load combination,

(f) Effects due to shrinkage and heat of hydration could be considered as *DL*. While these effects are taken in load combination Nos. 1,3, 7 the load combination factor $_i$ shall be taken as 0.75 to obtain the design value,

(g) Wherever applicable, the impact effect of moving load should be

TABLE 3.1 : LOAD COMBINATIONS FOR LIMIT STATE OF STRENGTH

Design Condition and Load Combination Type		Load Comb. No.	Load Comb. Factor i	Load Factors fij																
				DL	$LL^{(1)}$	$H^{(2)}$	R_0	T_0	P_t	P_v	E_0 or WC or FF	E_{ss} or W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA/ME MI/MT or M_t	F_h
Normal Design Condition																				
LC1 :	Normal	1	1.0	1.4	1.6	1.6	1.6	-	-	-	-	-	-	-	-	-	-	-	-	-
	Load	2	0.75	1.4	1.6	1.6	1.6	1.4	-	-	-	-	-	-	-	-	-	-	-	-
	Combinations	3	1.0	1.4	1.6	1.6	1.6	-	-	1.6	-	-	-	-	-	-	-	-	-	-
		4	0.75	1.4	1.6	1.6	1.6	1.4	-	1.6	-	-	-	-	-	-	-	-	-	-
		5	1.0	1.4	1.6	1.6	-	-	1.6	-	-	-	-	-	-	-	-	-	-	-
LC2 :	Severe	6	1.0	0.9	-	-	-	-	-	-	1.6	-	-	-	-	-	-	-	-	-
	Environmental	7	1.0	1.4	1.6	1.6	1.6	-	-	-	1.6	-	-	-	-	-	-	-	-	-
	Load	8	0.75	1.4	1.6	1.6	1.6	1.4	-	-	1.6	-	-	-	-	-	-	-	-	-
	Combinations	9	1.0	1.4	1.6	1.6	1.6	-	-	1.6	1.6	-	-	-	-	-	-	-	-	-
		10	0.75	1.4	1.6	1.6	1.6	1.4	-	1.6	1.6	-	-	-	-	-	-	-	-	-

TABLE 3.1 : LOAD COMBINATIONS FOR LIMIT STATE OF STRENGTH (Contd.)

Design Condition and Load Combination Type	Load Comb. No.	Load Comb. Factor i	Load Factors fij																
			DL	$LL^{(1)}$	$H^{(2)}$	R_0	T_0	P_t	P_v	E_0 or WC or FF	E_{ss} or W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA/ME MI/MT or M_t	F_h
Abnornal Design Condition																			
LC3 : Extreme Environ-mental Load Combinations	11	1.0	0.9	-	-	-	-	-	-	-	1.0	-	-	-	-	-	-	-	-
	12	1.0	1.0	1.0	1.0	1.0	-	-	-	-	1.0	-	-	-	-	-	-	-	-
	13	1.0	1.0	1.0	1.0	1.0	1.0	-	-	-	1.0	-	-	-	-	-	-	-	-
LC4 : Abnormal Load Combinations	14	1.0	1.0	1.0	-	-	-	-	-	-	-	1.25	1.0	1.0	-	-	-	-	1.0
	15	1.0	1.0	1.0	1.0	1.0	1.0	-	1.0	-	-	-	-	-	-	-	-	1.0	-
LC5 : Abnormal Severe Envi-ronmental Load Combinations	16	1.0	1.0	1.0	-	-	-	-	-	1.15	-	1.15	1.0	1.0	1.0	1.0	1.0	-	-
	17	1.0	1.0	1.0	1.0	1.0	1.0	-	1.0	1.0	-	1.0	-	-	-	-	-	1.0	-
LC6 : Abnormal Extreme Env. Load Comb.	18	1.0	1.0	1.0	1.0	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-	-

Note: (1) All load combinations shall be checked for full and zero live load condition.
(2) Effect of lateral earth pressure shall be considered in design when it is critical.

included in live load.

3.4.12 Loads for Special Cases

Concrete structures, which by themselves are not classified as nuclear safety-related, but the deformation/collapse of which may cause damage to nearby nuclear-safety-related structures and components, should be evaluated for acceptable deformation/no collapse under the abnormal design conditions.

TABLE 3.2 : PARTIAL FACTOR OF SAFETY FOR MATERIALS FOR LIMIT STATE OF STRENGTH

Design Condition	γ_m	
	Concrete (γ_c)	**Steel (γ_s)**
Normal	1.5	1.15
Abnormal	1.3	1.0

TABLE 3.3: EXPRESSIONS OF *R* (f_k) FOR LIMIT STATE OF STRENGTH

Material	Limit States	$R(f_k)$
Concrete	Direct compression	$0.6\, f_{ck}$
	Flexure compression	0.67
	Direct tension	0.35
	Flexure tension	$0.55(f_{ck})^{1/2}$
	Shear	Ref. cl. 3.6.2
	Bond(1):Deformed bars in tension	$0.6(f_{ck})^{1/2}$
	:Plain bars in tension	$0.4(f_{ck})^{1/2}$
Steel	All limit states	f_y

Note: (1) For bars in compression, these values should be increased by 25%.

Design Strength and Partial Factor of Safety for Materials (Material Factors)

3.4.13 Design value of strength for a given limit state of strength is expressed by the following :

$$R_{sd} = R_s\ (R_d), \tag{3.4}$$

where,

$R_s(R_d)$ = computed resistance for member cross section which is determined from R_d,

$$R_d = (1/\ _m)\ R(f_k), \tag{3.5}$$

$_m$ = partial factor of safety for materials given in Table 3.2,

$R\ (f_k)$ = expression of respective material strength under consideration and is expressed in terms of f_k. (ref. Table 3.3), and

f_k = characteristic strength of materials.

Values of $_{\mathrm{m}}$ are given in Table 3.2 and shall be applied to relevant material strength denoted by $R(f_k)$ given in Table 3.3.

3.5 Limit State of Strength : Flexure and Axial Force

3.5.1 Structural elements shall be designed for all forces induced due to external loading effect irrespective of their disposition as members like beam, columns, slabs, etc..

3.5.2 For flexure members and for members subjected to combined flexure and compressive axial load, when the ultimate axial load (P_u) is less than 0.1 $f_{ck}\ A_g$, the tensile reinforcement provided should satisfy the following:

$$p_{tb} < 0.75\ p_b, \tag{3.6}$$

where,

p_{tb} = percentage of tensile reinforcement provided, and

p_b = percentage of tensile steel for cross-section when tension steel reaches the strain corresponding to its yield strength and concrete reaches the limiting strain level simultaneously.

The p_b value is calculated using stress strain relationship given in IS:456 and taking the value of $_c$ and $_s$ as unity. For members with compression reinforcement, the portion of p_b equalised by compressive reinforcement need not be reduced by factor 0.75.

3.5.3 Moment Magnification for Flexural Members

In frames not braced against lateral deflection flexural member shall be designed for the total end moment (M_T) as given below:

$$M_T = M_{ns} + \ {}_s M_s, \tag{3.7}$$

where, M_{ns} and $_s M_s$ are calculated for all the compression members at the joint. $_s$ and M_s are defined in cl. 3.5.7. M_{ns} is the factored end moment due to load that results in no appreciable lateral deflection, calculated by conventional elastic frame analysis.

(This clause is not applicable when structural analysis is done taking into account the influence of axial load, variable moment of inertia on member stiffness and fixed end moments, effect of lateral deflection on moment and forces, and effects of duration of loads).

Limit State of Strength : Axial Force

3.5.4 When a member is subjected to axial tension, the tensile capacity of the concrete shall be neglected and, the design reinforcement shall be calculated from the following expression.

$$P_u = (P_{bd}/100)\ (f_y /\ _s), \tag{3.8}$$

where P_u is the factored axial load (tensile), f_y is the strength of reinforcement and p_{bd} is the percentage of steel to be provided. Partial factor for safety steel of reinforcing $_s$ is given in Table 3.2. However, depending on crack control and other serviceability requirements (if any), suitable higher value of $_s$ should be taken.

Limit State of Strength : Combined Flexure and Axial Force

3.5.5 Effective Length of Compression Members

The effective length (l_{ef}) of column is determined from

$$l_{ef} = kl_c, \tag{3.9}$$

where,

l_c is the unsupported length of column, and

k is the effective length factor (ref. Appendix-D)

3.5.6 Consideration of Slenderness Effects

(a) For compression members braced against lateral deflection the slenderness effect may be neglected in cl. 3.5.8 when the slenderness ratio satisfies the following relationship:

$$kl_c/r \leq 34 \quad 12\ (M_1/M_2) \tag{3.10}$$

where (kl_c/r) is the slenderness ratio, r is the radius of gyration and M_1 and M_2 are as defined in cl. 3.5.7.

(b) For compression member not braced against lateral deflection, the slenderness effect may be neglected when

$(kl_c/r) \leq 22.$

(3.11)

(c) For compression members not braced against lateral deflection and slenderness ratio greater than 22, the slenderness effects can be considered, using provisions specified in cl. No(s) 3.5.8; 3.5.9 and 3.5.10,

(d) For all compression members with kl_c/r > 100 the design forces and moments should be determined from an analysis of structure considering the P- effect.

3.5.7 Calculations of End Moments, M_1 and M_2.

(1) For compression members in a frame braced against lateral deflection M_2 is taken as the larger end moment from a conventional frame analysis and M_1 the smaller. In such frames lateral drift effects can be neglected in the design of compression members and ${}_sM_s$ given in cl. 3.5.3 can be taken as zero provided that the lateral load in the direction under consideration is resisted by a primary lateral load-carrying member or members extending from the foundation to the upper extremity of the frame.

(2) For compression members in a frame not braced against lateral deflection, the calculation of the end moments M_1 and M_2 are based on two cases of lateral deflection for the frame, as follows:

(a) For combinations of vertical load which do not produce appreciable lateral deflections[(1)], M_2 is taken as the larger end moment, from a conventional frame analysis, and M_1 the smaller,

(b) When appreciable lateral deflections result from combinations of vertical load, or combinations of vertical and lateral load, M_2 and M_1 are taken as the larger and smaller end moments obtained by combining the moments, M_{ns}, plus the magnified moments, ${}_sM_s$. where M_{ns} is the factored end moment on a compression member due to load that result in no appreciable lateral deflection calculated by conventional elastic frame analysis.

(1) Appreciable lateral deflection means; lateral deflection $\geq L/1500$

M_s is factored end moment on a compression member due to loads which result in appreciable lateral deflection, calculated by conventional elastic frame analysis. The lateral drift effect $_{s}M_s$ shall be calculated as per cl. 3.5.10.

3.5.8 Member Stability Effect

(1) Compression members shall be designed using the factored axial load, P_u from an elastic analysis and a magnified factored moment M_c defined by

$$M_c = \ _{b}M_2, \tag{3.12}$$

where,

$$_{\mathrm{b}} = (C_m) / [1 - (P_u / \ _{m}P_c)] \geq 1.0, \tag{3.13}$$

where,

$_{\mathrm{m}} = 0.65$, and

$$P_c = \ ^2 EI \quad kl_c)^2. \tag{3.14}$$

In Equation (3.14) k shall be computed for braced member according to Appendix-D.

(2) In lieu of a more accurate calculation, EI in equation (3.14) may be taken as either

$$EI = (0.2E_cI_g + E_sI_{se}) / (1 + \ _{d}), \tag{3.15}$$

or conservatively,

$$EI = 0.25E_c I_g, \tag{3.16}$$

where,

I_g = moment of inertia of gross section about centroidal axis neglecting reinforcement,

I_{se} = moment of inertia of reinforcement about centroidal axis of member cross section, and

$_d$ = Absolute value of ratio of the maximum factored dead load moment to the maximum factored total load moment.

(3) In equation (3.13) the value of C_m shall be taken as follows:

(a) For members without transverse loads between supports.

$$C_m = 0.6 + 0.4\ (M_1 / M_2), \tag{3.17}$$

but not less than 0.4.

(b) For all other cases C_m shall be taken as 1.0.

3.5.9 If computations show no moment at both ends of a compression member in a frame or that the computed end eccentricities are less than the minimum stipulated eccentricity as in IS:456, M_2 in equation (3.12) should be based on the stipulated minimum eccentricity about each principal axis separately. The ratio M_1/M_2 in equation (3.17) shall be determined by either of the following:

(1) when computed end eccentricities are less than stipulated minimum eccentricity computed end moments may be used to evaluate M_1/M_2 in equation (3.17); or

(2) if computations show that there is essentially no moment at both ends

of a compression member, the ratio M_1/M_2 is taken as equal to 1.0.

3.5.10 Lateral Drift Effect

The magnified moment for lateral drift, $_sM_s$ referred in cl.3.5.3 and 3.5.7 should be calculated using a second-order analysis using appropriate values of member stiffness or shall be based on

$$_sM_s \ = \{(1)/[1 \quad P_u (\quad _m \quad P_c)]\} \ M_s \geq M_s. \tag{3.18}$$

In calculating P_c in equation (3.18), k shall be computed according to Appendix-D for unbraced condition. P_u and P_c are the summations for all the columns that are rigidly attached to beams or footings offering restraint in the storey under consideration. The value of $_m$ is 0.65.

3.6 Limit State of Strength : Shear and Torsion

3.6.1 In the design of members, shear force and torsional moment shall be treated simultaneously alongwith other forces if they act together.

Design Shear Strength

3.6.2 The design shear strength of concrete $_c$ is expressed as follows:

$$_c = (1/ \ _c) \ R(f_k), \tag{3.19}$$

where,

$$R(f_k) = [0.18 \quad (f_{ck})^{1/2}] \tag{3.20}$$

in which is given below:

(1) for member subjected to shear and flexure,
= 1, and

(2) for members subjected to shear and axial tension

$= 1 - (P_u / P_r)$, (3.21)

where P_u is the factored axial force normal to the cross sectional area of the members acting simultaneously with the factored shear force V_u under consideration, and P_r is the limiting axial tensile strength of the section without considering reinforcement. Both P_u and P_r are positive for tension. In calculating P_r , the influence of the co-existing flexural moment shall be taken into account. Alternatively, P_r may be taken as:

$$P_r = [0.35\ (f_{ck})^{½}]\ b_w d. \quad (3.22)$$

(c) For member subjected to shear and axial compression

$$= 1 + (3P_u / A_g f_{ck}) \text{ subject to } \leq 1.5 \quad (3.23)$$

in which P_u is the factored axial force as defined in (2) above and positive when compression.

Maximum Shear Strength

3.6.3 When the nominal shear stress $_v$ [where $_v = V_u / (b_w d)$] of a member exceeds $_c$, reinforcement for shear and torsion or their combination shall be provided. However, $_v$ shall not exceed $_{cmax}$, which is given by

$$_{cmax} = (1/\ _c)\ [0.88(f_{ck})^{½}]. \quad (3.24)$$

In case nominal shear stress $_v$ exceeds $_{cmax}$ the cross-section of the member shall be changed.

Shear Reinforcements

3.6.4 When shear reinforcements are required, they shall be designed by satisfying the following condition:

$$V_u \leq V_r \tag{3.25}$$

where,

V_r = Total shear resistance of the section

$$= V_c + V_s \tag{3.26}$$

V_c = Shear resistance provided by concrete

$$= {}_c b_w d \tag{3.27}$$

V_s = Shear resistance offered by reinforcement, and

${}_c$ = Design shear strength of the concrete and is given by equation (3.19).

Torsion

3.6.5 In design for torsion of reinforced concrete structures two conditions may arise:

(a) Equilibrium torsion, i.e. the torsional moment which is applied on the structure. This torsional moment must be resisted in order to maintain the force equilibrium of the overall structural system,

(b) Compatibility torsion, i.e. the torsional moment induced in the members of statically indeterminate structures to maintain the deformation compatibility of the structures.

3.6.6 In the case of equilibrium torsion, the members shall be designed for full value of torsion. In this case the torsional moment on members due to applied forces shall be determined using total torsional stiffness of the members determined for uncracked condition. When cracked section is considered for flexural stiffness, torque-twist diagram should be used for torsional stiffness calculation.

3.6.7 In the case of compatible torsion, the torsional moment on the member may be determined using a torsional stiffness value which is 20% the torsional stiffness value of the member calculated for uncracked section. This torsional stiffness value shall be used in the structural analysis along with flexural stiffness values calculated for uncracked section.

3.6.8 Torsional effect shall be included in the design when,

$$T_u > (1/\ _c)\ [0.15(f_{ck})^{½}](1/3)(\ \ x^2y), \tag{3.28}$$

where, T_u is the factored torsional moment, x is the shorter overall dimension of rectangular part of cross section, y is the longer overall dimension of rectangular part of cross section and $\ \ x^2y$ is the torsional section properties and are calculated as follows:

(i) For members with rectangular or flanged section, $\ \ x^2y$ is taken for the component rectangles of the section, but the overhanging flange width used in design shall not exceed three times the flange thickness,

(ii) A rectangular box section may be taken as a solid section provided the wall thickness h is atleast $x/4$. A box section with wall thickness less than $x/4$, but greater than $x/10$ may also be taken as solid section except that $\ \ x^2$y is multiplied by $4h/x$. When h is less than $x/10$, wall stiffness is to be considered. Fillets shall be provided at interior corners of all box sections.

3.6.9 When calculated torsional moment in the member is more than the value given in cl.3.6.8 torsional reinforcement shall be provided. The minimum torsional reinforcement shall be calculated from the following torsional moment.

$$T' = [0.18(f_{ck})^{½}]\ (1/3)\ (\ \ x^2y). \tag{3.29}$$

3.6.10 The following interaction equation shall be satisfied when a reinforced concrete section is designed for combined effects of shear and torsion:

$$(V_u \,/\, V_0)^2 \;+\; (T_u \,/\, T_0)^2 \;\leq 1.0 \qquad (3.30)$$

where V_u is the factored shear force acting on the element, T_u is the factored torsional moment acting on the element, V_0 is the shear strength of the beam which is equal to $(V_c + V_s)$ and T_0 is the strength of the section in pure torsion.

3.6.11 The following interaction equation shall be satisfied in the case of member subjected to flexure and torsion:

For yielding of bottom bars:

$$(1/r')(T_u/T_0)^2 + (M_u/M_0) \leq 1.0. \qquad (3.31)$$

For yielding of top bars:

$$(T_u/T_0)^2 \quad [r'\,(M_u \,/\, M_0)] \leq 1.0, \qquad (3.32)$$

where in equations (3.31) and (3.32) M_u is the factored moment acting on the section, M_0 is the pure flexural capacity of the section,
r' is the ratio of the yield forces of flexural tension and compression reinforcement i.e.

$$r' = A_{st} f_y \,/\, A_{sc} f_y \qquad (3.33)$$

where A_{st} is the area of tensile steel, A_{sc} is the area of compressive steel and f_y is the yield strength of the steel.

3.6.12 Expression of T_0 to be used on equations (3.30), (3.31) and (3.32) is given below.

$$T_0 = 2X_0Y_0 \left[\left(\frac{A_t f_y}{s} \right) \left(\frac{A_l f_{ly}}{2\,(X_0+Y_0)} \right) \right]^{1/2} \qquad (3.34)$$

where X_0 is the shorter dimension of stirrup, Y_0 the larger dimension of stirrup, A_t the area of one leg of stirrup, A_l the total area of longitudinal steel,

s the spacing of stirrup, f_{ly} the yield strength of longitudinal steel, f_y the yield strength of stirrup steel.

Shear Friction

3.6.13 The concept of shear friction is used to design against direct shear failures occurring on planes of weakness. The provision of shear friction design is applied where it is appropriate to consider shear transfer across a given plane, such as an existing or potential crack, an interface between dissimilar materials or an interface between two concretes cast at different times.

3.6.14 The resistance of cross sections subject to shear transfer shall be calculated as per provisions described hereinafter.

3.6.15 A crack is assumed to occur along the shear plane considered. The required area of shear friction reinforcement A_{vf} across the shear plane shall be designed using the following. Any other shear transfer design method is acceptable provided shear resistance computed by such method is in substantial agreement with results of comprehensive tests.

3.6.16 (1) When shear friction reinforcement is perpendicular to the shear plane, the following condition shall be satisfied.

$$V_u = (1/\ _s)\ A_{vf}\, f_y \mu, \quad (3.35)$$

where A_{vf} is the area of shear friction reinforcement, μ is the coefficient of friction in accordance with cl. 3.6.17.

(2) When shear friction reinforcement is inclined to the shear plane at an angle $\ _f$, such that shear force produces tension in the shear friction reinforcement, the following condition shall be satisfied.

$$V_u = (1/\ _s)\ A_{vf} f_y\ (\mu \sin\ _f + \cos\ _f). \quad (3.36)$$

3.6.17 The coefficient of friction μ in equations (3.35) and (3.36) of cl. 3.6.16 is as follows:

(1)

for concrete placed monolithically	:	1.25,
for concrete placed against hardened concrete with the surface intentionally roughened as specified in (2) below.	:	0.9,
for concrete placed against hardened concrete not intentionally roughened.	:	0.5, and
for concrete anchored to as-rolled structural steel by headed studs or by reinforcing beams as described in (3) below.	:	0.6.

(2) When concrete is placed against previously hardened concrete, the interface of shear transfer should be clean and free of laitance. If μ is assumed to be 0.9 , the interface shall be roughened to a full amplitude of approximately 5 mm.

(3) When shear is transferred between as rolled steel and concrete using headed studs or welds for reinforcing bars, the steels should be clean and free of paint.

3.6.18 When the shear force to be transferred through shear friction, the factored shear force shall be less than V_{max}. V_{max} is lesser of V_1 and V_2.

where,

$$V_1 = (1/\ {}_{c})\ (0.2\ f_{ck}\ A_{cv}), \quad (3.37)$$

$$V_2 = (1/\ {}_{c})\ (0.65\ A_{cv}), \quad (3.38)$$

where A_{cv} is the area of concrete section resisting shear transfer.

3.6.19 The net tension across shear plane shall be resisted by additional reinforcement. The net compression across shear plane may be considered when calculating the required shear friction reinforcement, provided that only the minimum value of the compression which will occur during the life of the structure is considered.

3.6.20 Shear friction reinforcement shall be appropriately placed along the shear plane and anchored to develop the yield strength on both sides by embedment, hook or welding to special devices.

3.7 Design for Limit State of Serviceability

3.7.1 Unless otherwise stated, the following limit states of serviceability should be considered.

- Deflection,
- Cracking.

3.7.2 Unless specified otherwise , shall be taken as unity (1.0).

Load Combinations for Limit State of Serviceability

3.7.3 (a) Load combinations given in Table 3.4 shall be used unless otherwise stated,

(b) In lieu of using load combinations given in (a) above for limit state of deflection and cracking, the structural response may be computed by factored load analysis corresponding to load combination given in cl. 3.4.11 and divided by factor $_a$. Unless otherwise determined by computation, the factor $_a$ shall be as follows:

for load combinations 1,3,5,6,7,9 - $_a = 1.5$,

for load combinations 2,4,8,10 - $_a = 1.2$, and for load combinations 11 to 18 - $_a$ = 1.0.

Limit State of Serviceability : Limiting Deflection Criteria

3.7.4 Reinforced concrete members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations which may adversely affect the strength and serviceability of structural and nonstructural elements.

3.7.5 At discontinuous edge of a two way slab, an edge beam shall be provided with a stiffness ratio which shall not be less than 0.8 where:

= ratio of flexural stiffness of beam section to flexural stiffness of slab,

= $(E_{cb}I_b) / (E_{cs}I_s)$,

E_{cb} = modulus of elasticity of beam concrete,

E_{cs} = modulus of elasticity of slab concrete,

I_b = moment of inertia about centroidal axis of gross section of beam. For monolithic or fully composite construction, a beam includes that portion of slab on each side of the beam extending to a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than 4 times the slab thickness,

I_s = moment of inertia about centroidal axis of gross section of slab,

= $h^3/12$ times width of slab bounded laterally upto the centre line of adjacent panel from the edge of the beam, and

h = slab thickness.

3.7.6 Unless otherwise specified by system requirements, the reinforced concrete construction should satisfy the deflection limits given in Table 3.5 for load

TABLE 3.4 : LOAD COMBINATION FOR LIMIT STATE OF SERVICEABILITY

Load Combination Type		Load Comb. No.	Load Factors f_{ij}																
			DL	$LL^{(1)}$	$H^{(2)}$	R_0	T_0	P_t	P_v	E_0 or WC or FF	E_{ss} or W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA or ME MI or MT or M_t	F_h
LC1	Normal Load Combinations	1	1.0	1.0	1.0	1.0	1.0	-	1.0	-	-	-	-	-	-	-	-	-	-
		2	1.0	1.0	1.0	-	-	1.0	-	-	-	-	-	-	-	-	-	-	-
LC2	Severe Environmental Load Combinations	3	1.0	1.0	1.0	1.0	1.0	-	1.0	1.0	-	-	-	-	-	-	-	-	-
LC3	Extreme - Environmental Load Combinations		4	1.0	1.0	1.0	1.0	1.0	-	-	-	1.0	-	-	-	-	-	-	-
LC4	Abnormal Load Combinations	5	1.0	1.0	1.0	-	-	-	-	-	-	1.0	1.0	1.0	-	-	-	-	1.0
		6	1.0	1.0	1.0	1.0	1.0	-	1.0	-	-	-	-	-	-	-	-	1.0	-

TABLE 3.4 : LOAD COMBINATION FOR LIMIT STATE OF SERVICEABILITY (Contd.)

Load Combination Type		Load Comb. No.	Load Factors *fij*																
			DL	$LL^{(1)}$	$H^{(2)}$	R_0	T_0	P_t	P_v	E_0 or WC or FF	E_{ss} or W_t	P_a	R_a	T_a	Y_j	Y_m	Y_r	MA or ME MI or MT or M_t	F_h
LC5	Abnormal Severe Environmental Load Combinations	7	1.0	1.0	1.0	-	-	-	-	1.0	-	1.0	1.0	1.0	-	-	-	-	-
LC6	Abnormal Extreme Environmental Load Combinations	8	1.0	1.0	1.0	-	-	-	-	-	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-	-

Note: (1) All load combinations shall be checked for full and zero live load condition.

(2) Effect of lateral earth pressure shall be considered in design when it is critical

combinations given in cl. 3.7.3.

3.7.7 (a) Reinforced concrete beams and one-way construction should satisfy the minimum depth/thickness requirements specified in Table 3.6, whereas two-way constructions should satisfy the requirements for minimum thickness as given in Table 3.7. When the minimum thickness requirements are satisfied the requirements of limiting deflection given in Table 3.5 for load combinations as per cl. 3.7.3 may not be checked in the design with the exception given in cl. 3.7.8.

(b) Lesser thickness may be used if it is determined by computation that the resulting deflections will not adversely affect strength and serviceability.

3.7.8 When deflection limits more stringent than those specified in Table 3.5 are required to ensure the proper functioning of systems or certain nonstructural systems, the minimum thickness specified in Tables 3.6 and 3.7 does not apply, and the members shall be sized such that calculated deflections are within the required limits.

3.7.9 When deflection computations are performed, these computations shall be based on loading condition critical for flexure (refer cl 3.7.3.). When calculations are performed, the sum of long term deflection due to all appropriate sustained loads, and the immediate elastic deflection due to all appropriate nonsustained loads should be considered. Due consideration shall be given to effective moment of inertia at each of these stages.

3.7.10 The long-term deflection should be determined in accordance with cl. 3.7.11 through 3.7.14 but may be reduced to the amount of long-term deflection that occurs after the attachment of the non-structural elements or the levelling of equipment. This amount of long-term deflection is determined on the basis of accepted engineering data relating to the time-deflection characteristics of members similar to those being considered.

TABLE 3.5 : MAXIMUM DEFLECTIONS FOR LOADS UNDER SERVICEABILITY CONDITION

Design Conditions	Beam	Slab(1)
Normal	L/400 or 20mm whichever is less.	L/320
Abnormal	L/250	L/200

Note : (1) For two-way construction L shall be replaced by L_x, in which L_x is the length of clear span in shorter direction of two-way construction measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.

TABLE 3.6 : MINIMUM DEPTH/THICKNESS OF BEAMS OR ONE WAY CONSTRUCTION (NONPRESTRESSED) UNLESS DEFLECTIONS ARE COMPUTED

Member	Simply Supported	One End Continuous	Both Ends Continuous	Canti-lever
Solid one-wayslab	L/20	L/25	L/25	L/7
Beams	L/10	L/12	L/16	L/5

Note: (1) Thickness of any one-way construction shall not be less than 150 mm thickness.
(2) Valid for normal weight concrete and reinforcement grade Fe 415.

(3) For reinforcement grade having yield strength of 250 MPa, the values obtained from the table are to be multiplied by 0.75.

TABLE 3.7 : MINIMUM THICKNESS OF TWO-WAY CONSTRUCTION (NONPRESTRESSED) UNLESS DEFLECTIONS ARE COMPUTED

Edge Continuity	**Minimum Thickness (mm)**	
	= 1.0	= 2.0
Simply suported	L_x /25	L_x /20
One short edge continuous	L_x /30	L_x /25
Two adjacent long edges or two adjacent short edges continuous	L_x /35	L_x /25

Note: (1) = ratio of clear spans in long and short directions of two-way slabs.

(2) Thickness of any two-way slab shall not be less than 150 mm.

(3) Valid for normal weight concrete and reinforcement grade Fe 415.

(4) For reinforcement grade having yield strength of 250 MPa, the thickness values in the table are to be multiplied by 0.9.

(5) For other values of the minimum thickness may be linearly interpolated.

Deflection of Beam or One-way Slab

3.7.11 Where deflections are to be computed, deflections that occur immediately on application of load should be computed by usual methods or formulas for elastic deflection, considering effects of cracking and reinforcement on member stiffness.

3.7.12 (a) Unless stiffness values are obtained by a more detailed analysis, immediate deflection is computed with the modulus of elasticity E_c for concrete, and with the effective moment of inertia as follows, but not greater than I_g.

$$I_e = I_g\,(M_{cr}/M_a)^3 + I_{cr}\,[1 \quad (M_{cr}/M_a)^3\,], \qquad (3.39)$$

where,

$M_{cr} = f_{cr}\, I_g / Y_t$,

I_{cr} = Moment of Inertia calculated for cracked section transferred to concrete, and

Y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension.

The values of f_{cr} and M_u are obtained from equation (2.1) given in cl. 2.3.8 and the load combinations given in cl. 3.7.3.

(b) Where the computation of deflection is to be based on I_g or I_{cr}, the deflection calculated by an analysis using I_g or I_{cr} may be used, if the deflection thus calculated is increased by a factor (I_g/I_e) or (I_{cr}/I_e).

3.7.13 For continuous members, effective moment of inertia may be taken as the average of values obtained from the above equation for critical positive and negative moment sections. For prismatic members, effective moment of inertia may be taken as the value obtained from the above equation at midspan for simple and continuous spans, and at support for cantilevers.

3.7.14 Unless values are obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members should be determined by multiplying the immediate deflection caused by the sustained load considered by the factor ${}_c$,

$${}_c = T/(1 + 50\,p_c)\,, \qquad (3.40)$$

where p_c, is the ratio, $A_{sc}/(bd)$ of the compression reinforcement is the value at midspan for simple and continuous spans and at support for cantilevers. Time dependent factor T for sustained loads may be taken from the Table 3.8.

TABLE 3.8 : VALUE OF *T* IN COMPUTATION OF LONG-TIME DEFLECTIONS

Duration of Load in months	***T***
60 or more	2.0
12	1.4
6	1.2
3	1.0

3.7.15 Where deflections are to be computed, those that occur immediately on application of load should be computed by the usual methods or formulas for elastic deflections. These computations shall also take into account the size and shape of the panel, the conditions of the support, and the nature of restraints at the panel edges. For such computations, the modulus of elasticity of concrete shall be that specified in accordance with cl. 2.3.7. When the effect of Poisson's ratio is neglected the effective moment of inertia shall satisfy the provisions of cl. 3.7.12 and 3.7.13. Other values may be used if they result in predictions of deflection in reasonable agreement with the results of experiments.

3.7.16 Unless values are obtained by a more detailed analysis or test, the additional long-term deflection for two-way construction shall be computed in accordance with cl. 3.7.14.

Limit State of Serviceability : Cracking

Permissible Crack Width

3.7.17 Unless otherwise stated permissible crack width W_c shall not be greater than the value given in Table 3.9, when crack width is calculated for sustained loading using the methodology given in Appendix-C. The limits given in the table are applicable when concrete cover is not greater than 100 mm.

TABLE 3.9 : PERMISSIBLE CRACK WIDTH (W_c)

Exposure Type	Crack width W_c (mm)
Normal	0.0075 C[(1)]
Corrosive	0.006 C
Severe Corrosive	0.005 C

Note : (1) C is the clear cover of reinforcement in mm.

3.7.18 In lieu of satisfying the requirement of permissible crack width of cl. 3.7.17, for flexure member, the design may satisfy the following criteria, unless otherwise required:

(a) Compliance with spacing requirements of reinforcement given in section E.2 of Appendix-E shall be followed to control flexural cracking. When f_y for tension reinforcement exceeds 250 N/sq.mm., cross-sections of maximum positive and negative moments shall be so proportioned that the following condition is satisfied.

$Z \leq 30$ KN/mm, for interior exposure, and
≤ 25 KN/mm, for exterior exposure,

where,

$$Z = f_s \, (d_c \, A_{tc})^{1/3},$$

d_c = thickness of concrete cover measured from extreme tension fibre of centre of bar or where located closest thereto, (mm)

A_{tc} = Effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that of reinforcement, divided by the number of bars (sq.mm). When flexural reinforcement consists of different bar sizes, the number of bars is computed as the total area of reinforcement divided by the area of the largest bar used.

f_s = Calculated stress in reinforcement at sustained loads (N/sq. mm.) is computed as the moment divided by the product of steel area and the internal moment arm. In lieu of such computations, f_s may be taken as 0.40 f_y.

(b) For structures subject to severe environmental exposure condition or designed to be leaktight, the above provisions for controlling cracks may not be sufficient. For such structures, special investigations and precautions shall be required as discussed in chapter-7.

Compression Members

3.7.19 Cracks due to flexure in compression member, except where leak tightness is important, design requirement and subject to a design axial load greater than 0.2 f_{ck} A_g, need not be checked. A member subjected to load lesser than 0.2 f_{ck} A_g should be treated as a flexural member for the purpose of crack control.

3.8 Design for Limit State of Stability

3.8.1 All structures shall be checked for stability (rigid body movement of the overall structure) for the limit states specified in cl. 3.1.8.

3.8.2 Safety against rigid body movement of the overall structure shall be checked using equation (3.1). In the examination, F_{sd} will be taken as the effect of disturbing force and R_{sd} as the restoring effect. The values of for different limit shall comply with Table 3.10.

TABLE 3.10 VALUES OF 1/ FOR LIMIT STATE OF STABILITY

Load Combination	1/		
	Flotation	**Sliding**	**Overturning**
0.9 $DL+H+E_0$	-	1.5	1.5
0.9 $DL+H+WC$	-	1.5	1.5
0.9 $DL+H+E_s$	-	1.1	1.1
0.9 $DL+H+FF$	1.1	-	-

4. ALLOWABLE STRESS DESIGN METHOD

4.1 General

4.1.1 The provisions contained in this chapter is applicable to design of reinforced concrete and plain concrete structures using Allowable Stress Design method. The design of structures or its parts by this method shall be done in accordance with requirements of Structural Design (Working Stress Method) of IS:456 with additional stipulations given hereinafter. The stipulations of this standard will govern, when there are discrepancies between similar stipulations of IS codes/ standards and this standard.

4.1.2 All provisions of this standard for concrete structural members, except those permitting moment redistribution shall apply to the structure or part of it designed by allowable stress design method. For design aspects of various structural elements not covered by this chapter, appropriate requirements of this standard apply.

4.1.3 All structures or parts thereof designed by Allowable Stress Design method shall be examined for limit state of strength as per chapter-3 of the present standard. However, elements subjected to flexure due to gravity load only need not be examined for limit state of strength.

4.2 Design Method

4.2.1 In Allowable Stress Design Method, unless specified otherwise, the strength of a structural member shall be confirmed by checking that the stress in reinforcement and concrete is less than the respective allowable stresses. Checking shall be by satisfying the following criteria

$$ {}_{\text{cal}} \leq {}_{a}, \qquad (4.1) $$

where ${}_{\text{cal}}$ is the calculated stress in concrete or reinforcement and ${}_{a}$ is the allowable stress.

Calculated Stresses

4.2.2 The calculated stress in design $_{cal}$ may be determined by elastic theory for the induced forces such as axial force, bending moments, shear forces and torsion.

4.2.3 Linear structural analysis shall be applied in determining the induced forces due to the effect of applied load.

Allowable Stresses

4.2.4 Allowable stresses shall be determined from the following expression:

$$_{a} = {}_{n}R\ (f_k) \qquad (4.2)$$

where,

$_{n}$ is the factor representing safety as given in Table 4.1,

$R\ (f_k)$ is the expression of respective stresses under considerations expressed in terms of f_k (ref. Table 4.2) and f_k the characteristic strength of materials.

4.2.5 When effects of temperature change and when shrinkages are considered, the value of $_{n}$ may be increased up to 1.15 times the value given in Table 4.1.

4.2.6 For safety class 4 structures, the value of $_{n}$ of cl. 4.2.4 should be increased by 1.33 times when earthquake or wind loads are considered. It should be increased by 1.5 times when the effect of temperature change, shrinkage, etc. are considered along with earthquake or wind load.

4.2.7 However, the value of $_{n}$ when increased in view of cl.4.2.5 and 4.2.6 shall not exceed 0.67 for concrete and 1.0 for steel.

4.2.8 In flexural members the allowable tensile stress is applicable at the centroid of tensile reinforcement subject to the condition that when more than one layer of tensile reinforcement is provided, the stress at the centroid of the outermost layer shall not exceed more than 10% of the allowable value.

TABLE 4.1 : VALUES OF $_n$ FOR ALLOWABLE STRESS DESIGN METHOD

Design condition	Stress Parameters		$_n$ Concrete	$_n$ Steel
Normal	Compression	Direct	0.25	0.45
		Bending	0.33	0.55
	Tension	Direct	0.30	0.55
		Bending	0.50	0.55
	Shear		0.40	0.55
	Bond	Plain bars	0.25	—
		Deformed bars	0.35	—
	Bearing		0.30	—
Abnormal	Compression	Direct	0.50	0.75
		Bending	0.60	0.87
	Tension	Direct	0.50	0.87
		Bending	0.67	0.87
	Shear		0.50	0.87
	Bond[(1)]	Plain bars	0.50	—
		Deformed bars	0.67	—
	Bearing		0.67	—

Note: (1) For bars in compression, increase n by 25%.

TABLE 4.2 : $R(f_k)$ FOR DIFFERENT STRESS PARAMETERS IN ALLOWABLE STRESS DESIGN METHOD

Material	**Stress parameters**	$R(f_k)$
Concrete	Compression	f_{ck}
	Tension	$0.7(f_{ck})^{½}$
	Shear(1)	$0.18\ (f_{ck})^{½}$
	Bond	$0.7(f_{ck})^{½}$
	Bearing Whole area	f_{ck}
	Bearing Local area(2)	f_{ck}
Steel	For all stress parameters	f_y

Note: 1. For values of ref. cl. 4.4.5.

2. $= (A_1 / A_2)^{½} \leq 2.0.$

where,

A_1 = supporting area for bearing which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal,

A_2 = loaded area at the column base.

Allowable Stress of Plain Concrete

4.2.9 Allowable stress in plain concrete shall not be more than the values calculated from cl. 4.2.4 with following limitation on its maximum value:

(a) Allowable compressive stress $\leq$ 4.5 MPa,

(b) Allowable tensile stress in flexure $\leq$ 0.25 MPa,

(c) Allowable bearing stress $\leq$ 5.0 MPa for whole area,

$\leq$ 10.0 MPa for local area.

4.2.10 Increase in allowable stresses for plain cement concrete is acceptable only when the effect of earthquake or wind is considered in accordance with cl. 4.2.6.

Reinforcement Detailing

4.2.11 Reinforcement detailing shall be done following good engineering practices and the guidelines given in Appendix-E.

4.3 Load

4.3.1 All individual loads described in Section 3.4 shall be considered in the Allowable Stress Design.

4.3.2 All applicable load combinations given in cl. 3.4.11 shall be considered in the allowable stress design taking the values of $_i$ and $_{fij}$ as unity (1.0).

4.4 Strength Design

Flexure

4.4.1 Strains are assumed to vary linearly in proportion to the distance from the neutral axis, except for deep flexural member.

4.4.2 In reinforced concrete member, the concrete is assumed to resist no tension.

4.4.3 The modular ratio, m, should be calculated from the following.

$$m = 1/(_c f_{ck})$$

where $_c$ is the limiting strain of concrete.

Compression with or without Flexure

4.4.4 Slenderness effect and lateral drift effect shall be considered appropriately in the design.

Shear and Torsion

4.4.5 The value of for $R(f_k)$ (ref. Table 4.2) in calculating allowable shear stress of a concrete member shall be calculated from the following.

(a) $= 1$, when there is no axial force.

(b) $= 1 \quad (P/P_r)$, (4.3)

where the member is subjected to axial tensile load, P_r is determined from the following expression:

$$P_r = 0.2(f_{ck})^{½}\, b_w d \tag{4.4}$$

(c) $= 1 + (5P/\, A_g f_{ck})$, but not exceeding 1.5 (4.5)

where P is the axial compressive force.

4.4.6 When shear reinforcement is provided, the nominal shear stress $_c$ shall not exceed $_{c\max}$ for beam or 0.5 $_{c\max}$ for slab, where $_{c\max}$ is given by

$$_{c\max} = \ _n\,[0.88(f_{ck})^{½}] \tag{4.6}$$

4.5 Serviceability and Stability Requirements

4.5.1 Serviceability requirements in terms of limiting deflection, span-to-depth ratio, crack width, etc. shall satisfy the requirements of section 3.7.

4.5.2 The stability requirement of section 3.8 shall be satisfied.

5. SPECIAL DESIGN REQUIREMENTS OF STRUCTURAL ELEMENTS

5.1 Slab

5.1.1 It is suggested that the flat, ribbed, hollow slab construction be avoided in buildings having primarily beam-column structural system.

5.1.2 The moment coefficients can be used in slab design, when the variation in the span does not exceed 15% of the longest span with substantially uniformly distributed loading. If the variation exceeds this value, the design moment may be adjusted using moment distribution approach starting from the moment calculated by coefficients. However, complicated case requires a detailed analysis.

5.1.3 Where a cantilever of a length exceeding one third of the adjacent span occurs, the condition of maximum load on the cantilever and minimum load on the adjacent span is a design consideration.

5.1.4 At any section, top reinforcement of slab should not be less than the mid-span reinforcement at bottom. This provision is not applicable for cantilever slab and slabs which are actually simply supported.

Shear Design for Slab

5.1.5 The shear strength of slabs in the vicinity of concentrated load or reactions shall be governed by the more severe of the following two conditions:

(a) Beam actions for slab or footing with a critical section extending in a plane across the entire width *b* and located at a distance *d* from the face of concentrated load or reaction area. For this condition, the design shall be in accordance with cl. 3.6.1 to 3.6.4,

(b) Two-way action of (punching shear) a slab with a critical section perpendicular to plane of slab or wall and located so that its perimeter b_0 is minimum, but need not approach closer than $d/2$ to the perimeter of the concentrated load or reaction area shall be designed in accordance with cl. 5.1.6. For square or rectangular load or reaction area, the critical section may be assumed to have four straight sides. Critical section located at a distance not closer than $d/2$ from the face of any change in slab thickness and located such that the perimeter, b_0 is a minimum, must also be investigated.

5.1.6 When the factored shear force V_u is not greater than V_c, the shear reinforcement need not be provided, where

$$V_c = \quad {}_c\, b_0 d \tag{5.1}$$

where,

$$\quad = (0.5 + \quad) \tag{5.2}$$

${}_c$ is defined in cl. 3.6.2 and is the ratio of clear spans in short and long sides of the concentrated load or reaction area, and b_0 is the perimeter of the critical section as defined in cl. 5.1.5(b).

5.1.7 ${}_v$ shall not be more than ${}_{c\max}$, where,

$${}_{c\max} = [(1/ \ {}_c)\ [0.5(f_{ck})^{½}\ ] \tag{5.3}$$

5.1.8 When nominal shear stress ${}_v$ exceeds the value of ${}_c$, shear reinforcement shall be provided. When shear reinforcement is required it shall be designed in accordance with cl. 3.6.4. In calculating V_c by equation (3.27), the parameter b_w is to be replaced by b_0.

Openings

5.1.9 (a) When the diameter or equivalent diameter of the opening is less than four times the slab thickness, the effect of opening may be taken care of by proper detailing of the reinforcement around the opening. Extra reinforcement equivalent to 1.25 times the reinforcement lost due to the opening should be provided around the edge along the spanning directions and also at a 45º of the span,

(b) When the diameter (or equivalent diameter in case of non-circular opening) of the opening is more than four times the slab thickness appropriate analysis shall be carried out.

5.1.10 When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when opening in a flat slab are located within column strip the critical slab section for shear defined in cl. 5.1.5 (b) shall be modified as follows:

(a) For slabs without shearheads, the part of perimeter of critical section that is enclosed by straight lines, projecting from centroid of the load or reaction area and tangent to the boundaries of the openings shall be considered ineffective,

(b) For slabs having column heads with or without drop the ineffective portion of the perimeter shall be one-half of that defined in (a) above.

5.2 Walls

General Requirements

5.2.1 The following provisions shall apply for design of walls subject to axial load, with or without flexure,

(a) Walls shall be designed for eccentric loads and any lateral or other loads to which they may be subjected,

(b) Walls subject to axial loads shall be designed in accordance with cl. 5.2.7 and cl. 5.2.8,

(c) Walls shall be anchored to intersecting elements such as floors, roofs, or the columns, pilasters, buttresses, and intersecting walls, and footings,

(d) Walls subjected to axial loads, combination of axial load and bending, axial load and in-plane bending shall be designed as per provisions of chapters-3 or 4,

(e) Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each vertical concentrated load shall not exceed centre-to centre distance between loads, nor the width of bearing plus 4 times the wall thickness,

(f) Outer limits of the effective cross section of spirally reinforced or tied reinforced compression member built monolithically with a concrete wall or pier shall be taken not greater than 40 mm outside the spiral or tie reinforcement.

Fire Resistance

5.2.2 The minimum wall dimensions and covers required to provide adequate resistance to fire, shall be obtained from Table E.4.

Minimum Reinforcement

5.2.3 Minimum vertical and horizontal reinforcement shall be in accordance with Appendix-E unless a greater amount is required for shear by cl. 4.2.4.

5.2.4 In addition to the required minimum reinforcement as given in Appendix-E not less than two 16 mm bars shall be provided around all openings. Such bars shall be extended for full development beyond the corners of the openings but not less than 600 mm.

Slenderness

5.2.5 The ratio of effective length or height of stocky walls to their thickness shall be 15 or less. The effective height is obtained by multiplying the clear height between floors by the effective length and height factor k given below:

- for walls restrained at top and bottom against lateral translation and restrained against rotation at one or both ends (top or bottom or both) : 0.8,

- for walls restrained at top and bottom against lateral translation and un-restrained against rotation at one or both ends. : 1.0,

- for walls not restrained against lateral translation but partially restrained against rotation. : 1.5,

- for free-standing walls. : 2.5

Empirical Design Method

5.2.6 Walls of solid rectangular cross-section may be designed by the empirical provisions given herein if the resultant of all factored loads is located within the middle-third of the overall thickness of wall and all limits of cl. 5.2.1 are satisfied.

5.2.7 Design axial force P_u of a wall satisfying limitations of the preceeding paragraph shall be computed using following expression:

$$P_u = (0.40/\ _c)\ f_{ck}\ A_g [1\ \ (kL_c/32h)^2] \tag{5.4}$$

where L_c is the clear vertical distance between supports, h is the overall wall thickness, and k is the effective length factor as given in cl. 5.2.5.

5.2.8 The minimum wall thickness designed by empirical design method shall be as follows:

(a) Load-bearing or exterior walls shall not be less than (1/25) the unsupported height or length, whichever shorter, nor less than 200 mm,

(b) In wet ground a minimum thickness of 300 mm shall be used. In any case, the thickness cannot be less than that of the wall above.

Walls Designed as Flexure Members

5.2.9 The cantilever retaining walls shall be designed for flexure and shear in accordance with chapter 3 and 4.

Shear Design

5.2.10 Design of shear forces perpendicular to the face of wall shall be in accordance with cl. 5.1.5 through 5.1.8.

5.2.11 Design of horizontal shear force in plane of wall shall be in accordance with cl. 3.6.2 through 3.6.3. For the purpose of design, d shall be taken as 0.8 L_w where, L_w is the horizontal length of wall unless specified otherwise.

5.2.12 Where two walls intersect the design for shear strength at interface and the design for shear strength at construction joints shall be in accordance with cl. 3.6.13 through 3.6.20.

Limiting Deflection Criteria of Wall

5.2.13 Walls subject to transverse loads shall also satisfy the requirements as specified in this chapter for non-prestressed one-way or non-prestressed two-way, prestressed construction, or composite construction, as appropriate.

Opening in Walls

5.2.14 Adequate reinforcement shall be provided to cater to stress concentrations developed at the corners of openings. This reinforcement should also take the form of diagnoal bars positioned at the corners of the openings, and shall be adequate to resist a tensile force equal to twice the shear force in the vertical components of the wall, but not less than two 16 mm diameter bars across each corner of the opening.

5.2.15 The wall with the opening of the area larger than the lesser of the following shall be analysed and suitably designed:

(1) 50% of wall panel area, and

(2) Area equivalent to circle of diameter having 4 times the wall thickness.

Walls as Grade Beams

5.2.16 Walls designed as grade beams shall comply with the requirement of reinforcement for moment and shear in accordance with provisions of chapter 3 and 4.

5.3 Foundations

5.3.1 Provisions herein shall apply for the structural design of foundations, i.e., isolated, combined and strip footings, raft/mat foundations, and piles and pile caps. The safety related to geotechnical aspects shall satisfy the provisions of safety guide, " Geotechnical Aspects for Buildings and Structures Important to Safety of Nuclear Facilities", AERB/SG/CSE-2.

Design Features

5.3.2 (a) The foundations shall be designed for geotechnical considerations using unfactored forces and moments transmitted to the founding strata based upon permissible soil pressure or permissible pile capacity selected by satisfying the requirements of AERB/SG/CSE-2,

(b) Foundation elements shall be designed to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this standard and as provided herein.

Shear Design

5.3.3 Design for shear in footings including the effect of two-way or punching shear shall be in accordance with cl. 5.1.5 to 5.1.8.

5.3.4 (a) Location of critical section for shear in accordance with cl. 5.1.5 shall be measured from face of column, pedestal, or wall for footings supporting a column, pedestal, or wall,

(b) For footings supporting a column or pedestal with steel base plates, the critical section shall be taken halfway between face of column and edge of steel base plate.

Special Design Features

5.3.5 Sloped or Stepped Footings

(a) In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section,

(b) Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

5.3.6 Combined Footings and Mats/Rafts

Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of this standard. 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.

5.3.7 Pile Caps

(a) Pile caps shall be provided with brace beams unless batter/raker piles are used,

(b) Minimum overall depth of pile cap shall be 450 mm,

(c) Minimum reinforcement in two orthogonal directions at the top and bottom faces of a pile cap shall be in accordance with Appendix-E,

(d) In addition, fully lapped, circumferential horizontal reinforcement consisting of bars not less than 12 mm diameter at a vertical spacing not more than 250 mm shall be provided.

5.3.8 Tie, Grade or Brace Beams

(a) The tie or grade beam shall be designed for all factored loads it may be subjected to,

(b) As a minimum, the brace beam shall be designed for an axial force equal to 10% of the larger axial load of the column foundations it may be connected to,

(c) The design axial load of the brace beam connecting pile caps shall be not less than 10% of the axial capacity of pile group being braced,

(d) In addition, the braced beam shall be designed for an appropriate simultaneous design load due to the overlying weight of the soil, slab-on-grade, live load on said slab-on-grade and all other loads supported by the brace beam.

6. PRESTRESSED CONCRETE STRUCTURES

6.1 General

6.1.1 Prestressed members shall meet the strength requirements specified in this chapter.

6.1.2 Design of prestressed members shall be based on strength and serviceability conditions at all load stages that may be critical during the life of the structure from the time prestress is first applied.

6.1.3 Provisions herein shall apply to members, prestressed with wire, strands, or bars, conforming to provisions for prestressing tendons in AERB/SG/CSE-4.

Scope

6.1.4 This chapter describes the acceptable design stipulations for prestressed concrete DC3 structures. The provisions in IS:1343 shall be adopted where no guidelines are available on the subject matter in this standard.

General Design Requirements

6.1.5 Only the following types of prestressed concrete construction are acceptable.

(a) Type-1 : no tensile stress allowed, and

(b) Type-2 : tensile stress allowed but no cracking allowed.

However, selection of a particular type from the above two for a structural member shall be made after giving due consideration to the importance of the structure towards safety.

6.1.6 Design shall be based on limit state method and/or allowable stress design method. The design aspects which are not covered in this chapter shall comply with the appropriate stipulations of relevent chapters of this standard.

6.1.7 The value of (ref. cl. 3.2.1) shall be taken as unity unless specified otherwise.

6.1.8 Stress increase due to age factor shall not be taken into account in calculating the strength of members for design of a new plant.

6.1.9 Loss of prestressing should be determined using the methodology given in section 6.7.

6.1.10 Use of low relaxation steel is recommended.

Reinforcement Limits

6.1.11 (a) The minimum nontension reinforcement to be provided shall be not less than 0.2% of the cross-sectional area. The minimum reinforcement shall be provided throughout the length and/or dimension of the member and shall be distributed uniformly on the section.

(b) The minimum shear reinforcement provided shall be 0.2% of the area of the member perpendicular to the plane at which shear force is acting.

6.1.12 Other limits on reinforcements shall be in accordance with Appendix-E.

Reinforcement Detailing

6.1.13 Reinforcement detailing shall be done following good engineering practices and the guidelines given in Appendix-E.

6.2 Limit State of Strength

6.2.1 The limit state of strength of the structure or its part shall be assessed from collapse of critical sections, elastic or plastic instability, and/or overturning. The resistance to bending, shear, torsion, and axial load at any section shall be less than appropriate value at that section produced by the most unfavourable combination of loads on the structures using appropriate partial factor of safety.

6.2.2 (a) All members shall be designed for the individual load described in cl. 3.4.3 through 3.4.10 and the load combinations given in cl. 3.4.11 and loads for special case (cl. 3.4.12),

(b) In the above load combinations the effect of prestressing force should be included, similar to a dead load.

6.2.3 The partial safety factor ${}_m$ for materials given in cl. 3.4.13 shall be used in the design of prestressed concrete member for limit state of strength.

Limit State of Strength : Flexure

6.2.4 All members shall be designed for the provision of flexure and flexure with axial load satisfying the provision of section 3.5.

6.2.5 The following condition shall be satisfied to limit prestressing:

$$(A_p / bd)\ (f_p / f_{ck}) \leq 0.18, \qquad (6.1)$$

where A_p is the area of prestressing tendon and f_p is characteristic strength of prestressing tendon.

Limit State of Strength : Shear

6.2.6 The design shear strength of concrete section shall not be more than

$${}_c = (1/\ {}_c)\ [f_t^2 + 0.8\ f_{cp}\ f_t]^{1/2}, \qquad (6.2)$$

where f_t is 0.15 $(f_{ck})^{1/2}$ and f_{cp} is the compressive stress at centroidal axis due to prestress taken as positive.

6.2.7 When the induced shear force due to factored shear force and/or factored torsional moment is more than ${}_c$, shear reinforcement shall be provided.

6.2.8 If the shear stress due to factored shear force and torsion is more than ${}_{c\max}$, the sectional dimension shall be changed. ${}_{c\max}$ is defined in cl. 3.6.3.

6.3 Examination of Stresses

6.3.1 Allowable stresses in concrete immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following:

extreme fibre stress in compression - $0.5\ f_{ci}$,

where $0.5\ f_{ck} \leq f_{ci} \leq 0.8\ f_{ck}$

extreme fibre stress in tension - $0.4\ (f_{ci})^{1/2}$,
when permitted.

in which f_{ci} is the characteristic strength of concrete at i^{th} day after casting.

6.3.2 Where computed tensile stresses exceed these values, bonded auxiliary reinforcement (non-prestressed or prestressed) shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

6.3.3 Allowable stresses in concrete (after allowance for all prestress loss) for all load combinations of section 4.3 shall not exceed the value calculated from cl. 4.2.4 through 4.2.7.

6.3.4 Allowable stresses in concrete of cl. 6.3.1 and 6.3.3 may be exceeded if shown by test or analysis that performance will not be impaired.

6.3.5 Permissible tensile stress in prestressing tendons shall not exceed the following unless justified otherwise:

(a) immediately after prestress transfer ($f_{p0,\max}$) - $0.7\ f_p$,

(b) average stress in the cable for all applicable load combinations:

normal design conditions	-	0.6 f_p,
abnormal design conditions	-	0.87 f_p.

6.4 Limit State of Serviceability

6.4.1 The limit state of serviceability to be considered in design class DC3 prestressed concrete structures shall be in accordance with section 3.7.

Limit State of Serviceability : Deflection

6.4.2 Immediate camber and deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of gross concrete section is used for uncracked sections.

6.4.3 Additional long-time camber and deflection of prestressed concrete members shall be computed taking into account stresses and strain in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

6.4.4 The computed deflection shall not exceed limits stipulated in Table 3.5.

6.5 Design for Limit State of Stability

6.5.1 Design for limit state of stability shall be done in accordance with section 3.8.

6.6 Special Design Requirements

Redistribution of Negative Moments

6.6.1 Redistribution of negative moments due to gravity loads in continuous prestressed flexural members allowed for limit state of strength shall not be more than 10%.

Slab

6.6.2 Design for factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be carried out in accordance with provisions of section 5.1 or by more detailed design procedure.

6.6.3 All serviceability limitations, i.e., including specified limits on deflections, etc. shall be met.

6.6.4 For normal live loads and loads uniformly distributed, spacing of prestressing tendons or groups of tendons in one direction shall not exceed 8 times the slab thickness, nor 1500 mm. Spacing of tendons also shall provide a minimum average prestress, after allowance for all prestress losses of 0.85 N/sq.mm on the slab section tributary to the tendon or tendon group. A minimum of two tendons shall be provided in each direction through the critical shear section over columns. Special consideration of tendon spacing shall be provided for slabs with concentrated loads.

6.6.5 Minimum nontension reinforcement to be provided shall be not less than 0.1% in each direction equally distributed at each face.

Tendon Anchorage Zones

6.6.6 Reinforcement shall be provided, where required, in tendon anchorage zones to resist bursting, splitting, and spalling forces induced by tendon anchorages. Regions of abrupt change in section shall be adequately reinforced.

6.6.7 End blocks shall be provided, where required, for support bearing or for distribution of concentrated prestressing forces.

6.6.8 Post-tensioning anchorages and supporting concrete shall be designed to resist maximum jacking force for strength of concrete during prestressing.

6.7 Losses of Prestressing

6.7.1 Prestressing losses due to following phenomena should be considered in the design:

(a) Instantaneous losses due to

- anchorage slip,
- friction, and
- elastic shortening.

(b) Long term losses due to

- shrinkage of concrete,
- creep of concrete, and
- relaxation of steel.

Loss of Prestress due to Anchorage Slip ($\boldsymbol{f_{pl1}}$***)***

6.7.2 Prestressing loss due to anchorage slip should be determined from the experimenal results or from the data supplied by prestressing system manufacture.

Loss of Prestress due to Friction ($\boldsymbol{f_{pl2}}$***)***

6.7.3 Loss of prestressing due to friction and wobble effect at a distance *x* from the operating anchorage device is given by

$$f_{pl2} = f_{p0,\max} [1 \quad e^{(\mu \quad + Kx)}], \tag{6.3}$$

where,

$f_{p0,\max}$ = stress due to maximum jacking force

= coefficient of friction in curve. In the absence of test data or manucfacturer’s data, the minimum value of may be taken as:

0.55 for steel moving on smooth concrete,

0.30 for steel moving on steel fixed to duct, and

0.25 for steel moving on lead

In circular constructions, where circumferential tendons are tensioned by jacks, the minimum value of μ for calculating friction may be taken as:

0.45 for steel moving on smooth concrete,

0.25 for steel moving on steel bearers fixed to the concrete, and

0.10 for steel moving on steel rollers.

K = Coefficient of Wobble effect. In the absence of test data, the values of K may be taken from the range 15 x 10 ^{4} to 50 x 10 ^{4} per meter.

= Total angular displacement over the distance x.

Loss of Prestress due to Elastic Shortening (f_{pl3}**)**

6.7.4 Such a loss occurs when prestressing tendons upon release from tensioning devices, cause the concrete to be compressed. This loss is proportional to modular ratio and intial prestress in concrete and shall be calculated as below:

(a) For pretensioning, the loss of prestress in tendons at transfer shall be calculated on a modular ratio basis using stress in the concrete surrounding the tendons, and

(b) For members with post-tensioned tendons which are not stressed simultaneously, there is a progressive loss of prestress during transfer due to gradual application of prestressing forces. The loss of prestress should be computed based on sequence of tensioning.

In the abscence of detailed analysis and test results, instantaneous prestressing loss due to elastic shortening may be taken as 1% of $f_{p0,\max}$.

Loss of Prestress due to Shrinkage of Concrete (f_{pl4}**)**

6.7.5 The loss of prestressng due to shrinkage of concrete shall be the product of the modulus of elasticity of steel and the shrinkage strain of concrete. The shrinkage strain of concrete should be calculated in accordance with the methodology given in Appendix-B.

Loss of Prestress due to Creep of Concrete (f_{pl5}**)**

6.7.6 Prestressing loss due to creep of concrete under load shall be determined for all permanent load including the prestress. It should be determied as the product of modulus of elasticity of prestressing steel and the ultimate creep strain of concrete determined in accordance with the methodology given in Appendix-B.

Loss of Prestress due to Relaxation of Steel (f_{pl6}**)**

6.7.7 Total percentage relaxation loss at time t hours after stressing of cables is given as

$$p(t) = k_1 \; k_2 \; p_{1000} \; k_{\mathrm{t}}, \tag{6.4}$$

where,

p_{1000} = 1000 hours relaxation at 20^{o}C for initial stressing of 0.7 f_p,

k_1 = correction factor for initial stress which could be determined from Figure 6.1,

k_2 = correction factor for temperature

= $T/20$ but not less than 1.0,

T = average temperature of the structure (^{o}C),

k_t = correction factors for time

= (t/1000) (6.5)

= 0.12 for normal relaxation steel,

= 0.19 for low relaxation steel.

Total Prestressing Loss

6.7.8 Total instantaneous loss of prestressing (f_{pls})

$$f_{pls} = \quad f_{pl1} + \quad f_{pl2} + \quad f_{pl3}. \tag{6.6}$$

For calculation of total long term loss, the value of inital prestressing (f_{pi}) shall be taken as

$$f_{pi} = f_{p0,\max} \quad f_{pls}. \tag{6.7}$$

Total long term prestressing losses (f_{pll})

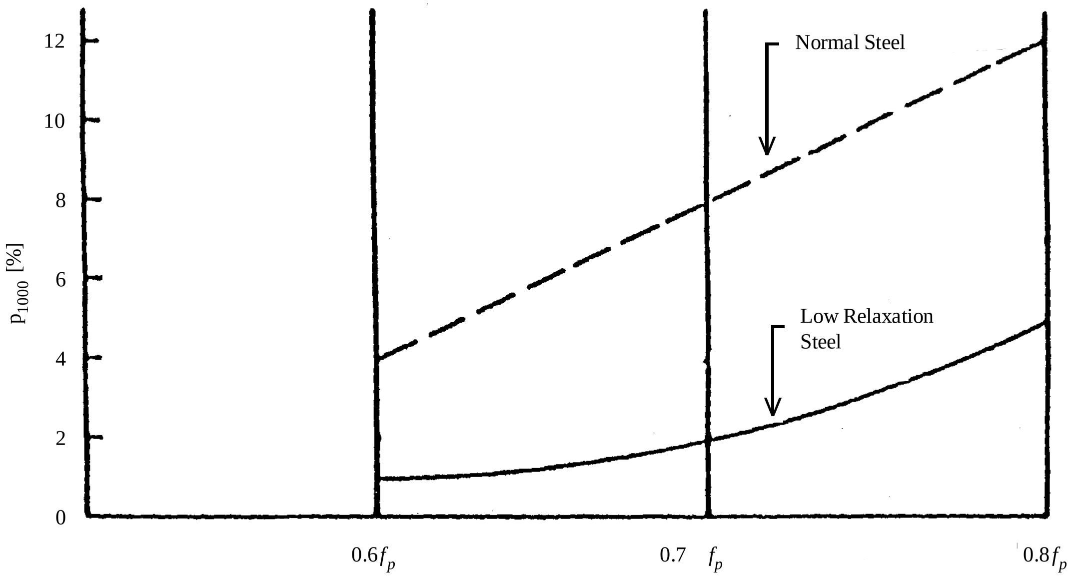


Fig. 6.1: RELAXATION LOSSES IN % FOR DIFFERENT STRESS LEVELS

$$f_{pll} = f_{pl4} + f_{pl5} + (5/6)\ f_{pl6.} \tag{6.8}$$

Total prestressing loss (f_p)

$$f_p = f_{pls} + f_{pll.} \tag{6.9}$$

7. LIQUID-RETAINING STRUCTURES

7.1 General

7.1.1 The provisions contained herein shall apply mainly to liquid-retaining structures for storage of liquids. The provisions also apply in designing of reinforced concrete and prestressed concrete structures for conveyance of liquids, such as aqueducts.

7.1.2 This chapter does not cover the requirements for reinforced and prestressed concrete structures for storage of hot liquids and liquids having low viscosity and high penetrating power, like petrol, diesel oil, etc. Special problems of shrinkage, arising from storage of non-aqueous liquids and the measures necessary where chemical attack is possible, are also not dealt with.

7.1.3 The recommendations generally apply to storage of aqueous liquids and solutions which have no detrimental action on concrete and steel, or where sufficient precautions are taken to ensure protection of concrete and steel from damages due to action of such liquids which may have corrosive action.

Testing

7.1.4 Testing of liquid-retaining structures, whenever required, shall be in accordance with provisions of Appendix-G.

7.2 Design Basis

7.2.1 Liquid-retaining concrete structures for the containment, treatment or transmission of water, liquid waste shall be designed and constructed to be water-tight with near-zero leak tightness under the required load combinations determined from the considerations of functional and safety requirements.

7.2.2 Design of liquid-retaining structures important to safety, strength, serviceability, stability requires, durability and low permeability all in equal measure. The concrete of these structure should possess the following qualities:

(a) be extremely dense and impermeable to minimise leakage (ingress or egress);

(b) provide maximum resistance to corrosion; and

(c) provide smooth surfaces to minimise flow resistance.

7.2.3 Design shall be carried out for all individual loadings given in AERB/SS/CSE and their suitable combinations, such as those described in chapter 3, 4 and 6.

7.2.4 (a) Design shall be carried out using allowable stress design methods as specified in chapter 4.

(b) In addition, effects arising from sloshing of liquid due to seismic or any other dynamic loading conditions shall be considered in the design.

(c) The design shall be checked for limit state of stability.

Steel liner, if provided for functional/safety requirements, shall be designed in accordance with standard on Design, Fabrication and Erection of Steel Structures important to safety of Nuclear Facilities, (AERB/SS/CSE-2).

Considerations for Leaktightness

7.2.5 The ability of a structure to retain liquids will be reasonably assured if:

(a) the concrete mixture is well proportioned and the concrete well consolidated without segregation;

(b) crack width is minimised;

(c) joints are properly spaced, sized, designed and constructed;

(d) impervious membrane (metallic or non-metallic) is used, and

(e) adequate reinforcing steel is provided.

7.2.6 Usually it is dependable to resist liquid permeation through the use of quality concrete, proper design of joint details, and adequate reinforcement rather than by means of an impervious protective barrier or coating. In addition, for some concrete structure to retain contaminated liquid (say spent fuel pool), impervious membrane of steel is provided.

7.2.7 Minimum permeability of concrete is obtained by using water cement ratios as low as possible consistent with satisfactory workability and good compaction. Impermeability increases with age of concrete and is improved by extended periods of moist curing. Surface treatment is important, and surface troweling and use of smooth forms give good impermeability.

7.2.8 Air entrainment reduces segregation and bleeding, increases workability, and provides resistance to the effects of freeze-thaw cycles. Thus the use of an air-entraining agent results in better compacted concrete and permits use of a lower water cement ratio. Other admixtures, such as water reducing agents, and pozzolanas are useful when they lead to increased workability and compaction with lower water cement ratios. Pozzolanas also reduce permeability.

7.2.9 Cracking can be kept to a minimum by proper design, reinforcement distribution and spacing of joints. Cracking caused by drying shrinkage can also be minimised by proper use of shrinkage-compensating concrete, but for the design to be successful, the engineer must recognise the characteristics and properties of shrinkage-compensating concrete. Some shrinkage will always occur in normal concrete, and joints and reinforcement should be designed to control the effects of shrinkage.

7.2.10 Joint design should also consider the movement resulting from thermal dimensional changes and differential settlements. Joints permitting movement along predetermined control planes that should form a barrier to the passage of fluids should include water stops. Good placement operations, adequate consolidation, and proper curing also are essential to control cracking.

7.2.11 For durability refer cl. 2.3.11 through 2.3.15.

7.3 Design Requirements

7.3.1 When load combinations given in cl. 3.7.3 are used for examining serviceability, the partial safety factor $_{fij}$ shall be taken as 1.0 for all individual loads.

Loads

7.3.2 All structures required for retaining liquids should be designed for both full and empty conditions, and the assumptions regarding arrangement of loading should be such as to cause the most critical effects. Particular attention should be paid to possible sliding and overturning.

7.3.3 Liquid loads should allow for the actual density of the contained liquid and possible transient conditions, e.g. suspended or deposited silt or grit where appropriate. For limit state of strength liquid level should be taken up to the top of walls assuming that liquid outlets are blocked. For serviceability examination the liquid level should be taken to the working level or the overflow level as appropriate to working conditions.

7.3.4 Allowance should be made for adverse effects of any soil pressures on walls, according to the compaction and/or surcharge of the soil and the condition of the structure during constructions and in-service. For normal design condition, no relief should be given for beneficial soil pressure effects on walls when the liquid-retaining structure is considered full.

7.3.5 Effect of solar radiation as well as that of contained liquid shall be considered. (An example of a critical adverse loading effect occurs when thermal expansion of a roof forces the walls of an empty structure into the surrounding backfill. In this case the passive soil pressure on walls may be limited by insertion of a thickness of compressible and durable material and/or by providing a sliding joint between the top of the wall and the underside of the roof. This can be either a temporary free sliding joint that is not cast into a fixed or pinned connection until reflective gravel or other solar protective material is placed on the roof, or a permanently sliding joint of assessed limiting friction. Movement of a roof may also occur where there are substantial variations in the temperature of contained liquid. Where a roof is rigidly connected to a wall this may lead to additional loading in the wall that should be considered in the design.)

7.3.6 Earth covering on reservoir roofs may be taken as dead load, but due account should be given to construction loads from plant and heaped earth which may exceed the intended design load.

Additional Requirements for Allowable Stress Design

7.3.7 Reinforced and prestressed members shall be so proportioned that induced stress in all load combinations induced stresses do not exceed permissible values. The requirements given hereinafter are in addition to those of chapter-4.

7.3.8 In the case of a member subjected to axial load and bending moment (i.e. an equivalent of an axial force with eccentricity) the following interaction equation is to be satisfied when equivalent eccentricity is less than 0.3 times the depth of section (depth of section being considered in the direction of eccentricity).

$$\frac{f_{cc}}{f_{ca}} + \frac{f_{bc}}{f_{ba}} \le 1 \qquad (7.1)$$

where f_{cc} is a calculated direct compressive stress in concrete, f_{ca} the permissible direct compressive stress in concrete (ref. 7.3.9 and 7.3.10), f_{bc} the calculated bending compressive stress in concrete and f_{ba} the permissible bending compressive stress in concrete (ref. cl. 7.3.9 and 7.3.10)

7.3.9 The allowable stress for concrete shall be determined using the provisions of cl. 4.2.4 through 4.2.7 and that for non-prestressed reinforcing steel shall be taken from Table 7.1.

7.3.10 For prestressed concrete cylindrical tanks the following conditions shall be satisfied.

(a) When the tank is full, there shall be a minimum compressive stress as follows:

normal design condition	:	0.67 N/sq.mm,
abnormal design condition	:	0.5 N/sq.mm.

(b) When the tank is empty, the tensile stress at any point shall not be more than the following:

normal design condition	:	0.75 N/sq.mm,
abnormal design condition	:	1.0 N/sq.mm.

(c) When a tank is to be emptied and filled at frequent intervals, or may be left empty for a prolonged period, the tank shall be designed such that there is a residual compression when the tank is either empty or full. —

Examination of Serviceability: Cracking

7.3.11 Unless otherwise stated, the permissible crack width for non-prestressed concrete shall not be more than the limiting values given in Table 7.2, when crack width is calculated using methodology given in Appendix-C. The limits given in the table are applicable when concrete cover is not greater than 100 mm.

7.3.12 Computed tensile stress shall be limited to (0.7 f_{ck}) at radioactive liquid-retaining face, otherwise liner shall be used.

7.3.13 In case of prestressed structures the general basis of design shall be in accordance with the provisions of chapter 6. The cracking of the liquid-retaining face, when unlined with a metallic liner, shall be avoided under all design load combinations.

Examination of Serviceability : Deflection

7.3.14 The provisions of serviceability as described in cl. 3.7.4 to 3.7.16 are applicable for horizontal members carrying uniformly distributed load unless stated otherwise.

7.3.15 For a cantilever wall which tapers uniformly away from the support and which is loaded with a triangular pressure, a net reduction factor should be applied to the ratios given in Table 3.6 if the thickness at the top is less than 0.6 times the thickness at the base. This reduction factor can be assumed to vary linearly between 1.0 and 0.78 where the thickness at the top varies between 0.6 and 0.3 times the thickness at the bottom.

TABLE 7.1: PERMISSIBLE STRESSES IN NON-PRESTRESSED STEEL REINFORCEMENT

Nature of Stress	Allowable Stress (N/sq.mm)			
	Mild Steel(1)		High-yield Strength Steel(2)	
	Normal	Abnormal	Normal	Abnormal
Tensile stress in members under direct tension	115	175	150	225
Tensile stress in members in bending:				
(a) On liquid-retaining face of members	115	175	150	225
(b) On face away from liquid for members less than 250 mm	115	175	150	225
(c) On face away from liquid for members 250 mm or more in thickness	125	190	190	290
Tensile stress in shear reinforcement				
(a) For members less than	115	175	150	225

250 mm

(b) For members 250 mm or more in thickness | 125 | 190 | 175 | 265

Compressive stress in columns subject to direct load	125	190	175	265

Note: (1) Round mild steel bars conforming to IS: 2062.
(2) High yield strength deformed bars conforming to IS:1786.

TABLE: 7.2 : LIMITING VALUES OF CRACK WIDTH FOR LIQUID RETAINING STRUCTURES

Type of liquid	Design condition	Allowable crack-width (1) for various exposure types		
		Normal	Corrosive	Severe corrosive
Non-radioactive liquid	Normal	0.005c(2)	0.004c	0.003c
	Abnormal	0.007c	0.006c	0.005c
Radioactive liquid	Normal	0.004c	0.003c	—(3)
	Abnormal	0.006c	0.005c	—

Note: (1) Unit of crack width is in mm,
(2) c is the clear cover,
(3) When a liquid retaining structure for radioactive liquid is to be constructed in the severely corrosive environment, proper precaution has to be taken such that the operative environment is maintained equivalent to the corrosive environmental condition.

7.3.16 In addition, allowance should be made for the significant additional deflection which occurs at the top of the wall due to rotation, if the pressure distribution under the base is triangular or vary asymmetrically trapezoidal. Limits for deflections will normally be those for non-liquid-retaining structures since only in exceptional circumstances will deflections be more critical with regard to freeboard, drainage or redistribution of load.

7.3.17 At least 75% of the liquid load should be considered as permanent when calculating deflections.

7.3.18 Retaining walls should be backfilled in even layers around the structure, the thickness of the layers being specified by the designer. Overcompaction adjacent to the wall should be avoided otherwise large differential deflections (and sliding) of the wall may occur.

7.3.19 Minimum Reinforcement for Temperature and Shrinkage

(a) The temperature and shrinkage reinforcement should be not less than the ratio given in Fig. 7.1,

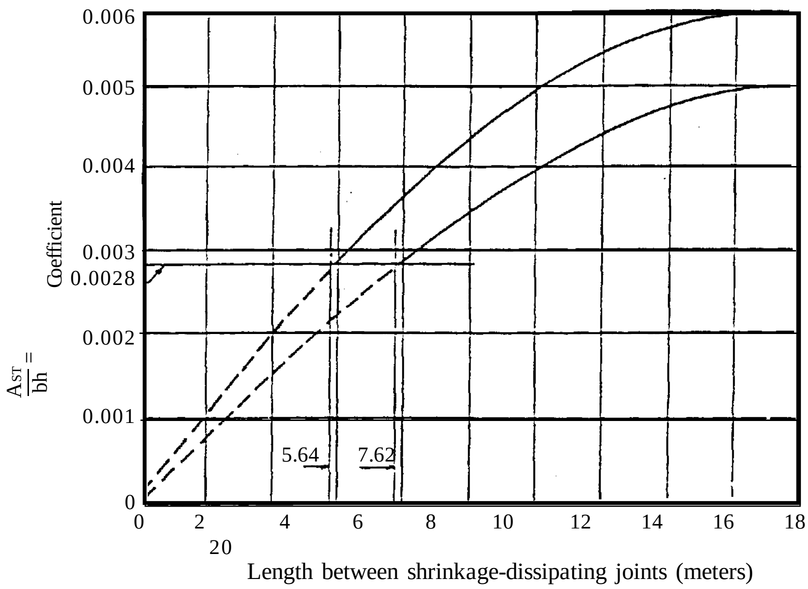


Fig. 7.1: RATIO OF SHRINKAGE AND TEMPERATURE REINFORCEMENT FOR CONCRETES

(b) If section thickness is more than 600 mm, reinforcement should be provided on both faces. The area of reinforcement at each face should be determined on the basis of 300 mm thickness,

(3) The reinforcement diameter should not be less than 10 mm and maximum spacing should be limited to 300 mm c/c on each face.

7.4 Limit State of Stability

7.4.1 The design shall comply with the requirements of limit state of stability in accordance with Section 3.8.

8. PRECAST AND COMPOSITE CONSTRUCTION

8.1 General

8.1.1 Provisions of this chapter apply to aspects related to the design and construction of precast concrete structural elements and composite construction (related primarily to reinforced concrete portion).

8.1.2 A precast concrete member is defined as concrete element cast at a place other than its final position in the structure. All provisions of this standard, not specifically excluded, and not in conflict with provisions herein, shall apply to precast concrete members.

8.2 Precast Concrete Structures

8.2.1 In precast construction that does not behave monolithically, effects at all interconnected and adjoining details shall be considered to assure proper performance of the structural system.

Design Principles

8.2.2 (a) Design of precast members shall consider all loading and restraint conditions from initial fabrication to completion of the structure, including form removal, storage, transportation and erection,

(b) Effects of initial and long-time deflections shall be considered, including effects on interconnected elements,

(c) Design of joints and bearings shall include effects of all forces to be transmitted, including shrinkage, creep, temperature, elastic deformation, wind and earthquake,

(d) All details shall be designed to provide for manufacturing and erection tolerances and temporary erection stresses.

Detailing

8.2.3 Reinforcement detailing of precast concrete members should be done in accordance with the relevant requirements of Appendix-E.

8.2.4 All details of reinforcement, connections, bearing seats, inserts, anchors, concrete cover, openings, lifting devices, fabrication, and erection tolerances shall be shown on the shop drawings.

8.2.5 When approved by the authorised engineer, embedded items (such as dowels or inserts) that either protrude from concrete or remain exposed for inspection may be embedded while concrete is in a plastic state, provided the embedded items are:

(a) not required to be hooked or tied to reinforcement within plastic concrete;

(b) maintained in correct position while the concrete remains plastic; and

(c) properly anchored to develop required factored loads.

Construction Aspects

8.2.6 (a) Each precast member or element shall be marked to indicate the location in the structure, top or outside surface, and the date of fabrication,

(b) Identification marks shall correspond to the placing plans.

8.2.7 During curing, form removal, storage, transportation and erection, precast members shall not be overstressed, warped, or otherwise damaged or have camber adversely affected.

8.2.8 Precast members shall be adequately braced and supported during erection to insure proper alignment and structural integrity until permanent connections are completed.

8.3 Composite Structural Elements

8.3.1 Provisions of this section apply to the design of composite concrete flexural members defined as cast-in-situ concrete elements constructed in separate placements but so interconnected that all elements act as a unit.

Analysis and Design Assumptions

8.3.2 An entire composite member or portions thereof may be used in resisting shear and moment.

8.3.3 Individual elements shall be investigated for all critical stages of loading.

8.3.4 If the specified strength, unit weight or other properties of various elements are different, properties of individual elements or the most critical values, shall be used in design.

8.3.5 In strength computations of composite members, no distinction shall be made between shored and unshored members.

8.3.6 Stress for normal design conditions shall be evaluated for both shored and unshored members.

8.3.7 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

8.3.8 Reinforcement shall be provided as required to control cracking and to prevent separation of individual elements of composite members.

8.3.9 Composite members shall meet the requirements for control of deflections in accordance with section 8.3.21.

Shoring

8.3.10 When used, shoring shall not be removed until supported elements will have developed design properties required to support all loads, and to limit deflections and cracking during removal of shoring.

Vertical Shear Strength

8.3.11 When an entire composite member is assumed to resist vertical shear, the design shall be in accordance with requirements of chapter 3 as for a monolithically cast member of the same cross-sectional shape.

8.3.12 Shear reinforcement shall be fully anchored into interconnected elements.

8.3.13 Extended and anchored shear reinforcement may be included as ties for horizontal shear.

Horizontal Shear Strength

8.3.14 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

8.3.15 Unless calculated in accordance with cl. 8.3.16 design of cross-sections subject to horizontal shear shall be based on

$$V_u \leq V_{nh}, \tag{8.1}$$

where V_u is the factored shear force at the section considered and V_{nh} is the factored shear resistance calculated in accordance with the following:

(a) When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} (in Newton), shall not be greater than 0.42 $b_v d$, in which b_v is the width of cross section at contact surface being investigated for horizontal shear (in mm), and d is the distance (in mm) from the extreme compression fibre to the centroid of tension reinforcement for entire composite section,

(b) When minimum ties are provided in accordance with cl. 8.3.18 through 8.3.20 and contact surfaces are clean and free of laitance, but not intentionally roughened, V_{nh} shall not be taken greater than 0.42 $b_v d$,

(c) When minimum ties are provided in accordance with cl. 8.3.18 through 8.3.20 and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 6 mm, V_{nh} shall not be taken greater than 1.8 $b_v d$,

(d) When factored shear force V_u at section considered exceeds 1.8 $b_v d$, the design for horizontal shear shall be in accordance with cl. 3.6.11 through 3.6.18.

8.3.16 Horizontal shear may be investigated by computing the actual compressive or tensile force in any segment, and provisions made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force shall not exceed horizontal shear strength as given in cl. 8.3.15.

8.3.17 When tension exists across any contact surface between interconnected elements, shear transfer by contact may be assumed only when minimum ties are provided in accordance with cl. 8.3.18 through 8.3.20.

Ties for Horizontal Shear

8.3.18 When ties are provided to transfer horizontal shear, tie area and spacing shall conform to the requirement of Appendix-E. Tie spacing shall neither exceed 4 times the least dimension of supported element nor 600 mm.

8.3.19 Ties for horizontal shear may consist of single bars or multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed).

8.3.20 All ties shall be fully anchored into interconnected elements.

Limiting Deflection Criteria

8.3.21 If composite flexural members are supported during construction so that, after removal of temporary supports, dead load is resisted by full composite section, the composite member may be considered equivalent to a monolithically cast member for computation of deflection. For non-prestressed members considered equivalent to a monolithically cast member, the values for minimum member thickness given in Tables 3.6 or 3.7 as appropriate, shall apply. If deflection is computed, account should be taken for curvatures resulting from differential shrinkage of cast-in-situ components and of axial creep effects in a prestressed concrete member.

8.4 Composite Construction of Steel and Concrete Elements

8.4.1 Provision of this section applies to design of flexural composite section having structural steel and concrete elements so interconnected that all elements act as a unit.

Flexural Strength of Composite Section

8.4.2 Composite structural elements (with steel and concrete material) shall be designed satisfying the provisions of this chapter and those stipulated in the standard on Design, Fabrication and Erection of Steel Structures Important to Safety of Nuclear Facilities (AERB/SS/CSE-2). All requirement of this standard shall apply to composite concrete flexural members, except as specifically modified herein.

8.4.3 The strength of a fully composite section shall depend on yield strength and section properties of steel elements (ref. AERB/SG/CSE-4), concrete element strength, and the interaction capacity of the shear connectors joining the concrete and steel elements. The moment of resistance of the composite section should be determined using allowable stress design method.

8.4.4 In determining the moment capacity, concrete shall be assumed to take only compressive stress.

8.4.5 The composite section (slab beam construction) shall be proportioned to ensure that neutral axis shall lie within the slab and the steel beam is fully in tension.

8.5 Composite Construction with Steel and Concrete for Compression Members

8.5.1 Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipe or tubing with or without longitudinal bars.

8.5.2 Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

8.5.3 Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

8.5.4 All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe or tube.

8.5.5 To evaluate slenderness effects, radius of gyration of a composite section shall not be greater than the value given by

$$r = \left[\frac{(E_c I_g/5) + E_s I_t}{(E_c A_g/5) + E_s A_t}\right]^{1/2} . \qquad (8.2)$$

In lieu of a more accurate calculation, EI in Eq. (3.14) may be taken either as Eq. (3.15) or

$$EI = \frac{(E_c I_g/5)}{} + E_s I_t , \qquad (8.3)$$

$1 + \quad d$

where A_t is the area of structural steel shape, pipe or tubing in a composite section and I_t is the moment of inertia of structural steel shape, pipe or tubing in a composite member cross-section.

Structural Steel Encased Concrete Core

8.5.6 For a composite member with concrete core encased by structural steel, the thickness of steel encasement shall not be less than

$b\ (f_y\ /\ 3E_s)^{1/2}$, for each face of width b,

nor

$h\ (f_y\ /\ 8E_s)^{1/2}$, for circular sections of diameter h.

8.5.7 Longitudinal bars located within encased concrete core may be considered in computing A_t and I_t.

Spiral Reinforcement around Structural Steel Core

8.5.8 A composite member with spirally reinforced concrete around a structural steel core shall conform to the following:

(a) characteristic strength of concrete f_{ck} shall be not less than 20 MPa;

(b) design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 350 MPa;

(c) spiral reinforcement shall conform to cl. E.4.8;

(d) longitudinal bars located within the spiral shall not be less than 0.01 nor more than 0.08 times the net area of concrete section; and

(e) longitudinal bars located within the spiral may be considered in computing A_t and I_t.

Tie Reinforcement around Structural Steel Core

8.5.9 A composite member with laterally tied concrete around a structural steel core shall conform to the following:

(a) characteristic strength of concrete f_{ck} shall be not less than 20 MPa;

(b) design yield strength of structural steel core shall be the specified minimum yield strength of grade of structural steel used but not to exceed 350 MPa; and

(c) lateral ties shall extend completely around the structural steel core and should satisfy the requirement of cl.E.4.7.

APPENDIX-A

THERMAL CONSIDERATIONS

A.1 General

A.1.1 Reinforced concrete structures important to safety of nuclear facilities shall conform to the minimum provisions of this standard and to the special provisions given herein for structural members subject to temperature variation of following types:

(a) Position dependent:

Variation through thickness and along the geometric centre of the member,

(b) Time-dependent:

Transient temperature distribution and the final steady-state condition.

These temperature variations are produced by combinations of ambient temperatures, operating temperatures and accident or abnormal temperatures.

A.1.2 Example of structures important to safety or part of them subjected to significant ambient, operating and/or accident temperatures are major shielding members, external walls and slabs, pipe supports and restraints and any structure in the vicinity of high energy piping system.

A.1.3 The provisions herein shall apply to concrete structures which are subjected to the above mentioned temperature variations and which have restraint such that thermal strains would result in thermal stresses.

A.1.4 The provisions herein do not address temperature requirements during curing, nor does it address the conventional shrinkage and temperature reinforcement.

A.2 Limiting Value of Concrete Temperatures

Normal Operation

A.2.1 The following temperature limitations shall be enforced for normal operation or any other long term period:

(a) temperature shall not exceed 65°C in general; and

(b) in local areas, such as around penetrations, the temperatures shall not exceed 90°C.

Accident Conditions

A.2.2 The following temperature limitations are for accident or any other short-term period:

(a) the general surface temperature shall not exceed 175°C; and

(b) in local areas the temperature may reach 340°C due to ejection of steam or water jet in the event of pipe failure.

Effect of High Temperature on Design Strength

A.2.3 Temperature higher than that given in cl. A.2.1 and A.2.2 may be allowed for concrete if tests are provided to evaluate the reduction in strength, and if this reduction is applied in designing the section.

A.2.4 If temperatures higher than that given in cl A.2.1 and A.2.2 are permitted, evidence shall be provided which verifies that the increased temperatures do not cause deterioration of the concrete either with or without load, and the required fixed water for concrete in radiation shield is not removed.

A.3 Design Approach

Assumptions

A.3.1 Linear thermal strain shall be assumed to cause stress under conditions of restraint, and a portion of such stress may be self-relieving. Mechanisms for such relief are: cracking, yielding, relaxation, creep and other time-dependent deformations.

A.3.2 Design provisions herein are based on the limit state design methods acceptable to this standard. The assumptions, principles and requirements specified in this standard shall be applicable for both normal design conditions and abnormal design conditions defined in cl. 2.1.3.

A.3.3 Serviceability of the structure shall be ensured under steady-state temperature conditions. Deflection and cracks shall be controlled in accordance with provisions given in section 3.3.

Design Requirements *(ref. Fig. A.1)*

A.3.4 (a) The effects of gradient temperature distribution and the difference between mean temperature distribution (T_m) and base temperature (T_b) during normal operation or accident conditions shall be considered. Refer Fig. A.1 depicting different temperature distributions across a structural member,

(b) The mean temperature distribution T_m is defined as

$+0.5h$

$$T_m = \frac{1}{h} \int_{0.5h} (T_{NL} \quad T_b)\mathrm{d}y, \qquad \text{(A3.1)}$$

where, h is the overall thickness of the member; T_b the base temperature and T_{NL} the nonlinear temperature distribution.

The integral is over the area bounded between T_{NL} and T_b. The T_{NL} - curve may be replaced by the equivalent linear temperature distribution, T_L (Fig. A.1) so that the latter produces uncracked moment about the centre line of the section same as the T_{NL}-curve would produce. The equivalent linear temperature distribution has an average temperature of T_m and a gradient T. For rectangular sections,

$$T = \frac{12}{h^2} \int_{0.5h}^{+0.5h} T_{NL}\, y\mathrm{d}y. \qquad \text{(A3.2)}$$

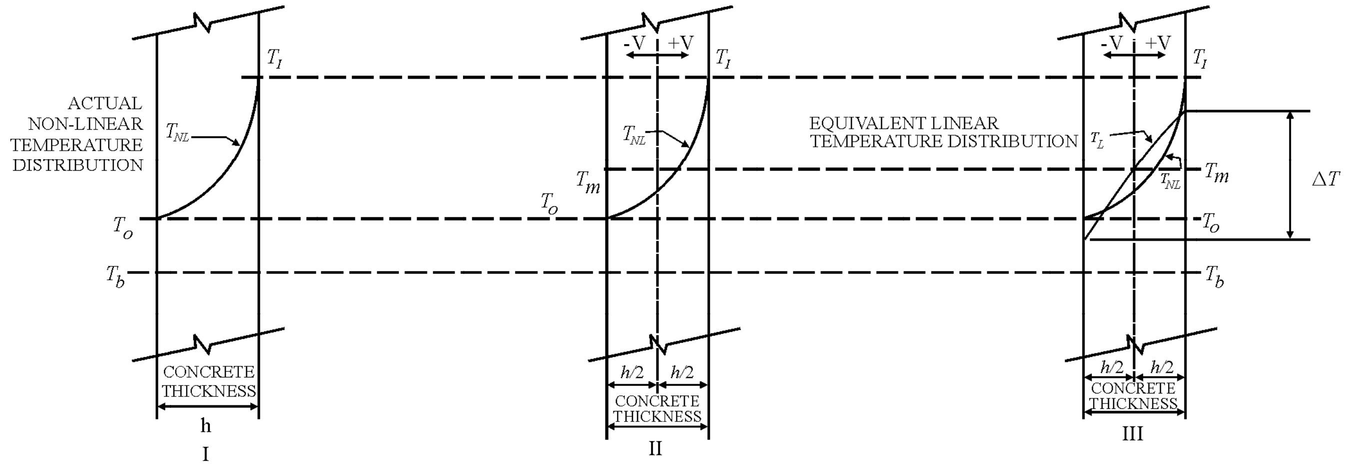
This integral is the moment of the area under T_{NL} - curve about the section centre line.

A.3.5 Time-dependent variations of temperature distributions shall be considered in evaluating thermal strains for both normal and abnormal design conditions.

A.3.6 Thermal stress shall be evaluated considering the rigidity of the section (which depends on cracking of cross-section), stiffness of the member (which depends on variation of cracking along the length of member), and the degree of restraint of the structure (freedom of the member to move under thermal loads). The evaluation may be based on cracked section properties, provided the following conditions are met:

(a) the tensile stress for any section exceeds 0.7 $(f_{ck})^{1/2}$, the section is considered cracked;

(b) redistribution of internal stresses and strains due to cracking are included; and



T_b = Base Temperature

T_m = Mean Temperature

T_I = Inner Higher Face Temperature

T_o = Outer Lower Face Temperature

Fig. A.1 : LINEARIZATION OF TEMPERATURE DISTRIBUTION ACROSS WALL THICKNESS

(c) all concurrent loads, as specified in chapters 4 and 5 shall be considered along with thermal loading.

A.3.7 Alternatively, the thermal stress problem may be handled by performing a structural analysis where thermal load is assumed to act on a monolithic section which may be considered to be uncracked. Such an analysis will give conservative results because it does not account for the self-relieving nature of thermal stress due to cracking and deformation.

A.3.8 Thermal stress shall be combined with stress due to other loads to determine design stress. The magnitude of design stress shall not be less than the magnitude of stress due to other loadings alone unless the following are considered:

(a) the effect of cracking in the tensile zone of flexural members on reduction of flexural rigidity and on redistribution of stress;

(b) the reduction of long-term stresses due to creep, and shrinkage; and

(c) stress combinations that reduce the magnitude of stress due to other loads utilizing actual temperatures and temperature distributions which act concurrently with other loads.

APPENDIX-B

ESTIMATION OF CREEP AND SHRINKAGE

B.1 General

B.1.1 The creep strain and shrinkage strain of concrete depends on several aspects and site-specific data. In the absence of site-specific data and detailed studies, the creep strain and shrinkage strain may be predicted as given below. These predictions consider only those parameters that are normally known to the designer. These are valid for ordinary structural concrete (15 MPa < $f_{ck} \leq$ 100 MPa) subjected to a compressive stress of magnitude less than 40% of the mean compressive strength at the age of loading and exposed to mean relative humidities in the range of 40% to 100% at mean temperatures from 5^{o}C to 30^{o}C with seasonal variations between approx 20^{o}C and +40^{o}C.

B.2 Creep Strain

B.2.1 Creep is the increase in strain under sustained stress. Within the range of service compressive stresses of magnitude less than 40% of the mean compressive strength of concrete, creep is assumed to be linearly related to stress. At any age t, the creep strain $_{cc}$ of a concrete member under a constant compressive stress applied at time t_o may be calculated as

$$_{cc} = C_r\ (\ _c\ /\ E_c), \tag{B.1}$$

where t_o is the age of concrete at loading (days), t the age of concrete (days) at the moment considered ($t > t_o$), E_c is the short-term static modulus of elasticity of concrete, C_r is the creep coefficient and $_c$ the compressive stress, applied at time t_o, to which the concrete member under consideration is subjected.

To take the effect of creep in design, the value of modulus of elasticity of concrete may be modified as follows:

$$E_t = \quad E_c\ /\ (1 + C_r), \tag{B.2}$$

where E_t is the modified modulus of elasticity of concrete.

B.2.2 The creep coefficient may be calculated as

$$C_r = C_n \quad _c, \tag{B.3}$$

where C_n is the notional creep coefficient, and $_c$ is the coefficient to describe development of creep with time after loading.

B.2.3 The notional creep coefficient may be estimated as

$$C_n \quad = C_{\mathrm{RH}} \quad (f_{cm}) \quad (t_0), \tag{B.4}$$

with

$$C_{\mathrm{RH}} \quad = 1 + \frac{1 \quad (\mathrm{RH}/100)}{0.46(h/100)^{1/3}}, \tag{B.5}$$

$$(f_{cm}) = \frac{5.3}{\qquad}, \tag{B.6}$$

$$(0.1\ f_{cm})^{0.5}$$

$$(t_0) = \frac{1}{0.1 + (t_0)^{0.2}}, \qquad \text{(B.7)}$$

where,

RH = relative humidity of the ambient environment (%),

h = notional size of member (mm),

= $2A_c/u$, where A_c is the cross-section and u the perimeter of the area of the member in contact with the atmosphere,

f_{cm} = mean compressive strength of concrete at the age of 28 days (MPa)

= $0.8\,f_{ck} +\quad f$, where f_{ck} is the characteristic compressive strength (cube) of concrete at the age of 28 days (MPa) and $f = 8$ MPa.

B.2.4 The development of creep with time is given by

$$_c = \left[\frac{(t\ \ t_0)}{_H + (t\ \ t_0)}\right]^{0.3}, \qquad \text{(B.8)}$$

with,

$$_H = 1.5\ h\{1 + [1.2(\mathrm{RH}/100)]^{18}\} + 250 \leq 1500. \qquad \text{(B.9)}$$

Effect of Type of Cement and Curing Temperature

B.2.5 These effects may be accounted for by modifying the age at loading t_0 as shown below:

$$t_0 = t_{0,T}\ \frac{9}{\quad} + 1 \quad \geq 0.5 \text{ day}, \qquad \text{(B.10)}$$

$$2 + (t_{0,T})^{1.2}$$

with

$$t_{0,T} = \sum_{i=1}^{n} t_i \exp \left[13.65 - \frac{4000}{273+T} \right], \qquad \text{(B.11)}$$

where,

$t_{0,T}$ = temperature adjusted age of concrete at loading (days). This accounts for deviations from a mean concrete temperature of 20°C for the range of approximately 0°C to 80°C (maximum),

= 1 for slow hardening cements,

= 0 for normal or rapid hardening cements,

= 1 for rapid hardening high-strength cements,

t_i = number of days when a temperature T prevails, and

T = temperature (°C) during the time period t_i.

Effect of High Stresses

B.2.6 For stress levels exceeding 40% but within 60% of mean compressive strength of concrete at the age of loading, the nonlinearity of creep may be taken into account as given below:

$$C_{n,k} = C_n \exp[\quad (k \quad 0.4)], \qquad \text{(B.12)}$$

where,

$C_{n,k}$ = Nonlinear notional creep coefficient which replaces C_n in equation (B.3),

k = ratio of stress-to-mean strength at the age t_0, and

= 1.5. For mass concrete and at very high humidities, may be as low as 0.5.

B.3 Shrinkage Strain

B.3.1 The shrinkage strain ${}_{st}$ may be calculated as

$$ {}_{st} = {}_{n}\;{}_{s}, \qquad \text{(B.13)} $$

where ${}_{n}$ is the notional shrinkage coefficient and ${}_{s}$ the coefficient to describe the development of shrinkage with time.

B.3.2 The notional shrinkage coefficient may be estimated as:

$$ {}_{n} = {}_{s}\;{}_{RH}, $$

with

$$ {}_{s} = [160 + 10\;{}_{sc}(9 \quad 0.1\, f_{cm})]\; 10^{\;6}, \qquad \text{(B.14)} $$

where,

f_{cm} = as defined in B.2.3,

${}_{sc}$ = 4 for slowly hardening cements,

= 5 for normal or rapid hardening cements,

= 8 for rapid hardening high-strength cements,

$$ {}_{RH} = \quad 1.55\;{}_{sRH} \text{ for } 40\% \le RH < 99\%, \qquad \text{(B.15a)} $$

$$ {}_{RH} = + 0.25 \text{ for } RH \ge 99\%, \qquad \text{(B.15b)} $$

with

$$ {}_{sRH} = 1 \quad (RH/100)^{3}, \qquad \text{(B.16)} $$

RH is as defined in B.2.3.

B.3.3 The development of shrinkage with time is given by

$$ {}_{s} = \frac{(t \;\; t_s)}{350(h/100)^2+(t \;\; t_s)}^{\,0.5} , \qquad \text{(B.17)}$$

where h is as defined in B.2.3, t is the age of concrete (days), and t_s is the age of concrete (days) at the beginning of shrinkage i.e. curing period (days).

APPENDIX-C

ESTIMATION OF CRACK WIDTH

C.1 One-Way Construction

C.1.1 The crack width w_c due to flexural tension should be calculated using Gergely-Lutz formula.

$$ w_c = 0.076 \; (h_2/h_1) \; f_s \; (d_c \;\; A_{tc})^{1/3} \mathrm{x}10^{\;\,3}, \qquad \text{(C.1)}$$

where w_c is the maximum probable crack width (inches), h_2 is the distance from neutral axis to the tension fibre (inches), h_1 is the distance from neutral axis to the reinforcing steel (inches), f_s is the reinforcing steel stress (ksi), d_c is the thickness of cover from tension fibre to centre of bar closest thereto (inches), $A_{tc} = 2\, d_c\, s$ = area of concrete symmetric with reinforcement divided by number of bars (sq.in.), and s is the spacing of bar (inches).

C.2 Two-Way Construction

C.2.1 There are limitations in correct estimation of crack width for two-way slabs/ walls. However, based on the present state-of-art the following expression may be used,

$$w_c = K \ {}' f_s \ (M_I)^{½},$$ (C.2)

where,

$$M_I = \ {}_x \ s_y / p_x$$ (C.3)

f_s = 40% of design yield strength f_y (ksi),

$_x$ = diameter of bar in direction x to the concrete outer fibres (inches).

s_y = reinforcement spacing in perpendicular direction y,

x = direction for which the crack control check is to be made,

p_x = A_{sx} / A_{tx},

A_{sx} = area of tensile reinforcement per unit width in direction x (sq. inches),

A_{tx} = concrete stress area in direction x

= $12\,(2\ c_x + \ _x)$ (sq. inches), and (C.4)

c_x = clear cover to steel bar in direction x (inches).

C.2.2 The value of coefficient K in equation (C.2) could be taken as 2.8 x 10 ^{5} for fixed slab with uniformly distributed loading. For simply supported slab it is 4.48×10^{5}. For partial fixity, K is taken between 2.8×10^{5} and 4.48×10^{5}, but closer to the lower value.

C.2.3 The value of ' varies from 1.20 to 1.35. To simplify calculations ' may be taken as 1.30.

APPENDIX-D

EFFECTIVE LENGTH OF COLUMNS

D.1 General

D.1.1 The effective height, l_{ef} of a column in a given plane may be obtained from the following equation:

$$l_{ef} = k\, l_c. \qquad \text{(D.1)}$$

Values of k are given in Tables D.1 and D.2 for braced and unbraced columns respectively as a function of end conditions of the column. Formulae of cl. D.3 may be used to obtain a more rigorous assessment of effective length, if desired. It should be noted that the effective height of a column in the two plan directions may be different.

In Tables D.1 and D.2 the end conditions are defined in terms of a scale from 1 to 4. Increase in this scale corresponds to a decrease in end fixity. An appropriate value can be assessed from D.2.1.

D.2 End Conditions

D.2.1 The four end conditions are as follows:

(a) Condition 1: The end of the column is connected monolithically to beams on either side which are at least as deep as the overall dimension of the column in the plane considered. Where the column is connected to a foundation structure, this should be of a form specifically designed to carry moment,

(b) Condition 2: The end of the column is connected monolithically to beams or slabs on either side which are shallower than the overall dimension of the column in the plane considered,

(c) Condition 3: The end of the column is connected to members which, while not specifically designed to provide restraint to rotation of the column will, nevertheless, provide some nominal restraint,

(d) Condition 4: The end of the column is unrestrained against both lateral movement and rotation (e.g. the free end of a cantilever column in an unbraced structure).

TABLE D.1: VALUES OF *k* FOR BRACED COLUMNS

End condition at top	End condition at bottom		
	1	2	3
1	0.75	0.80	0.90
2	0.80	0.85	0.95
3	0.90	0.95	1.00

TABLE D.2 : VALUES OF *k* FOR UNBRACED COLUMNS

End condition at top	End condition at bottom		
	1	2	3
1	1.2	1.3	1.6

2	1.3	1.5	1.8
3	1.6	1.8	—
4	2.2	—	—

D.3 Rigorous Assessment of Effective Column Length

D.3.1 Simplified recommendations are given in D.1.1 for the assessment of effective column heights for common situations. Where a more accurate assessment is desired, the equations given hereinafter may be used.

Braced columns: Effective Height for Framed Structures

D.3.2 The effective height for framed structures may be taken as the lesser of:

$$l_e = l_c\,[0.7 + 0.05\,({}_{c,1} + {}_{c,2})] < l_c, \tag{D.2}$$

$$l_e = l_c\,[0.85 + 0.05\,({}_{c,\min})] < l_c. \tag{D.3}$$

Unbraced columns : Effective Height for Framed Structures

D.3.3 The effective height for framed structures may be taken as the lesser of:

$$l_e = l_c\,[1.0 + 0.15\,({}_{c,1} + {}_{c,2})], \tag{D.4}$$

$$l_e = l_c\,(2.0 + 0.3\;{}_{c,\min}). \tag{D.5}$$

D.3.4 In equations (D.2) to (D.5) following symbols are used.

l_e - effective height of a column in the plane of bending considered,

l_c - clear height between end restraints,

${}_{c,1}$ - ratio of the sum of the column stiffness to the sum of beam stiffness at the lower end of a column,

$_{c,2}$ - ratio of the sum of column stiffness to the sum of beam stiffness at the upper end of a column, and

$_{c,\min}$ - lesser of $_{c,1}$ and $_{c,2}$.

D.3.5 In the calculation of $_{c}$ only members properly framed into the end of the column in the appropriate plane of bending should be considered. For computation of stiffness of members, unsupported length of the members is to be considered.

Relative Stiffness

D.3.6 In specific cases of relative stiffness the following simplifying assumptions may be used:

(a) flat slab construction: the beam stiffness is based on an equivalent beam of the width and thickness of the slab forming the column strip;

(b) simply supported beams framing into a column: $_c$ to be taken as 10;

(c) connection between column and base designed to resist only nominal moment: $_c$ to be taken as 10;

(d) connection between column and base designed to resist column moment: $_c$ to be taken as 1.0.

APPENDIX-E

DETAILING OF REINFORCEMENTS

E.1 General Requirements

E.1.1 (a) Reinforcing steel, conforming to the requirements of AERB/SG/CSE-4, shall be used as reinforcement in a structural member,

(b) Bars may be arranged singly or in pairs in contact or in groups of three or four bars bundled in contact.

E.1.2 All structures shall be detailed for achieving ductility.

Diameter

E.1.3 The diameter of round bar is its nominal diameter, and in the case of bars which are not round or in the case of deformed bars the diameter is taken as the diameter of a circle giving an equivalent effective area. For bundled bars, the equivalent diameter shall be considered.

Minimum Spacing

E.1.4 The minimum clear distance between two parallel main reinforcing bars shall be not less than the greatest of the following:

(a) the diameter of the largest bar if the diameters are unequal; and

(b) 5 mm more than the nominal maximum size of coarse aggregate.

The size of aggregates may be reduced around congested reinforcement to comply with this provision.

E.1.5 When needle vibrators are used the horizontal distance between groups of bars may be reduced to two-thirds the nominal maximum size of coarse aggregate, provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

E.1.6 Where parallel reinforcement is placed in two or more layers, bars in upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 25 mm, nor less than the space, given in cl. E.1.4.

E.1.7 (a) Clear distance limitation between bars shall also apply to the clear distance between a contact lap splice and adjacent splices of bars,

(b) Clear distance between pretensioning tendons at each end of a member shall be not less than 4 times the wire diameter or 3 times the strand diameter whichever is applicable.

Maximum Distance Between Bars

E.1.8 (a) For bars in beams refer section E.2,

(b) For bars in slab, shells and folded plate refer section E.3,

(c) For bars in columns and walls refer section E.4,

(d) For bars in foundation refer section E.5.

Clear Cover

E.1.9 The minimum requirements for clear concrete cover to reinforcement is given in cl. E.1.11 to E.1.13 for cast-in-situ concrete (non-prestressed), precast concrete, and prestressed concrete respectively. Increased cover thicknesses are required when surfaces of concrete members are exposed to the action of harmful chemicals (as in the case of concrete in contact with earth faces contaminated with such chemicals), acid vapour, saline atmosphere, sulphurous smoke, etc. Clear concrete cover to reinforcement shall not exceed 80 mm.

E.1.9.1 For the purpose of cover thickness, the following environmental conditions (exposure type) are considered:

(a) Normal: ordinary conditions both outdoor and indoor,

(b) Corrosive:

(i) As compared to normal exposure, reinforcement subject to severe weather, detrimental influences such as alternate wetting and drying, chemicals, etc.

(ii) Structures exposed to soil or rock and structures below underground water containing harmful substances.

(iii) Structures submerged in sea water and structures exposed to mild marine environment, etc.

(c) Severe Corrosive Environment:

(i) Reinforcement subjected to considerable detrimental influence.

(ii) Structures exposed to tides, splashes, severe, ocean winds, etc.

Cast-in-Situ Concrete (Non-prestressed)

E.1.10 The minimum concrete cover shall be provided in accordance with Table E.1 for reinforcement. In no case cover shall be less than the largest diameter of bars used.

Precast Concrete

E.1.11 The minimum concrete cover should be provided in accordance with Table E.2 for reinforcement. In no case the cover shall be less than diameters of bars.

Prestressed Concrete

E.1.12 (a) The following minimum concrete cover shall be provided in accordance with Table E.3 for prestressed reinforcement, ducts and end fittings.

(b) For prestressed concrete members exposed to earth, weather or corrosive environments, and in which permissible tensile stress of cl. 6.3.3 is exceeded, minimum cover shall be increased by 50%.

(c) For prestressed concrete members manufactured under plant condition, minimum concrete cover for non-prestressed reinforcement shall be as required in E.1.11.

TABLE E.1 : MINIMUM CONCRETE COVER FOR NON-PRESTRESSED CAST-IN-SITU CONCRETE

Member Type	Minimum Cover (mm) Normal Environment	Corrosive Environment	Severe Corrosive Environment
Slabs, walls, and joints.	25 or diameter of bar whichever is larger.	50	75
Beams and columns			
- Main reinforcement	40	50	75
- Ties/stirrup/spiral	25	30	40
Shells and folded plates	15 or [1] whichever is greater.	40	—

Note: (1) is diameter of bars

(2) For concrete cast against soil/rock, the covers specified for severe corrosive environment

should be adopted.

Clear Cover Requirement for Fire Protection

E.1.13 The member sizes and reinforcement covers that shall be designed to provide for fire resistance for different fire rating is given in Table E.4. When cover is more than 40 mm, wire mesh shall be used for protection against spalling. IRC fabric as per IS:498 is suggested. Typical wire mesh which could be used is 100 x 100 x 2 mm dia.

TABLE E.2 : MINIMUM CONCRETE COVER FOR PRECAST CONCRETE MEMBERS

Member type	Exposure type	Minimum Cover (mm) (for different diameter of bar)			
		10 and smaller	12-16	32 and smaller	40 and above
Slab and joints	Normal	—	15	15	30
	Corrosive	—	30	40	50
Wall panels	Normal	—	15	15	30
	Corrosive	—	20	30	40
Beams and columns - Main reinf.	Normal	—	35	35	35
	Corrosive	—	45	45	45
Beams and columns - Ties/stirrup/ spiral	Normal	—	15	15	15
	Corrosive	—	20	20	20

Shells and folded plate	Normal	10	15		
	Corrosive	—	35	45	45 or whichever is greater

E.1.14 In case of beam, when the width is greater than minimum value as given in Table E.4, the minimum cover of reinforcement may be adjusted by applying the correction given in Table E.5. In no case the resulting cover shall be less than specified for slab.

TABLE E.3 : MINIMUM COVER FOR PRESTRESSED CONCRETE MEMBERS

Member Type	Minimum Cover (mm) Normal Environment	Corrosive Environment	Severe Corrosive Environment
Slabs, walls, and joints.	20	30	75
Beams and columns			
- Main reinf.	40	40	75
- Ties/stirrup/spiral	25	25	40
Shells and folded plates	15 for bar dia. up to 12 mm. 20 for bar dia above 12 mm.	40	75

E.2 Reinforcement Requirements for Beams

Flexure Reinforcement

E.2.1 Minimum Reinforcement

(a) At any section of a flexural member, except as provided in (b) below where positive reinforcement is required by analysis, the ratio p provided shall not be less than that given by

$$p_{\min} = 1.4 / f_y \tag{E.1}$$

In T-beams and joints where the web is in tension, p shall be computed for this purpose using the width of web,

(b) Alternatively, the area of reinforcement provided at every section, positive or negative shall be atleast one-third greater than that required by analysis,

(c) The minimum area of compression reinforcement shall be 0.2% of the total area of the web, including the portion in the slab or (1/3)rd of the tensile reinforcement (required by analysis at that section) whichever is greater.

TABLE E.4 : FIRE PROTECTION REQUIREMENTS OF R.C. MEMBERS

Element	**Parameter**	**Dimension and corresponding minimum cover (mm) for Resistance Period**			
		1 hour	**2 hour**	**3 hour**	**4 hour**
Column	Side	200	300	400	450
	Cover	25	35	35	40
Load-bearing wall	Thickness	125	150	200	250
	Cover	25	25	25	25
Slab	Thickness	100	125	150	175

	Cover	20	25	35	55
Beam	Width	125	200	250	275
	Cover	30	60	70	80

Note: Covers indicated relate to main reinforcement bars.

E.2.2 Maximum Tension Reinforcement

The area of tension reinforcement shall not exceed 0.04 $b_w h$, in which b_w is the width of the web for flanged beams and the beam width for rectangular beams.

TABLE E.5 : VARIATION OF COVER TO MAIN REINFORCEMENT WITH MEMBER WIDTH

Minimum increase in width (mm)	Decrease in cover (mm)
25	5
50	10
100	15
150	15

E.2.3 Maximum Compression Reinforcement

The maximum area of compression reinforcement shall not exceed 0.04 $b_w h$. The compression reinforcement shall be enclosed by stirrups for effective lateral restraint. The stirrup shall meet the requirements for both ties in columns and stirrups in beams.

E.2.4 Maximum Spacing of Tension Reinforcement

The distance between parallel reinforcement bars or groups near the tension face of a beam shall not be greater than 300 mm for f_y = 250 and 200 mm for f_y = 415 grade steel.

E.2.5 Side Face Reinforcement

(a) When the depth of web of the beam exceeds 750 mm, side face reinforcement shall be provided (to control cracking) along the two faces. Such longitudinal bars on each face of the web shall not be less than 0.001 times the gross area of the web,

(b) The spacing of bars should neither exceed 250 mm nor the horizontal distance between the side face reinforcing bars,

(c) The reinforcement should be provided at each face between the lower most layer of top reinforcement or soffit of slab (in case of flanged beam) and the upper most layer of bottom reinforcement.

Shear Reinforcement

E.2.6 (a) Shear reinforcement in beams should be taken around the outermost tension and compression bars. In T-beams, L-beams and I-beams, such reinforcement should pass around the longitudinal bars located close to the outer face of the flange,

(b) Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports consists of closed ties, closed stirrups or spirals extending around the flexural reinforcement,

(c) Closed ties or stirrups may be formed in one piece by overlapping standard stirrups or ties and hooks around a longitudinal bar, or formed in one or two pieces lap-spliced with a splice lap of l.3 L, in which L is the development length.

E.2.7 Minimum Shear Reinforcement

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$(A_{sv} / bs_v) \geq (0.4 / 0.87 f_y),$$

where $f_y \leq 415$ N/mm^2, A_{sv} is the total cross-sectional area of stirrup legs effective in shear, s_v is the stirrup spacing along the length of the member, and

b the breadth of the beam or breadth of the web of flanged beam.

E.2.8 Maximum Spacing of Shear Reinforcement

(a) Spacing of shear reinforcement placed perpendicular to the axis of member shall not exceed 0.5 d or 300 mm whichever is lower,

(b) When V_s, the nominal shear strength provided by shear reinforcement, exceeds 0.35 $b_w d\,(f_{ck})^{½}$, the maximum spacing, given in the preceeding, shall be reduced by 50%,

(c) The following requirement of stirrup spacing is applicable up to a distance equal to twice the member depth from the face of support towards mid-span:

(i) The first stirrup shall be located not more than 500 mm from the face of supporting member,

(ii) The maximum spacing of stirrup shall not exceed the minimum of the following:

(a) 0.25 d,

(b) 8 times the diameter of the smallest longitudinal bar,

(c) 24 times the diameter of stirrup, and

(d) 300 mm.

E.2.9 Hoops in flexural members are allowed to be made up of two pieces of reinforcement: a U-stirrup having hooks not less than 135° with six diameter (but not less than 75 mm) extension anchored in the confined core and a cross tie to make a closed hoop. Consecutive cross ties engaging the same longitudinal bar shall have their 90° hooks at opposite sides of flexural member. If the longitudinal reinforcing bars secured by the cross ties are confined by a slab only on one side of the flexural frame member, the 90° hooks of the cross ties shall all be placed on that side.

Torsion Reinforcement

E.2.10 When a member is designed for torsion, both transverse and longitudinal torsion reinforcement shall be provided to meet the following requirements. The longitudinal reinforcement, when placed together with tension and compression reinforcement, shall be in addition to tension and compression reinforcement, as will be required for forces other than torsion. The transverse reinforcement shall be in addition to stirrups, ties or spirals, required for transverse shear forces.

E.2.11 The transverse reinforcement for torsion shall be rectangular or circular closed stirrups placed along the axis of member. The spacing of stirrups shall not exceed the least of x_1, $(x_1 + y_1)/4$, and 300 mm where x_1 and y_1 are respectively the short and long dimensions of the stirrup. The stirrups shall also meet the requirements provided in cl E.2.8 and E.2.9.

E.2.12 Torsion reinforcement shall be provided at least a distance $(b + d)$ beyond the point theoretically required, in which b is the width of that part of member cross-section containing the closed stirrups resisting torsion.

E.2.13 Longitudinal reinforcement should be placed as close as is practicable to the corners of cross-section and in all cases, these shall be at least one longitudinal bar in each corner of the ties. When the cross-sectional dimension of the member exceeds 450 mm, additional longitudinal bars shall be provided to satisfy the requirements of minimum reinforcement (cl. E.2.1) and spacing given in cl. E.2.5 and shall not exceed 300 mm.

E.3 Reinforcement Requirements of Slabs, Plates and Shells

Minimum Reinforcement

E.3.1 All exposed concrete surfaces shall be provided with reinforcement placed in two approximately perpendicular directions. Concrete surfaces shall be considered to be exposed if they are not cast against existing concrete or against rock. The reinforcement shall be developed for its specified yield strength. The minimum area of such reinforcement shall be in accordance with requirements given hereinafter. This requirement may be met in total or in part by reinforcement otherwise required to resist design loads. Reinforcement shall be spaced not farther apart than 450 mm or specified otherwise.

E.3.2 For concrete sections less than 1200 mm thick such reinforcement shall provide at least a ratio of area of reinforcement to gross concrete area of 0.0012 in each direction at each face.

E.3.3 For concrete sections having a thickness of 1200 mm or more, such reinforcement shall provide an area A, in each direction at each face given by

$$A_{smin} = [0.7(f_{ck})^{½} / f_s]\ A, \text{ but need not exceed } A/100.$$

The minimum reinforcement size shall be 20 mm. In lieu of computation, f_s may be taken as 60% of yield strength f_y.

E.3.4 For concrete sections having a thickness of 1800 mm or more, no minimum reinforcement is required for members which are constructed by the principles and practice for non-reinforced massive concrete structures.

E.3.5 On a tension face of a structural slab, plate or shell, where a calculated reinforcement requirement exists, the ratio of reinforcement area provided at the tension face to gross concrete area shall be not less than 0.0018 units unless the reinforcement area provided at the tension face is at least one-third greater than that required by analysis. All exposed faces of structural slab, wall or shell shall be reinforced to meet the minimum requirements of sections E.3.1, E.3.2, E.3.3.

Minimum Distribution Reinforcement

E.3.6 In addition to satisfying the requirements in cl. E.3.1, the distribution reinforcement per unit width of slab shall also be not less than one-quarter of the main reinforcement in the perpendicular direction.

E.3.7 Minimum Reinforcement for Liquid Retaining Structures

(a) The amount of shrinkage and temperature reinforcement to be provided is dependent on the distance between joints that will dissipate shrinkage and temperature stress in the direction of reinforcement. The shrinkage and temperature reinforcement should not be less than the coefficient given in Fig. E.1,

(b) Concrete section having thickness more than 600 mm may have minimum reinforcement at each face based on 300 mm thickness,

(c) The spacing of reinforcement should not be more than 300 and they are divided equally between two surfaces.

E.3.8 Maximum Size of Reinforcement

The diameter of reinforcing bars shall not exceed one-eighth of the total slab thickness.

E.3.9 Spacing of Reinforcement

The spacing of main reinforcement shall not exceed 2 *h* or 300 mm, whichever is smaller.

The spacing of the distribution reinforcement in one-way slabs shall not exceed 3*d* or 400 mm whichever is smaller.

E.4 Reinforcement Requirements of Columns

Longitudinal Reinforcement

E.4.1 The cross-sectional area of longitudinal reinforcement shall neither be less than 1% nor more than 5% of the gross cross-sectional area of the scolumn. However, in the zone where bars are lapped the percentage of reinforcement should not be more than 8% of the gross area.

E.4.2 In the case of pedestals in which longitudinal reinforcement is not taken into account in strength calculations, nominal longitudinal reinforcement not less than 0.15% of the cross-sectional area shall be provided.

E.4.3 (a) The minimum number of longitudinal bars provided in a column shall be four in rectangular columns and six in circular columns,

(b) A reinforced concrete column having helical reinforcement shall have at least six bars of longitudinal reinforcement within the helical reinforcement,

(c) The longitudinal bars shall be not less than 12 mm in diameter.

E.4.4 Clear spacing of longitudinal bars of the column shall not exceed 10 times the dia of bar or 250 mm whichever is greater. It shall also not be less than 3 times the dia of bar or 50 mm whichever is higher.

Transverse Reinforcement (Ties)

E.4.5 General Requirements

A reinforced concrete compression member shall have transverse or helical reinforcement so disposed that every longitudinal bar nearest to the compression face has effective lateral support against buckling subject to provisions in E.4.6 below. The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension or by polygonal links (lateral ties) with internal angles not exceeding 135^{o}. The ends of transverse reinforcement shall be properly anchored.

E.4.6 Arrangement of Transverse Reinforcement (Ref. Fig. E.1)

(a) If longitudinal bars are not spaced more than 75 mm on either side, transverse reinforcement need only to go round the corner and alternate bars for providing effective lateral supports,

(b) If longitudinal bars spaced at a distance of not exceeding 48 times the tie diameter, are effectively tied in two directions, additional

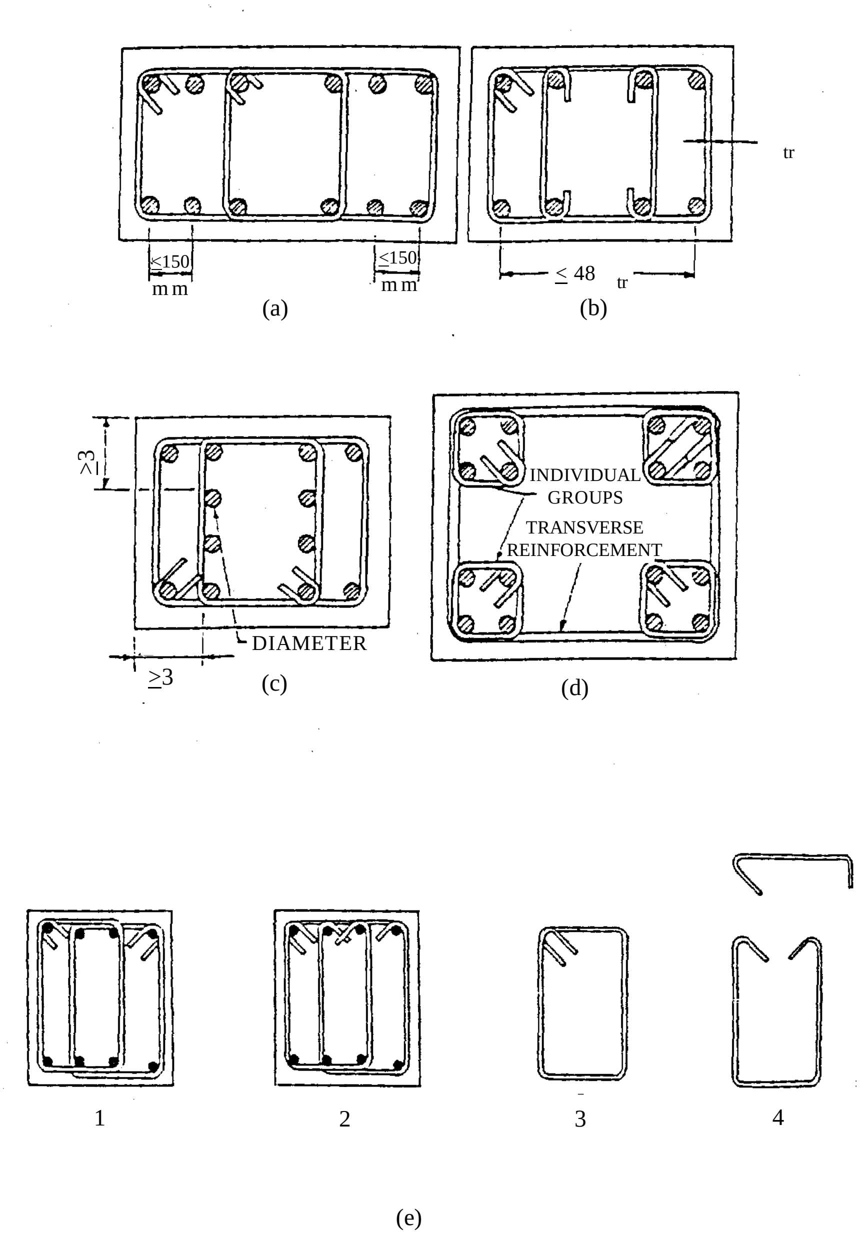


Fig. E.1 : ARRANGEMENT OF TRANSVERSE REINFORCEMENT IN COLUMNS

longitudinal bars between these bars need to be tied in one direction by open ties,

(c) Where longitudinal reinforcing bars in a compression member are placed in more than one row, the effective lateral support to longitudinal bars in the inner rows may be assumed to have been provided if:

  (i) transverse reinforcement is provided for the outermost row in accordance with this clause, and if

  (ii) no bar of the inner row is closer to the nearest compression face than three times the diameter of the largest bar in the inner row,

(d) Where longitudinal bars in a compression member are grouped (not in contact) and each group adequately tied with transverse reinforcement in accordance with this clause, the transverse reinforcement for compression member as a whole may be provided assuming that each group is a single longitudinal bar for determining the pitch and diameter of the transverse reinforcement in accordance with this clause. The diameter of such transverse reinforcement need not, however, exceed 20 mm.

E.4.7 Pitch and Diameter of Lateral Ties

(a) Pitch: The pitch of transverse reinforcement shall be not more than the least of the following distances:

  (i) the least lateral dimension of the compression member; in case of composite section (refer chapter-8), half the least side dimension of the composite member,

  (ii) sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied,

  (iii) forty eight times the diameter of the transverse reinforcement and

  (iv) for one-sixth length or twice the larger lateral dimension or 300 mm, whichever is greater, at each end of columns, the spacing shall be halved, but it shall not exceed 100 mm.

(b) Diameter:

  (i) The diameter of polygonal links or lateral ties shall be not less than one-fourth the diameter of the largest longitudinal bar, and in no case less than 8 mm,

(ii) In case of composite section (ref. chapter-8) the lateral ties shall have a diameter not less than 1/50 times the greatest side dimension of composite member, except that ties shall not be smaller than 8 mm diameter bar, and need not be larger than 16 mm. Welded wire fabric of equivalent area may be used.

E.4.8 Helical or Spiral Reinforcement

(a) Pitch: Helical or spiral reinforcement shall be of regular formation with the turns of helix spaced evenly and its ends anchored properly by providing one and a half extra turns of the spiral bar. Where an increased load on the column with helical reinforcement is allowed for, the pitch of helical turns shall be not more than 75 mm, nor more than one-sixth of the core diameter of the column, nor less than 25 mm, nor less than three times the diameter of the steel bar forming the helix. In other cases, the requirements of E.4.5 and E.4.6 shall be complied with.

(b) Diameter: The diameter of helical reinforcement shall be in accordance with cl. E.4.7.

Offset at Splice

E.4.9 In columns where longitudinal bars are offset at a splice, the slope of inclined portion of the bar with column axis shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the column axis. Adequate horizontal support at the offset bends shall be treated as a matter of design, and shall be provided by metal ties, spirals, or parts of the floor construction. Metal ties or spirals so designed shall be placed near (not more than eight-bar diameters from) the point of bend. The horizontal thrust to be resisted shall be assumed as 1.5 times the horizontal components of the nominal stress in the inclined portion of the bar. Offset bars shall be bent before being placed in the forms. Where column faces are offset 75 mm or more, splices of vertical bars adjacent to offset face shall be made by separate dowels overlapped as required.

E.4.10 At beam column connections the ties of column shall be continued.

E.5 Reinforcement Requirements of Walls

E.5.1 The requirement of minimum reinforcement of cl. E.3.1 to E.3.5 should be satisfied in addition to the requirements given hereinafter.

E.5.2 Vertical Reinforcement

(a) In the case of load-bearing walls the minimum reinforcement in vertical direction shall be 0.4% of the gross cross-sectional area of the concrete on any unit length, and shall be equally divided between two faces of the wall,

(b) The maximum area of vertical reinforcement shall not exceed 4% of the gross cross-sectional area of the concrete in a metre length,

(c) Vertical reinforcement need not be enclosed by lateral ties or where vertical reinforcement is not required as compression reinforcement, if vertical reinforcement area is not greater than 1.0% of the gross concrete area. When vertical reinforcement resisting compression exceeds this value, links at least 6 mm or quarter the size of the largest compression bar should be provided through the thickness of the wall. The spacing of these links should not exceed twice the wall thickness in either the horizontal or vertical directions and in the vertical direction should not be greater than 16 times the diameter of bars. Any vertical compression bar not enclosed by a link should be within 200 mm of a restrained bar.

E.5.3 Horizontal Reinforcement

(a) The minimum area of horizontal reinforcement, when expressed as a percentage of gross cross-sectional area, shall be 0.25% and this shall be equally divided between the two faces,

(b) On the face and in the direction of tension, the minimum percentage of reinforcement shall be 0.18.

E.5.4 The maximum spacing of vertical and horizontal bars shall not exceed 300 mm.

Reinforcement Around Opening

E.5.5 In addition to the minimum reinforcement in cl. E.5.2and E.5.3, a minimum of two numbers of 16 mm dia bars or equivalent shall be provided around all openings.

E.6 Footings and Pile Caps

E.6.1 The requirement of minimum reinforcements given in cl. E.3.1 to E.3.5 is also applicable for pile caps,

E.6.2 (a) The spacing between reinforcement centres shall not exceed 200 mm for Fe415, and 300 mm for Fe250 grade reinforcement,

(b) In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across the entire width of the footing,

(c) In two-way rectangular footings, reinforcement shall be distributed as follows:

(i) Reinforcement in long direction shall be distributed uniformly across the entire width of footing,

(ii) For reinforcement in short direction, a portion of the total reinforcement given by Equation (E.2) shall be distributed uniformly over a band width (centred on the centre line of column or pedestal) equal to the length of short side of footing. The remainder of reinforcement required in short direction shall be distributed uniformly outside the centre band width of footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in shorter direction}} = \frac{2}{\ +1} \qquad \text{(E.2)}$$

in which is the ratio of the long side to the short side of the footing.

E.6. In pile caps, fully lapped, circumferential horizontal reinforcement consisting of bars not less than 12 mm diameter at a spacing not exceeding 250 mm, shall be provided.

E.6.4 (a) All reinforcement in rectangular isolated, combined and strip footings shall extend the full width of the footing;

(b) All reinforcement shall be fully developed beyond the critical section for its yield strength.

E.7 Curtailment of Bars

E.7.1 (a) Except at end supports of simple spans or free ends of cantilevers every bar shall extend beyond the point at which it is no longer needed, for a distance at least equal to the greater of:

(i) the effective depth of the member; or

(ii) 12 times the bar diameter.

(b) In addition, for a bar in the tension zone, one of the following requirements for all arrangements of the design limit state loads should be complied with:

(i) Shear at the cut-off point does not exceed two thirds that permitted, including shear strength of shear reinforcement provided,

(ii) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall not be less than $0.4b_w s/f_y$. Spacing s shall not exceed d/8 ${}_b$ where ${}_b$ is the ratio of area of reinforcement cut-off to total area of tension reinforcement at the section,

(iii) For 36 and smaller bars, continuing reinforcement provides double the area required for flexure at the cut-off point and shear does not exceed three-fourths that permitted.

Development of Positive Moment Reinforcement

E.7.2 (a) Atleast 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 to be a length l_d/3.0. In beams, such reinforcement shall extend into the support at least 150 mm,

(b) When a flexural member is part of a primary lateral load resisting system, the positive moment reinforcement required to be extended into the support as stated above, be anchored to develop the design strength i.e. f_y/f_s in tension at the face of support,

(c) At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that development length l_d computed for f_y satisfies Eq. (E.3); except, Eq. (E.3) need not be satisfied for reinforcement terminating beyond the c e n t e r l i n e of simple supports by a standard hook, or a mechanical anchorage at least equivalent to a standard hook.

$$l_d \leq (M_n / V_u) + l_a, \qquad \text{(E.3)}$$

where M_n is the nominal moment strength assuming all reinforcement at the section to be stressed to the design strength i.e. f_y/s, V_u is factored shear force at the section, and l_a at the support shall be the embedment length beyond the centre of support. The value of M_n/V_u may be increased by 30% when the ends of reinforcement are confined by a compressive reaction.

APPENDIX-F

REQUIREMENTS FOR CONSTRUCTION AND EXPANSION JOINTS, EMBEDDED PIPES AND PARTS

F.1 Construction Joints

F.1.1 Concreting should be carried out continuously up to the construction joints. Straight vertical joints should be avoided as far as practicable. Provision shall be made for transfer of shear and other forces through construction joints. The position of construction joints shall be indicated on the design drawings. Otherwise they should be permitted only at locations approved by the authorised engineer. The construction joints should be avoided at critical locations of moments and shear.

F.1.2 Construction joint should not be provided just below the soffit of a slab or beam. Such joints should be located at a depth at least equal to the smaller lateral dimension of the supporting column or at least equal to half the thickness of the supporting wall, but not less than 250 mm.

F.1.3 Construction joints, when essential in floors, may be located at a distance of (1/3) to (1/5) times the span from support unless a beam intersects a girder at the same location. In such a case joints should be offset at a distance equal to twice the beam width.

F.1.4 (a) Beams, girders or slabs supported by columns or walls should not be cast or erected within 48 hours of concreting the vertical support members,

(b) Beams, girders, column capitals and haunches should be considered as part of a slab system and should be cast simultaneously.

F.1.5 At the construction joint, the interface for shear transfer should be thoroughly cleaned and laitance removed. The value of coefficient of friction to be considered in case of design for shear friction is given in cl. 3.6.17. Even if full shear transfer is not required, all construction joints should be roughened, thoroughly wetted and standing water removed immediately before new concrete is placed.

F.1.6 In cases where concrete has not fully hardened, all laitance should be removed by scrubbing the wet surface with wire or bristle brushes, care being taken to avoid dislodgment of particles of aggregate. The surface should be wetted and all free water removed. The surface should then be coated with neat cement slurry. On this surface, a layer of concrete not exceeding 150 mm in thickness is first placed and is well rammed against old work, particular attention being placed to corners; work thereafter should proceed in the normal manner.

F.2 Movement Joints

F.2.1 (a) Location and nature of movement joints, when provided, shall be indicated on design drawings,

(b) No movement joint shall be provided in concrete structures containing liquids, except that in the case of pipes, such joints may be provided at their extremities,

(c) Movement joints may be provided at locations where abrupt changes in magnitude and direction of plan dimensions, height of structure and type of foundation take place,

(d) When provided, movement joints shall permit structures on both sides to move with a minimum of resistance.

F.2.2 (a) Structures adjacent to the joint should be supported on separate columns or walls but not necessarily on separate foundations if not required by other considerations. The structure should preferably be framed on both sides of the joint. The movement joint should pass through the whole structure above ground level,

(b) Reinforcement should not extend across the movement joint and the break between sections in the structure above foundation level should be complete,

(c) It should be ensured that movement joints are incorporated in finishes and in the cladding at the joint locations.

F.3 Embedded Conduits and Pipes

General

F.3.1 (a) Conduits, pipes and sleeves of any material not harmful to concrete may be embedded in concrete, provided they are not considered to replace structurally the displaced concrete,

(b) Conduits and pipes of aluminium shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminium-concrete reaction or electrolytic action between aluminium and steel,

(c) Conduits, pipes, and sleeves passing through a slab, wall or beam shall not impair significantly the strength of the construction,

(d) Conduits and pipes, with their fittings, embedded within a column shall not displace more than 4% of the area of cross-section on which strength is calculated or which is required for fire protection.

F.3.2 Except when plans for conduits and pipes are approved by the authorised engineer, conduits and pipes embedded within a slab, wall or beam (other than those merely passing through) shall satisfy the following, unless otherwise engineered properly.

(a) not larger in outside dimension than 1/3 times the overall thickness of slab, wall or beam in which they are embedded,

(b) not be spaced closer than 3 diameters or widths on centre, and

(c) not resulting in impairment of the strength of construction by more than 5%. Conduits, pipes and sleeves may be considered as replacing structurally in compression the displaced concrete provided they are not exposed to rusting or undergo other deterioration.

Pipe Containing Fluids

F.3.3 Pipes that contain liquid, gas or vapour not exceeding temperature 65°C and under pressure not more than 1.5 N/sq.mm may be embedded in structural concrete under the following conditions:

(a) Pipes and fittings shall be designed to resist the effects of contained material, pressure and temperature to which they will be subjected,

(b) All piping and fittings except as provided in (c) below shall be tested as a unit for leaks before concrete placement. Pressure tests shall be in accordance with applicable piping code or standard. Where pressure testing requirements are not specified in a code or standard, pressure testing shall meet the following requirements:

(i) The testing pressure shall be 50% in excess of pressure to which piping and fittings may be subjected, but minimum testing pressure shall be not less than 1.0 N/sq.mm above atmospheric pressure,

(ii) The pressure test shall be held for 4 hours with no drop in pressure except that which may be caused by air temperature,

(c) Drain pipes and other piping designed for pressures of not more than 0.1 N/sq.mm need not be tested as required in (b) above,

(d) Pipes carrying liquid, gas or vapour that is explosive or injurious to health shall again be tested as specified in (b) after concrete has hardened,

(e) No liquid, gas or vapour, except water not exceeding 32^{o}C nor 0.35 N/sq. mm pressure, shall be placed in pipes until the concrete will have attained a compressive strength in excess of 7.0 N/sq. mm,

(f) In solid slabs, the piping, unless it is for radiant heating or snow melting, shall be placed between top and bottom reinforcement,

(g) Concrete cover for pipes and fittings shall be not less than 40 mm for concrete exposed to earth or weather, or 20 mm for concrete not exposed to weather or in contact with ground,

(h) Reinforcement with an area not less than 0.002 times the area of concrete section shall be provided normal to the piping,

(i) Piping and fittings shall be assembled according to construction specifications. Screw connections shall be prohibited. Piping shall be so fabricated and installed that it will not require any cutting, bending, or displacement of the reinforcement from its proper location unless otherwise specified by the authorised engineer.

Pipes under High Temperature and Pressure

F.3.4 All piping containing liquid, gas or vapour pressure in excess of 1.5 N/sq. mm or temperature in excess of 65^{o}C shall be sleeved, insulated, or otherwise separated from the concrete and/or cooled to limit concrete stresses to design allowables, and to limit concrete temperatures to the following:

(a) For normal operation or any other long-term period, the temperatures shall not exceed 65^{o}C except for local areas which are allowed to have increased temperatures not to exceed 90°C,

(b) For accident or any other short-term period, the temperatures shall not exceed 175°C for interior surface. However, local areas are allowed to reach 340°C from fluid jets in the event of a pipe failure,

(c) Temperatures higher than those given in (a) and (b) above may be allowed in the concrete if tests are provided to evaluate the reduction in strength and this reduction is applied to design allowables. Evidence, which verifies that the increased temperatures do not cause deterioration of concrete either with or without load, shall also be provided.

F.4 Embedded Parts

F.4.1 Design, fabrication and erection of embedded parts shall be in accordance with AERB safety standard on Design, Fabrication and Erection of Embedded Parts and Penetrations Important to Safety of Nuclear Facilities, (AERB/SS/CSE-4).

APPENDIX-G

TEST FOR LIQUID-RETAINING STRUCTURES

G.1 General

G.1.1 The testing procedure, acceptance criteria and other related aspects furnished in this Appendix apply for liquid-retaining structures only.

G.1.2 The technical specification shall be developed before undertaking the testing. The technical specification shall contain the objective of test, detailed test procedure, acceptance criteria, etc. which shall be formulated in line with the safety function of structures.

G.2 Test Procedure

G.2.2 Leak tightness test shall be carried out; after initial grouting work, raft water-proofing, and prior to water-proofing treatment on the external surface of walls (if any) and metal lining on the inside surface.

G.2.1 For site, where water table is high and the spent fuel/tank system is not designed on the concept of tank inside tank, test for ingress of ground water should be carried out.

G.2.3 Test for ingress of ground water will be conducted first by allowing water table to rise slowly to a stable level by stopping dewatering system. All leakage spots and wet surfaces on the inner face of the wall and raft surfaces will be marked accurately. Water table outside the pool will be lowered by starting dewatering system and corrective treatment such as various types of injection grouting, surface coatings, repairs to the concrete etc. carried out. After satisfactory completion of the corrective measures, water level outside the pool will be allowed to rise again. Corrective treatment can be undertaken in presence of water, if performance of repair material in such condition is established. For grouts made of cement or similar material which cannot perform in presence of water, corrective treatment should be taken up only after emptying.

G.2.4 If any wet surfaces persist the procedure will be repeated until the acceptance criteria is satisfied.

Test for Egress of Stored Water

G.2.5 The test will be conducted by filling the storage pool with water. The filling will be in stages to properly identify the leakage spots and leakage paths if any. All wet patches and leak spots on external surfaces will be properly demarcated. After emptying the pool, corrective treatments such as various types of injection grouting, surface coating, repairs to concrete etc. shall be completed. Corrective treatment can be undertaken in presence of water, if performance of repair material in such conditions is established. For grouts made of cement or similar material which cannot perform in presence of water, corrective treatment should be taken up only after emptying. The pool will be filled again and external surfaces observed. In case wet patches appear, the above procedure will be repeated till the acceptance criteria is satisfied.

G.2.6 After satisfactory repairs, the pool will be filled with water. A period of seven days will be allowed for absorption of water and saturation of concrete. The exposed water surface of pool, may be protected by covering it with polythene sheets or suitable thin oil film to minimise evaporation losses. After initial fill of 7 days, the water level shall be recorded every 24 hours for a further 7 days. Water loss will be to the extent of losses accountable due to evaporation only since the process of absorption by concrete would be completed by then.

G.2.7 After successful completion of testing for acceptance as detailed above, the pool shall be emptied and water proof treatment will be carried out on the exterior faces of walls and metal lining will be provided on the inside surface. Backfilling around the storage bay will also be allowed only after satisfactory testing of the pool.

G.3 Acceptance Criteria

G.3.1 The test aims at achieving nearly zero leakage giving due allowance for evaporation and absorption losses as specified.

G.3.2 For water retaining structures following tank-in-tank concepts, the procedure of filling the tank and repairing shall be repeated for leak tightness test of stored water until total area of wet patches is less than 0.1% of surface area of each wall. Even in case where acceptance criteria is met, large single patch or multiple patches in a localized area should be further repaired.

G.3.3 For other water retaining structures 'The procedure of filling the tank and repairing shall be repeated both for test for ingress of ground water (where required) and leak tightness test for stored water until no wet patches appear on the surface opposite to liquid facing'.

G.3.4 A drop in water level at the rate of 6 mm or 12 mm for 24 hours may be allowed as evaporation losses for covered or open pool condition respectively as the case may be.

G.4 Test Report

G.4.1 The test report shall contain at least the following information:

(a) description of test set-up;

(b) total number of tests undertaken;

(c) time spent on one test:

(i) time spent after filling water;

(ii) time required for repairing, and

(iii) time spent after repairing and before filling of water;

(d) the time gap between starting of two consecutive tests;

(e) repairing:
- (i) description of repairing work, and
- (ii) whether repairing work was undertaken after complete drying of pool walls following the test;

(f) whether walls were completely dried after a test and prior to next test;

(g) environmental conditions (for each day): temperature, humidity and wind velocity;

(h) results:
- (i) drop in water every day (24 hours) for each test.

G.4.2 Test results shall be properly analysed and evaluated to confirm that acceptance criteria have been satisfied.

APPENDIX-H

STRENGTH EVALUATION OF EXISTING STRUCTURES

H.1 General

H.1.1 If any doubt arises concerning the safety of a structure or member, the authorised engineer shall order a structural strength investigation by analysis or by load tests, or by a combination of both.

H.2 Analytical Investigations

H.2.1 If strength evaluation is by analysis, a thorough field investigation shall be made of dimensions and details of members, properties of materials and other pertinent conditions of the structure as actually built.

H.3 Load Tests

H.3.1 If strength evaluation is by load tests, the authorised engineer shall control and supervise such tests.

H.3.2 Load test shall not be made until that portion of the structure to be subject to load is at least 56 days old, or unless agreed upon for an earlier age by AERB.

H.3.3 When only a portion of the structure is to be load-tested, the portion in question shall be load-tested such as to adequately test the suspected source of weakness.

H.3.4 Forty-eight hours before application of test load, a load to simulate effect of that portion of dead loads not already acting shall be applied and shall remain in place until all testing has been completed.

Load Tests of Flexural Members

H.3.5 When flexural members, including beams and slabs, are load-tested, the additional provisions herein shall apply as follows:

(a) Base readings (datum for deflection measurements) shall be made immediately before test load is applied,

(b) That portion of the structure selected for loading shall be subject to a total load, including dead loads already acting, equivalent to 0.85 ($1.4\ DL + 1.6\ LL + 1.6\ E_0$). Determination of LL shall include live load reductions as permitted for calculation of seismic forces,

(c) Test load shall be applied in not less than four approximately equal increments and without shock to the structure and in such a manner as to avoid arching of loading materials,

(d) After the test load has been in position for 24 hours initial deflection readings shall be taken,

(e) Test load shall be removed immediately after initial deflection readings and final deflection readings taken 24 hours after removal of the test load.

H.3.6 If the portion of tested structure shows visible evidence of failure, the portion tested shall be considered to have failed the test and no retesting of the previously tested portion shall be permitted.

H.3.7 (1) If the portion of tested structure shows no visible evidence of failure, the following criteria shall be taken as indication of satisfactory performance.

(a) If measured maximum deflection of a beam, floor or roof (in mm) is less than $L^2/(800\ h)$ where L is the effective span of beam or one-way slab (mm) and h is the overall depth or thickness of member (mm),

(b) If the measured maximum deflection of a beam, floor or roof exceeds $L^2/(800h)$, deflection recovery within 24 hours after removal of test load shall be at least 75% of the maximum deflection for non-prestressed concrete, or 80% for prestressed concrete,

(2) In sub-para (1) (a) above L for cantilevers shall be taken as twice the distance from support to cantilever end, and deflection adjusted for any support movement.

H.3.8 Non-prestressed concrete construction failing to show 75% recovery of deflection as required by cl. H.3.7(2) above may be retested not earlier than 72 hours after removal of the first test load. The portion of structure tested shall be considered satisfactory if:

(a) it shows no visible evidence of failure in the retest;

(b) Deflection recovery caused by second test load is at least 80% of the maximum deflection in the second test; and

(c) prestressed concrete construction shall not be retested.

Non-Flexural Members

H.3.9 Members other than flexural members preferably shall be investigated by analysis.

Provision for Lower Load Rating

H.3.10 If the structure under investigation does not satisfy conditions or criteria of cl. H.2.1, H.3.7(1) or H.3.7(2), AERB may approve a lower load rating for that structure based on results of load test or analysis.

Safety

H.3.11 (a) Load tests shall be conducted such as to provide for safety of life and structure during the test,

(b) No safety measures shall interfere with load test procedures or affect results.

ANNEXURE-I

CONSIDERATION OF IMPULSIVE AND IMPACTIVE EFFECTS

I.1 General

I.1.1 Concrete structures important to safety of nuclear facilities shall be designed for impulsive and impactive loads (ref. cl. A.4 of AERB/SS/CSE). This annexure provides some useful information for this purpose. For detailed information reference may be made to Code Requirements for Nuclear Safety Related Concrete Structures (ACI349-90) and commentry (ACI349R-90).

I.1.2 Impactive and impulsive loads are considered concurrent with other loads (e.g. dead and live load) in determining the required resistance of structural elements.

I.2 Design Approach

I.2.1 In Appendix-A of AERB/SS/CSE, the impulsive and impactive loads are defined as dynamic class of loading. Table A.1 of Appendix-A contains various individual loads that fall under the category of impulsive and impactive loading. It is also identified in this Appendix that the effect of which individual loads should be considered as static type for the design of structures though the structural response for this loading could be determined considering the dynamic behaviour of the structures. For all other impulsive and impactive load, the structure should be designed using energy balance concept or elasto-plastic method unless specified otherwise.

I.2.2 If deformation under the effect of impulsive and impactive loading is within elastic limit, impulsive loading should be combined with other loading as required by provisions of this standard. The strain energy capacity available for resisting impulsive loads must then be reduced by the amount of work done by other load (factored, if limit state method is used, during maximum response).

I.2.3 In case of impactive and impulsive loading where a structural element deforms beyond its elastic limit, the provisions of load combinations as specified in this standard may not be applicable. In such cases either of the following approach may be adopted:

(a) If energy balance method is used, proper care shall be taken to consider the work done by static load such as dead load, live load, etc. (refer Fig. I.2),

(b) If an elastoplastic analysis is performed, the effective ductility ratio to be used in the design which is given by:

$$' = \frac{X_m \;\; X_s}{X_y \;\; X_s} = \frac{{}_d\, X_y \quad X_s}{X_y \;\; X_s}, \qquad \text{(I.1)}$$

where ' is the effective ductility ratio, ${}_d$ the displacement ductility ratio, X_y the displacement at yielding of material, X_s the static displacement and X_m the maximum acceptable displacement.

I.2.4 The ductility ratio ${}_d$ is defined as the ratio of maximum acceptable displacement X_m to the displacement at effective yield point of the structural elements (ref. Fig. I.1). This is known as displacement ductility.

Dynamic Strength Increase

I.2.5 Due to the effect of impulsive and impactive loading the rate of increase of strain is rapid, as a result of which the member exhibits high strength under this loading. The increase in strength should be considered in design by means of dynamic increase factor (DIF). DIF is the ratio of dynamic and static yield strength/stress. The values of DIF for different materials are given below:

Concrete

(i)	axial compression and flexure	[0.9+0.1 (log '+5.0)] $\leq$ 1.25,	(I.2)
(ii)	shear	[0.9+0.1 (log '+5.0)]^{½} $\leq$ 1.10,	(I.3)
(iii)	steel (for $f_y \leq 415$)	[1.05+0.08 (log '+3.0)] $\leq$ 1.15,	(I.4)

where ' is the strain rate (mm/mm/sec).

Design Requirement

I.2.6 Ductility ratio is calculated from deformations considering both shear and flexure displacement.

I.2.7 Ductility ratio used in design shall be such that the maximum deformation satisfies the following:

(i) does not exceed the limiting values stipulated in this standard,

(ii) does not result in loss of intended function of the structural element, and

(iii) does not impair the functions of other structures, systems and components important to safety.

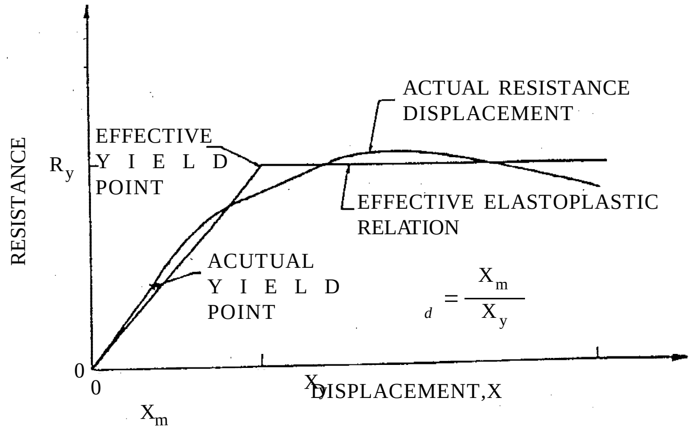
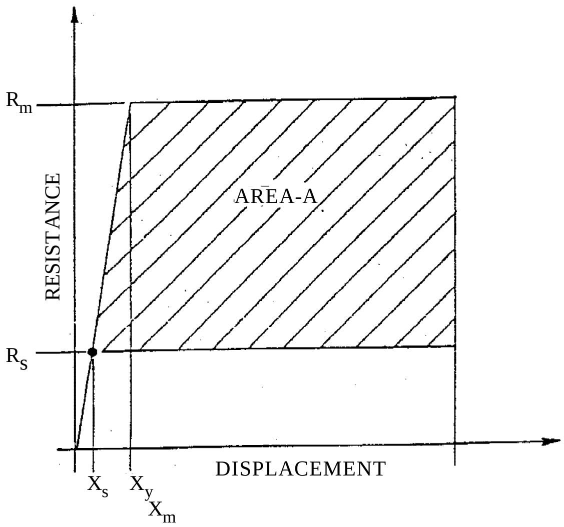


Fig. I.1: DUCTILITY RATIO d LOAD RESISTANCE RELATIONSHIP



$$d = \frac{X_m}{X_y}$$

R_S = Static force to be combined with impulsive or impactive loads

X_S = Displacement due to static loads

R_m = Maximum resistance

Fig. I.2: AVAILABLE RESISTANCE: IDEALISED RESISTANCE-DISPLACEMENT CURVE

I.2.8 The minimum structural resistance available for impulsive load shall be at least 20% greater than the magnitude of any portion of the impulsive loading which is approximately constant for a time equal or greater than the first fundamental period of the structural component.

I.3 Effects of Impulsive and Impactive Loading

Effects of Impulsive Load

I.3.1 When structural elements or systems of elements are subjected to impulsive loads, the structural response is determined by one of the following methods:

(a) Dynamic effects of impulsive loads may be considered by calculating a dynamic load factor (DLF). The resistance available for impulsive load is at least equal to the peak of impulsive load transient multiplied by DLF. DLF is calculated based on ductility criteria and the dynamic characteristics of the structure and impulsive load transient,

(b) Dynamic effects of impulsive loads may be considered using impulse, momentum, and energy balance techniques. Strain energy capacity is limited by ductility criteria,

(c) Dynamic effects of impulsive loads may be considered by performing a time-history dynamic analysis. Mass and inertial properties are included along with the nonlinear stiffnesses of structural elements under consideration. Simplified bilinear definitions of stiffness may also be used. Maximum predicted response is governed by ductility criteria.

Effects of Impactive Loading

I.3.2 In designing impactive effects, the following are considered.

(a) Design for impactive loads shall satisfy the criteria for both local effects and for overall structural response,

(b) Local impact effects may include penetration and perforation. For concrete structures, the considerations for local effects shall also include scabbing and punching shear,

(c) Penetration depth and required thickness to prevent perforation are based on applicable formulae or pertinent test data. When perforation of structural elements is precluded, the element thickness is at least 20% greater than that required to prevent perforation,

(d) Concrete structural elements protecting required system or equipment which could be damaged by secondary missiles (fragments of scabbed concrete) shall be designed to prevent scabbing, or provided with a scab shield, designed on the basis of applicable formulae or pertinent test data. In the absence of scab shields, the concrete thickness shall be at least 20% greater than that required to prevent scabbing,

(e) For concrete slabs or walls subject to missile impact effects where concrete thickness is less than twice that required to prevent perforation, the minimum reinforcement shall be 0.2% each way, each face, but not less than the requirement of minimum thickness given in this standard,

(f) When structural elements or systems of elements are subject to impactive loads, the structural response may be determined using the approach given in I.3.1.

I.4 Limiting Values of Ductility Ratio

I.4.1 For beams, walls, and slabs where flexure control design, the limiting value of ductility ratio is given by

$$0.05/(p \quad p_c) \leq 10.0 \qquad \text{(I.5)}$$

but not less than

$$(3.1 \quad 1.7\ p) \leq 1.0 \qquad \text{(I.6)}$$

I.4.2 The permissible ductility ratio in flexure shall be given by

$$0.4/(p \quad p_c) \leq 8.0 \qquad \text{(I.7)}$$

but not less than

$$0.8(3.1 \quad 1.7p) \geq 1.0 \qquad \text{(I.8)}$$

for loads generated by missiles.

I.4.3 In the design of flexure controls the rotational capacity r_e in radians of any yield hinge shall be limited to 0.0065 (d/x_u) but not exceeding 0.07 radians where x_u is the depth of neutral axis.

I.4.4 For flexure to control the design, thus allowing ductility ratios or rotational capacities given in clause I.4.1, I.4.2 and I.4.3 to be used, the load capacity of a structural element in shear shall be at least 20% greater than the load capacity in flexure, otherwise, the ductility ratios given in points I.4.5 or I.4.7 shall be used.

I.4.5 For beams, walls and slabs where shear controls design, the permissible ductility ratio shall be by the following, but not to exceed the available ductility ratio in flexure.

(a) for shear carried by concrete alone the permissible ductility ratio shall be 1.3;

(b) for shear carried by concrete and stirrups or bent bars, the permissible ductility ratio shall be 1.6; and

(c) for shear carried completely by stirrups, the permissible ductility ratio shall be 3.0.

I.4.6 For beam-columns, walls and slabs carrying axial compression loads and subject to impulsive or impactive loads producing flexure, the permissible ductility ratio in flexure shall be as follows:

(a) when compression controls the design, as defined by an interaction diagram, the permissible ductility ratio shall be 1.3,

(b) when compression load does not exceed 0.1 $f_{ck} A_g$ or one-third of that which would produce balanced conditions, whichever is smaller, the permissible ductility ratio shall be as given in I.4.1 or I.4.3, and

(c) the permissible ductility ratio shall vary linearly from 1.3 to that given in point I.4.1 or I.4.2 for conditions between those specified in (a) or (b).

I.4.7 For axial compressive impulsive or impactive loads, the permissible axial ductility ratio shall be 1.3.

BIBLIOGRAPHY

1. AERB (1998), Safety Standard on Civil Engineering Structures important to safety of Nuclear Facilities", AERB Safety Standard No. AERB/SS/CSE, Atomic Energy Regulatory Board, Mumbai, India.

2. DCL (1991), A Technical note on the P.C.C. Back Filling and Consolidation grouting for the approach Ramp Excavation to RB-3 and RB-4, Rajasthan Atomic Power Project, 3rd and 4th Units, Job No. 85R02, Report No. RAPP-3&4/15400/7001/SRA, Development Consultants Ltd., Calcutta.

3. AERB (1990), Code of Practice on Safety in Nuclear Power Plant Siting", AERB Safety Code No. AERB/SC/S, Atomic Energy Regulatory Board, Mumbai, India.

4. AERB (1990), Seismic Studies and Design Basis Ground Motion for Nuclear

Power Plant Sites, AERB Safety Guide No. AERB/SG/S-11, Atomic Energy Regulatory Board, Mumbai, India.

5. ACI (1990), 1990 Supplement, Code Requirements for Nuclear Safety Related Concrete Structures (ACI349-90) and Commentary (ACI349-90), American Concrete Institute, Redford Station, Detroit, USA.

6. AERB (1989), Code of Practice on Design for Safety in Pressurized Heavy Water Based Nuclear Power Plants, AERB Safety Code No. AERB/SC/D, Atomic Energy Regulatory Board, Mumbai, India.

7. ACI (1989), Building Code Requirements for Reinforced Concrete, (ACI318-83), American Concrete Institute, Redford Station, Detroit, Michigan, USA.

8. EDF-AFCEN (1989), Design and Construction Rules for Civil Works of PWR Nuclear Islands, RCC-G, Volume-1 Design, Electricite De France, Paris, France.

9. AERB (1988), Code of Practice on Quality Assurance for Safety in Nuclear Power Plants, AERB Safety Code No. AERB/SC/QA, Atomic Energy Regulatory Board, Mumbai, India.

10. ACI (1988), Details and Detailing of Concrete Reinforcement, Reported by ACI Committee 315, American Concrete Institute, Detroit, USA.

11. AERB (1996), Atomic Energy (Factories) Rules, India.

12. BSI (1987), British Standard Code of Practice for Design of Concrete Structures for Retaining Aqueous Liquids, BS8007:1987, British Standard Institution, U.K.

13. JSCE (1986), Standard Specification for Design and Construction of Concrete Structures, Part1(Design), First Edition, SP-1, Japan Soc. of Civil Engineers, Tokyo, Japan.

14. BSI (1985), British Standard, Structural Use of Concrete, Part-I, Code of

Practice for Design and Construction, BS8110, Part-I, British Standard Institution, U.K.

15. ACI (1985), Code Requirements for Nuclear Safety Related Concrete Structures (ACI349-85) and Commentary (ACI349R-85), American Concrete Institute, Redford Station, Detroit, USA.

16. CSA (1984), National Standard of Canada, Design of Concrete Structures for Buildings, CAN3-A23.3-M84, Canadian Standards Association, Toronto, Ontario, Canada.

17. SANZ (1982), New Zealand Standard Code of Practice for the Design of Concrete Structures, NZS3101, Part-I, Standards Association of New Zealand, Wellington, New Zealand.

18. SANZ (1982), Commentary on Code of Practice for the Design of Concrete Structures, NZS3101, Part-II, Standards Association of New Zealand, Wellington, New Zealand.

19. ACI (1982), Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures, ACI Committee 209, Publication SP-76, American Concrete Institute, Detroit, USA.

20. Nagataki Sand Yonek Ura A, Drying shrinkage and Creep of high strength concrete with super plasticiser, American Concrete Institute, Publication SP-76, American Concrete Institute, Detroit, USA.

21. USNRC (1981), Standard Review Plan 3.8.3- Concrete and Steel Internal Structures of Steel or Concrete Containments NUREG 0800, US Nuclear Regulatory Commission, Washington DC, USA.

22. USNRC (1981), Standard Review Plan 3.7.2 - Seismic System Analysis, NUREG 0800, US Nuclear Regulatory Commission, Washington DC, USA.

23. CSA (1981), Design Procedures for Seismic Qualification of CANDU Nuclear Power Plants, CAN3-N289.3-M81 Canadian Standards Association,

Ontario, Canada.

24. BIS (1980), Indian Standard Code of Practice for Prestressed Concrete (First Revision), IS:I343-1980, Bureau of Indian Standards, New Delhi.

25. ACI (1980), Fire Safety of Concrete Structures No. SP-80, American Concrete Institute, Detroit, USA.

26 BIS (1978), Indian Standard Code of Practice for Concrete Structures for Plain and Reinforced Concrete (Third Revision), IS:456-1978, Bureau of Indian Standards, New Delhi.

27. Park R. and Paulay T (1975), Reinforced Concrete Structures, John Wiley & Sons, New York.

28. BIS (1970), Indian Standard Code of Practice for Concrete Structures for the Storage of Liquids, IS:3370 (all parts), Bureau of Indian Standards, New Delhi, India.
29. Dayaratnam P. (1986), Design of Reinforced Concrete Structures, Oxford & IBH, New Delhi.
30. BIS (1989), Code of Practice for Fire Safety of Buildings (General); Details of Construction, IS: 1642, Bureau of Indian Standards, New Delhi.
31. AERB (1988), Safety Manual for Civil Engineering and Building Works of Nuclear Power Plants, AERB Safety Manual No. AERB/SM-S-1, Atomic Energy Regulatory Board, Mumbai, India.
32. P. Purushothaman (1984), Reinforced Concrete Structural Elements, Behaviour, Analysis and Design, Tata McGraw Hill, New Delhi.
33. CEB-FIP (1990), Model Code for Design of Concrete Structures.

LIST OF PARTICIPANTS

WORKING GROUP

Dates of meeting	:	March 26 & 27, 1991 March 19 to 21, 1992

Members of the working group*:

Dr. P. Dayaratnam (Chairman)	:	Ex-Dean of R&D, Indian Institute of Technology, Kanpur
Prof. V.N. Gupchup	:	Pro Vice-Chancellor (retd.), Mumbai University
Shri A.S. Warudkar	:	Nuclear Power Corporation of India Limited,

Mumbai

Dr. A.K. Kar : Engineering Services Int. Pvt. Ltd., Calcutta

Dr. P.C. Basu : Head, C&SED, AERB, Mumbai

---

* During the development phase of the standard, the above experts participated in one or more of the meetings.

CODE COMMITTEE FOR CIVIL AND STRUCTURAL ENGINEERING (CCCSE)

Dates of meeting: July 29, 1998
October 6 & 7, 1998

Members and invitees participating in the meeting:

Shri N.N. Kulkarni (Chairman) : Consultant, AERB

Prof. V.N. Gupchup : Pro-Vice-Chancellor (retd).,
Mumbai University

Director (Civil)	:	Bureau of Indian Standard, New Delhi
Dr. A. Dasgupta	:	DCL, Calcutta
Shri R.B. Gunde	:	Engineering Manager, TCE, Hyderabad
Shri A.S. Warudkar	:	Nuclear Power Corporation of India Ltd., Mumbai
Dr. A.K. Kar	:	Engineering Services Int. Pvt. Ltd. Calcutta
Dr. P.C. Basu	:	Head, C&SED, AERB, Mumbai
Shri L.R. Bishnoi (Permanent invitee since 27-7-1997)	:	C&SED, AERB, Mumbai

SUB-COMMITTEE OF EXPERTS (SCE)

Dates of meeting	:	December 10, 1991.
Joint meeting of SCE & CCCSE	:	May 5 & 6, 1992 June 10 & 11, 1992 June 24, 1992 July, 7 & 8, 1992 January 7 & 8, 1993

Members of the sub-committee and invitees participating in the meeting:

Dr. P. Dayaratnam (Chairman)	:	Ex-Dean of R&D, Indian Institue of Technology, Kanpur
Dr. A. Dasgupta	:	Professor of Civil Engineering and Dean (RD) Indian Institute of Technology, Guwahati
Shri S.P. Joshi	:	Tata Consulting Engineers, Mumbai

Director (Civil)	:	Bureau of Indian Standard, New Delhi
Shri N.C. Bandopadhyay (Alternate to Director, Civil)	:	Bureau of Indian Standard, New Delhi
Shri K.M. Kulkarni	:	Hindustan Construction Co. Ltd. Mumbai
Shri A.S. Warudkar	:	Nuclear Power Corporation of India Ltd., Mumbai
Shri U.S.P. Verma (Alternate to Shri A.S. Warudkar)	:	Nuclear Power Corporation of India Ltd., Mumbai
Shri S. Ramanujam	:	BARC, Mumbai
Dr. A.K. Kar	:	Engineering Services Int. Pvt. Ltd. Calcutta
Dr. S.K. Chowdhury	:	Dynatech Consultants Pvt. Ltd., Calcutta
Dr. P.C. Basu	:	Head, C&SED, AERB, Mumbai
Shri S.M. Palekar (Permanent invitee)	:	Tata Consulting Engineers, Mumbai

ADVISORY COMMITTEE ON NUCLEAR SAFETY (ACNS)\

Dates of meeting : March 28, 1998

Members and alternates participating in the meeting:

Shri S.K. Mehta (Chairman)	:	Director(Retd.), Reactor Group, BARC
Shri S.M.C. Pillai	:	Nagarjuna Power Corporation, Hyderabad
Prof. M.S. Kalra	:	IIT, Kanpur
Prof. U.N. Gaitonde	:	IIT, Bombay
Shri S.K. Goyal	:	BHEL

Shri Ch. Surendar	:	NPCIL (Formerly)
Dr. U.C. Mishra	:	BARC (Formerly)
Shri S.K. Sharma	:	BARC
Dr. V. Venkat Raj	:	BARC
Shri V.K. Chaturvedi	:	NPCIL
Shri M.S. Kumra	:	BARC (Formerly)
Shri S.P. Singh	:	AERB (Formerly)
Shri G.K. De (Member-Secretary),	:	AERB (Formerly)
Smt. Usha A. Menon (Permanent invitee),	:	AERB

PROVISIONAL LIST OF CIVIL AND STRUCTURAL ENGINEERING STANDARDS, GUIDES AND MANUALS

Safety Series Nos.	Provisional Title
Safety Standards	
AERB/SS/CSE	Civil Engineering Structures Important to Safety of Nuclear Facilities
AERB/SS/CSE-1	Design of Concrete Structures Important to Safety of Nuclear Facilities
AERB/SS/CSE-2	Design, Fabrication and Erection of Steel Structures Important to Safety of Nuclear Facilities
AERB/SS/CSE-3	Design of Nuclear Power Plant Containment Structures

PROVISIONAL LIST OF CIVIL AND STRUCTURAL ENGINEERING STANDARDS, GUIDES AND MANUALS (contd.)

Safety Series Nos.	Provisional Title
AERB/SS/CSE-4	Design, Fabrication and Erection of Embedded Parts and Penetrations Important to Safety of Nuclear Facilities
Safety Guides	
AERB/SG/CSE-1	Seismic Analysis and Qualification of Sructures, Systems, Equipment and Components Important to Safety of Nuclear Facilities (Being prepared by ACCGD as AERB/SG/D-23)
AERB/SG/CSE-2	Geotechnical Aspects for Buildings and Structures Important to Safety of Nuclear Facilities
AERB/SG/CSE-3	Quality Assurance for Construction of Civil Engineering Structures Important to Safety of Nuclear Facilities
AERB/SG/CSE-4	Materials of Construction for Civil Engineering Structures Important to Safety of Nuclear Facilities
Safety Manuals	
AERB/SM/CSE-1	Maintenance of Civil Engineering Structures Important to Safety of Nuclear Power Plants
AERB/SM/CSE-2	In-Service Inspection of Civil Engineering Structures Important to Safety of Nuclear Power Plants
AERB/SM/CSE-3	(Not to be published immediately)
AERB/SM/CSE-4	Civil Engineering Construction Inspection of Structures Important to Safety of Nuclear Facilities.
AERB/SM/CSE-5	Format of Safety Design Reports

NOTES