भारतीय मानक ब्यूरो
(उपभोक्ता मामले, खाद्य एवं सार्वजनिक वितरण मंत्रालय, भारत सरकार)
BUREAU OF INDIAN STANDARDS
(Ministry of Consumer Affairs, Food & Public Distribution, Govt. of India)

मानक भवन, 9, बहादुर शाह ज़फर मार्ग, नई दिल्ली – 110002
Manak Bhawan, 9, Bahadur Shah Zafar Marg, New Delhi – 110002
Mob.: 7071963270, Phones: 011-23608594, Extn: 8594
Website: www.bis.org.in , www.bis.gov.in

व्यापक परिचालन मसौदा

हमारा संदर्भ: सीईडी 46 /टी-6 **12 मार्च 2025**

तकनीकी समिति: भारत की राष्ट्रीय भवन निर्माण विषय समिति, सीईडी 46

प्राप्तकर्ता :

1. सिविल अभियांत्रिकी विभाग परिषद, सीईडीसी के सभी सदस्य
2. राष्ट्रीय भवन निर्माण संहिता विषय समिति, सीईडी 46 के सभी सदस्य
3. सीईडी 46 की उपसीमितयों और अन्य कार्यदल के सभी सदस्य
4. रुचि रखने वाले अन्य निकाय।

महोदय/महोदया,

निम्नलिखित मानक का मसौदा संलग्न हैं:

प्रलेख संख्या	शीर्षक
सीईडी 46 (26932)WC	**भारत की राष्ट्रीय भवन निर्माण संहिता भाग 6 संरचनात्मक डिजाइन अनुभाग 1 भार, बल और प्रभाव** [SP7(भाग 6 अनुभाग 1) का चौथा पुनरीक्षण] (आई सी एस नंबर: 01.120: 91.040.01, 91.080.01, 91.120.25)

कृपया इस मसौदे का अवलोकन करें और अपनी सम्मतियाँ यह बताते हुए भेजे कि यह मसौदा प्रकाशित हो तो इस पर अमल करने में आपको व्यवसाय अथवा कारोबार में क्या कठिनाइयां आ सकती हैं।

सम्मतियाँ भेजने की अंतिम तिथि: 13 अप्रैल 2025

सम्मति यदि कोई हो तो कृपया अधोहस्ताक्षरी को ई-मेल द्वारा ced46@bis.gov.in पर या उपरलिखित पते पर, संलग्न फोर्मेट में भेजें। सम्मतियाँ बीआईएस ई-गवर्नेंस पोर्टल, www.manakonline.in के माध्यम से ऑनलाइन भी भेजी जा सकती हैं।

यदि कोई सम्मति प्राप्त नहीं होती है अथवा सम्मति में केवल भाषा संबंधी त्रुटि हुई तो उपरोक्त प्रलेख को यथावत अंतिम रूप दे दिया जाएगा। यदि सम्मति तकनीकी प्रकृति की हुई तो विषय समिति के अध्यक्ष के परामर्श से अथवा उनकी इच्छा पर आगे की कार्यवाही के लिए विषय समिति को भेजे जाने के बाद प्रलेख को अंतिम रूप दे दिया जाएगा।

यह प्रलेख भारतीय मानक ब्यूरो की वैबसाइट www.bis.gov.in पर भी उपलब्ध हैं।
धन्यवाद।

भवदीय

ह/-
(द्वैपायन भद्र)
वैज्ञानिक 'ई' एवं प्रमुख (सिविल अभियांत्रिकी विभाग)

सलंग्न: उपरिलिखित

मानक भवन, 9, बहादुर शाह ज़फर मार्ग, नई दिल्ली – 110002
Manak Bhawan, 9, Bahadur Shah Zafar Marg, New Delhi – 110002
Mob.: 7071963270, Phones: 011-23608594, Extn: 8594
Website: www.bis.org.in , www.bis.gov.in

WIDE CIRCULATION DRAFT

Our Reference: CED 46/T-6 **12 March 2025**

National Building Code of India Sectional Committee, CED 46

ADDRESSED TO:

1. All Members of Civil Engineering Division Council, CEDC
2. All Members of the National Building Code Sectional Committee, CED 46
3. All Members of Subcommittees, Panels and Working Groups under CED 46
4. All others interests

Dear Sir/Madam,

Please find enclosed the following draft:

Doc No.	Title
CED 46 (26932) WC	**National Building Code of India Part 6 Structural Design Section 1 Loads Forces and Effects** [Fourth Revision of SP 7 (Part 6 Section 1)] (ICS No. 01.120: 91.040.01, 91.080.01, 91.120.25)

Kindly examine the attached draft and forward your views stating any difficulties which you are likely to experience in your business or profession, if this is finally adopted as National Standard.

Last Date for comments: 13 April 2025

Comments if any, may please be made in the enclosed format and emailed at **ced46@bis.gov.in** or sent at the above address. Additionally, comments may be sent online through the BIS e-governance portal, **www.manakonline.in**.

In case no comments are received or comments received are of editorial nature, kindly permit us to presume your approval for the above document as finalized. However, in case comments, technical in nature are received, then it may be finalized either in consultation with the Chairman, Sectional Committee or referred to the Sectional Committee for further necessary action if so desired by the Chairman, Sectional Committee.

The document is also hosted on BIS website **www.bis.gov.in**.

Thanking you,

Yours faithfully,

Sd/-
(Dwaipayan Bhadra)
Scientist 'E' / Director and Head
(Civil Engineering Department)

Encl: As above

FORMAT FOR SENDING COMMENTS ON THE DOCUMENT

[Please use A4 size sheet of paper only and type within fields indicated. Comments on each clause/sub-clause/ table/figure, etc, be stated on a fresh row. Information/comments should include reasons for comments, technical references and suggestions for modified wordings of the clause. **Comments through e-mail to ced46@bis.gov.in shall be appreciated**.

Doc. No.: CED 46 (26932) WC **BIS Letter Ref**: CED 46/T-6

Title: National Building Code of India Part 6 Structural Design Section 1 Loads Forces and Effects [Fourth Revision of SP 7 (Part 6 Section 1)] (ICS No. 01.120: 91.040.01, 91.080.01, 91.120.25)

Last date of comments: **13 April 2025**

Name of the Commentator/ Organization: ________________________________

Clause/ Para/ Table/ Figure No. commented	Comments/Modified Wordings	Justification of Proposed Change

NOTE- Kindly insert more rows as necessary for each clause/table, etc

BUREAU OF INDIAN STANDARDS

DRAFT FOR COMMENTS ONLY

(Not to be reproduced without the permission of BIS or used as a Standard)

Draft Indian Standard

National Building Code of India Part 6 Structural Design Section 1 Loads Forces and Effects

[Fourth Revision of SP 7 (Part 6 Section 1)]

(ICS No. 01.120: 91.040.01, 91.080.01, 91.120.25)

National Building Code Sectional Committee, CED 46	**Last Date for Comments: 13 April 2025**

C O N T E N T S

FOREWORD

1 SCOPE
2 DEAD LOAD
3 IMPOSED LOAD
4 WIND LOAD
5 EARTHQUAKE EFFECTS
6 SNOW LOAD
7 SPECIAL LOADS
8 LOAD COMBINATIONS
9 MULTI-HAZARD RISK IN VARIOUS DISTRICTS OF INDIA

ANNEX A DEAD LOADS – UNIT WEIGHTS OF CONSTRUCTION MATERIALS AND STORED MATERIALS

ANNEX B ILLUSTRATIVE EXAMPLE SHOWING REDUCTIN OF UNIFORMLY DISTRIBUTED IMPOSED FLOOR LOADS IN MULTI-STOREYED BUILDINGS FOR DESIGN OF COLUMNS

ANNEX C NOTATIONS

ANNEX D BASIC WIND SPEED AT 10 m HEIGHT FOR SOME IMPORTANT CITIES/TOWNS

ANNEX E CHANGES IN TERRAIN CATEGORIES

ANNEX F EFFECT OF A CLIFF OR ESCARPMENT ON EQUIVALENT HEIGHT ABOVE GROUND (k_3 FACTOR)

ANNEX G WIND FORCE ON CIRCULAR SECTIONS

ANNEX H SYMBOLS

ANNEX J MSK 1964 INTENSITY SCALE

ANNEX K EARTHQUAKE HAZARD ASSESSMENT FOR MACRO-ZONING AND SITE-SPECIFIC STUDIES

ANNEX L EARTHQUAKE ZONES OF SELECT TOWNS AS PER 2011 CENSUS OF INDIA

ANNEX M CHARACTERISTIC GROUND SNOW LOAD

ANNEX N SHAPE COEFFICIENTS FOR MULTILEVEL ROOFS

ANNEX P VIBRATIONS IN BUILDINGS

ANNEX Q BLAST LOAD

ANNEX R SUMMARY OF DISTRICTS HAVING SUBSTANTIAL MULTI-HAZARD RISK AREAS

LIST OF STANDARDS

National Building Code Sectional Committee, CED 46

FOREWORD

This Code (Part 6/Section 1) covers the various loads, forces and effects which are to be taken into account for structural design of buildings. The various loads that are covered under this Section are dead load, imposed load, wind load, seismic force, snow load, special loads and load combinations.

This Code was first published in 1970 and subsequently revised in 1983, 2005 and 2016. The first revision of this Section was modified in 1987 through Amendment No. 2 to the 1983 version of the Code to bring this Section in line with the latest revised loading code. Thereafter, in view of the revision of an important Indian Standard related to earthquake resistant design of structures, that is, IS 1893 a need arose to revise this part of NBC. Therefore, the second revision of 2005 was formulated to consider the revised standard, IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)' and to incorporate latest information on additional loads, forces and effects and the details regarding multi-hazard risk in various districts of India.

The significant changes incorporated in the second revision of 2005 included: the revision of the seismic zone map providing only four zones, instead of five, with erstwhile Zone I being merged into Zone II; changing of the values of seismic zone factors to reflect more realistic values of effective peak ground acceleration considering Maximum Considered Earthquake (MCE) and service life of structure in each seismic zone; specifying of response spectra for three types of founding strata, namely rock and hard soil, medium soil and soft soil; the revision of empirical expression for estimating the fundamental natural period T_a of multi-storeyed buildings with regular moment resisting frames; adoption of the procedure of first calculating the actual force that may be experienced by the structure during the probable maximum earthquake, if it were to remain elastic, also bringing in the concept of response reduction due to ductile deformation and associated energy dissipation by introducing the 'response reduction factor' in place of the earlier performance factor; specifying a lower bound for the design base shear of buildings, based on empirical estimate of the fundamental natural period T_a; deletion of the soil-foundation system factor and instead, introduction of a clause to restrict the use of foundations vulnerable to differential settlements in severe seismic zones; revision of torsional eccentricity values upwards in view of serious damages observed in buildings with irregular plans; revision of modal combination rule in dynamic analysis of buildings; inclusion of a new clause on multi-hazard risk in various districts of India and a list of districts identified as multi-hazard prone districts; incorporation of latest amendments issued to IS 875; introduction of a clause on vibration in buildings for general guidance; and inclusion of reference to the Indian Standards on landslide control and design of retaining walls which were formulated after the first revision of the Section.

The significant changes incorporated in the third revision of 2016 included: enhanced provisions for parapets and balustrades, incorporating concentrated horizontal loads and adjustments in horizontal load per unit length; new areas of application, such as appurtenances fixed to structures; aerodynamic roughness heights for different terrain categories were explicitly addressed, aiding in the determination of turbulence intensity and wind speed profiles; a new modification factor k_4 was introduced for cyclonic regions; simplified empirical expressions for variations in hourly mean wind speed and turbulence intensity across different terrains were suggested, along with provisions for directionality, area averaging, and correlation of pressures for design wind pressure; wind-induced interference for both tall and low-rise buildings was accounted for, with recommendations for wind tunnel or CFD studies for final designs of critical structures; updates to the wind speed map of India were included; and new provisions for design loads for helipads, emergency vehicles, and collisions with structural elements in parking areas. For earthquake effects, the design spectra was defined for natural periods up to 6 s, applicable to all buildings regardless of construction material; temporary structures and buildings with flat slabs were brought under the scope of the standard (though with some conditions), with guidelines for addressing structural irregularities and estimating the in-plane stiffness of masonry infill walls; additional revisions including simplified torsional provisions, methods for computing natural periods in various structures, new provisions for evaluating liquefaction potential and blast loads; and inputs were also derived from the Heliport Manual, 1995 of the International Civil Aviation Organization for the design of roof top helipad, etc.

In this revision, considering the recent developments and R&D efforts nationally and internationally, the following significant modifications and inclusions have been made:

a) Imposed loads for dwelling units planned and executed in accordance with IS 8888 : 2020 ‘Requirements of low income housing for Urban areas guide (*second revision*)’ have been made same as other residential buildings.

b) Unit weights of construction materials and stored materials have been included in Annex A.

c) Provisions relating to dead load for gardening and landscaping have been introduced.

d) Provision relating to testing of building fabrics against wind borne debris has been included.

e) Calculation of K_4 Factor as per IS 18315 : 2023 ‘Estimation of Cyclonic Factor (K4) – Guidelines’ has been included.

f) Table of basic wind speed for various cities of India has been updated.

g) The design earthquake hazard across the country has been revised comprehensively based on probabilistic considerations with the deterministic hazard taken as the minimum. Displacement hazard is also brought into focus alongside acceleration hazard.

h) The criteria for earthquake resistant design of structures have been revised substantially.

The changes in the latest provisions relating to Earthquake effects will be added suitably after the finalization/publication of Part 1 and 2 of Design Earthquake Hazard and Criteria for Earthquake Resistant Design of Structures - Part 1 : General Provisions and Part 2 Buildings [(Second Revision of IS 1893 (Part 1)].

i) A new term ‘characteristic ground snow load’ has been introduced and defined.

j) Characteristic ground snow load maps for union territory of Jammu and Kashmir, union territory of Ladakh, and states of Himachal Pradesh, Uttarakhand and Sikkim have been introduced, resulting from studies conducted by Defence Geoinformatics Research Establishment (DRDO), Chandigarh.

k) Equation for calculation of characteristic ground snow load has been suggested in Annex M.

l) Clause relating to partial loading due to melting, sliding, snow redistribution, and snow removal has been included.

m) Clause relating to ponding instability has been included.

n) Detailed provisions relating to design for blast loads have been included in Annex Q.

The provisions outlined in this Code are intended to assist structural designers in making informed engineering decisions when dealing with diverse loading scenarios, including special loads and unique load combinations that may arise in practical applications. It is expected that this Code will enable designers to adopt a rational and scientific approach in assessing structural loads and their combinations, thereby enhancing the resilience of structures against various external forces and ensuring their long-term performance. The Code serves as a fundamental reference for structural engineers, ensuring that the design approach aligns with safety requirements, reliability principles, and advancements in engineering practices. It integrates knowledge from past experiences, research developments, and international best practices while considering the specific conditions prevalent in India.

Structures are subjected to various loads, environmental effects, and material degradation over time, which can impact their performance and safety. Structural Health Monitoring (SHM) is a crucial process that enables the continuous or periodic assessment of structural integrity, ensuring early detection of potential damage and preventing catastrophic failures. SHM provides real-time insights into the behaviour of buildings and infrastructure under different conditions by utilizing advanced sensors, data acquisition systems, and analytical models. The need for SHM arises from the increasing complexity of modern structures, aging infrastructure, and the demand for enhanced safety and serviceability. It helps in identifying structural anomalies, degradation patterns, and potential failure modes before they become critical. The SHM system shall incorporate validated analytical models and industry-accepted evaluation techniques to ensure accurate assessment and decision-making. Implementing SHM in buildings and structures where necessary enhances durability, reduces maintenance costs, and supports proactive intervention strategies, ultimately improving structural resilience and public safety.

There has been a growing awareness among the consultants, academicians, researchers and practice engineers for design and construction of wind sensitive structures. In order to augment the available limited good quality meteorological wind data and structural response data, it is necessary to conduct full scale measurements in the field. Thus, as emphasized in the previous revision, all individuals and organizations responsible for putting-up of tall structures are encouraged to provide instrumentation in their existing and new structures at different elevations (at least at two levels) to continuously measure and monitor wind data. The instruments are required to collect data on wind direction, wind speed and structural response of the structure due to wind (with the help of accelerometer, strain gauges, etc). Such instrumentation in tall structures shall not in any way affect or alter the functional behaviour of such structures. The data so collected shall be very valuable in evolving more accurate wind loading of structures.

The information contained in this Section is largely based on the following Indian Standards:

IS 1893 (Part 1) : 20xx	Design earthquake hazard and criteria for earthquake resistant design of structures - Part 1 : General Provisions and Part 2 Buildings and buildings (sixth revision)
IS 1893 (Part 5) : 20xx	Design earthquake hazard and criteria for earthquake resistant design of structures - Part 5 Buildings
IS 875 (Part 1) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 1 Dead

	loads-unit weights of building material and stored materials (second revision)
IS 875 (Part 2) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 2 Imposed loads (*second revision*)
IS 875 (Part 3) : 2015	Design loads (other than earthquake) for buildings and structures — Code of practice: Part 3 Wind loads (*third revision*)
IS 875 (Part 4) : 2021	Code of practice for design loads (other than earthquake) for buildings and structures: Part 4 Snow loads (*second revision*)
IS 875 (Part 5) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 5 Special loads and load combinations (*second revision*)

This Section shall be read together with Sections 2 to 8 of Part 6 'Structural Design' of this Code.

A special publication, SP 64 (S&T): 2001 'Explanatory Handbook on Indian Standard Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures IS 875 (Part 3): 1987' is also available. This publication gives detailed background information on the provisions for wind loads and the use of these provisions for arriving at the wind loads on buildings and structures while evaluating their structural safety. But, in view of revision of IS 875 (Part 3) in 2015, applicability of this revised standard and this Section of the Code shall be taken as final in case of conflict of provisions with SP 64.

Reference may also be made to the Vulnerability Atlas of India, 2011 and Landslide Hazard Zonation Map of India, 2003, published by Building Materials and Technology Promotion Council, Ministry of Urban Development and Poverty Alleviation, Government of India. The Vulnerability Atlas contains information pertaining to each State and Union Territory of India, on (a) seismic hazard map, (b) cyclone, and wind map, (c) flood prone area map, and (d) housing stock vulnerability table for each district indicating for each house type the level of risk to which it could be subjected. The Atlas can be used to identify areas in each district of the country which are prone to high risk from more than one hazard. The information will be useful in establishing the need of developing housing designs to resist the combination of such hazards.

For criteria on structural safety of tall reinforced concrete (RC) buildings of heights greater than 50 m and less than 250 m, normally intended for use as residential, office and other commercial buildings, reference may be made to the Indian Standard IS 16700: 2023 ‘Criteria for Structural Safety of Tall Buildings’ (*first revision*).

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding of numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Important Explanatory Note for Users of the Code

In any Part/Section of this Code, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **"accepted standard"** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/ Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practices [6-1(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part/Section, that is, 875 (Part 1) : 1987 'Code of practice for design loads (other than Earthquake) for buildings and structures Part 1 Dead loads-unit weights of building material and stored materials (second revision)'

Draft Indian Standard

National Building Code of India Part 6 Structural Design Section 1 Loads Forces and Effects

[Fourth Revision of SP 7 (Part 6 Section 1)]

(ICS No. 01.120: 91.040.01)

National Building Code Sectional Committee, CED 46	Last Date for Comments: 13 April 2025

1 SCOPE

1.1 This Code (Part 6/Section 1) covers basic design loads to be assumed in the design of buildings. The loads specified in this section are minimum loads to be considered for design purposes in the design of buildings.

2 DEAD LOAD

2.1 Assessment of Dead Load

The dead load in a building shall comprise the weight of all walls, partitions, floors and roofs, and shall include the weights of all other permanent constructions (including built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, weight of fixed service equipment) in the building and shall conform to good practice [6-1(1)].

2.2 The list of dead loads of various construction materials and stored materials are given in Annex A.

2.3 Gardening and Landscaping

Elements on roof gardening or landscaping, including soil, plants, and drainage layers, along with other features like walkways, fences, and walls, are fixed in place and thus classified as dead loads. While the weight of walkways, fences, and walls remains constant. The weight of soil and drainage layers, which support plant growth, can vary significantly due to their ability to absorb and retain water. For load calculations, when the weight adds to other structural loads, the dead load should be based on fully saturated soil and drainage layers to account for maximum weight. If the weight is used to counteract uplift forces, calculations should assume the dry weight of soil and drainage layers. Vegetative or landscaped roof areas may retain even more water than fully saturated soil, either from rainfall or irrigation. This additional water retention should be included in the design to accurately assess load variations.

In the design calculation of overturning, uplifting and sliding effects, the beneficial effects of soil shall not be considered. When the soil is used as dead weight and locked permanently, the same may be considered.

3 IMPOSED LOAD

3.1 This clause covers imposed loads to be assumed in the design of buildings. The imposed loads specified herein are minimum loads which should be taken into consideration for the purpose of structural safety of buildings.

> NOTE – This Section does not cover detailed provisions for loads incidental to construction and special cases of vibration, such as moving machinery, heavy acceleration from cranes, hoists and the like. Such loads shall be dealt with individually in each case.

3.2 Terminology

3.2.1 For the purpose of imposed loads specified herein, the following definitions shall apply:

3.2.1.1 *Assembly buildings* – These shall include any building or part of a building where groups of people congregate or gather for amusement, recreation, social, religious, patriotic, civil, travel and similar purposes; for example, theatres, motion picture houses, assembly halls, city halls, marriage halls, town halls, auditoria, exhibition halls, museums, skating rinks, gymnasiums, restaurants (also used as assembly halls), place of worship, dance halls, club rooms, passenger stations and terminals of air, surface and other public transportation services, recreation piers and stadia, etc.

3.2.1.2 *Business buildings* – These shall include any building or part of a building, which is used for transaction of business (other than that covered by mercantile buildings); for keeping of accounts and records for similar purposes; offices, banks, professional establishments, court houses, and libraries shall be classified in this group so far as principal function of these is transaction of public business and the keeping of books and records.

> Note: The office buildings are the buildings primarily to be used as an office or for office purposes; 'office purposes' include the purpose of administration, clerical work, handling money, telephone and telegraph operating, and operating computers, calculating machines, 'clerical work' includes writing, book-keeping, sorting papers, typing, filing, duplicating, drawing of matter for publication and the editorial preparation of matter for publication, etc.

3.2.1.3 *Dwellings* – These shall include any building or part occupied by members of single/multi-family units with independent cooking facilities. These shall also include apartment houses (flats).

3.2.1.4 *Educational buildings* – These shall include any building used for school, college or day-care purposes involving assembly for instruction, education or recreation and which is not covered by assembly buildings.

3.2.1.5 *Imposed load* – The load assumed to be produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, loads due to impact and vibration, and dust loads but excluding wind, seismic, snow and other loads due to temperature changes, creep, shrinkage, differential settlement, etc.

3.2.1.6 *Industrial buildings* – These shall include any building or a part of a building or structure, in which products or materials of various kinds and properties are fabricated, assembled, manufactured or processed, for example, assembly plants, industrial laboratories, dry cleaning plants, power plants, generating units, pumping stations, fumigation chambers, laundries, buildings or structures in gas plants, refineries, dairies and saw-mills, etc.

3.2.1.7 *Institutional buildings* – These shall include any building or a part thereof, which is used for purposes such as medical or other treatment in case of persons suffering from physical and mental illness, disease or infirmity; care of infants, convalescents or aged persons and for penal or correctional detention in which the liberty of the inmates is restricted. Institutional buildings ordinarily provide sleeping accommodation for the occupants. It includes hospitals, sanatoria, custodial institutions or penal institutions like jails, prisons and reformatories.

3.2.1.8 *Occupancy or use group* – The principal occupancy for which a building or part of a building is used or intended to be used; for the purpose of classification of a building according to occupancy, an occupancy shall be deemed to include subsidiary occupancies which are contingent upon it.

3.2.1.9 *Mercantile buildings* – These shall include any building or a part of a building which is used as shops, stores, market for display and sale of merchandize either wholesale or retail. Office, storage and service and facilities incidental to the sale of merchandize and located in the same building shall be included under this group.

3.2.1.10 *Residential buildings* – These shall include any building in which sleeping accommodation is provided for normal residential purposes with or without cooking or dining or both facilities (except buildings under institutional buildings). It includes lodging or rooming houses, one or two-family private dwellings, dormitories, apartment houses (flats) and hotels.

3.2.1.11 *Storage buildings* – These shall include any building or part of a building used primarily for the storage or sheltering of goods, wares or merchandize, like warehouses, cold storages, freight depots, transit sheds, store houses, garages, hangers, truck terminals, grain elevators, barns and stables.

3.3 Imposed Loads on Floors Due to Use and Occupancy

3.3.1 *Imposed Loads*

The imposed loads to be assumed in the design of buildings shall be the greatest loads that probably will be produced by the intended use or occupancy, but shall not be less than the equivalent minimum loads specified in Table 1 subject to any reductions permitted in **3.3.2**.

Floors shall be investigated for both the uniformly distributed load (UDL) and the corresponding concentrated load specified in Table 1, and designed for the most adverse effects but they shall not be considered to act simultaneously. The concentrated loads specified in Table 1 may be assumed to act over an area of 0.3 m x 0.3 m. However, the concentrated loads need not be considered where the floors are capable of effective lateral distribution of this load.

All other structural elements shall be investigated for the effects of uniformly distributed loads on the floors specified in Table 1.

NOTES

1 Where, in Table 1, no values are given for concentrated load, it may be assumed that the tabulated distributed load is adequate for design purposes.

2 The loads specified in Table 1 are equivalent uniformly distributed loads on the plan area and provide for normal effects of impact and acceleration. They do not take into consideration special concentrated loads and other loads.

3 Where the use of an area or floor is not provided in Table 1, the imposed load due to the use and occupancy of such an area shall be determined from the analysis of loads resulting from:

a) weight of the probable assembly of persons;
b) weight of the probable accumulation of equipment and furnishing;
c) weight of the probable storage materials; and
d) impact factor, if any.

4 While selecting a particular loading, the possible change in use or occupancy of the building should be kept in view. Designers should not necessarily select in every case the lower loading appropriate to the first occupancy. In doing this they might introduce considerable restrictions in the use of the building at a later date, and thereby reduce its utility.

5 The loads specified herein, which are based on estimations, may be considered as the characteristic loads for the purpose of limit state method of design till such time statistical data are established based on load surveys to be conducted in the country.

6 When an existing building is altered by an extension in height or area, all existing structural parts affected by the addition shall be strengthened where necessary, and all new structural parts shall be designed to meet the requirements for building hereafter erected.

7 The loads specified in the Section does not include loads incidental to construction. Therefore, close supervision during construction is essential to ensure that overloading of the building due to loads by way of stacking of building materials or use of equipment (for example, cranes and trucks) during construction or loads which may be induced by floor to floor propping in multi-storeyed construction, does not occur. However, if construction loads were of short duration, permissible increase in stresses in the case of working stress method or permissible decrease in load factors in limit state method, as applicable to relevant design codes, may be allowed for.

8 The loads in Table 1 are grouped together as applicable to buildings having separate principal occupancy or use. For a building with multiple occupancies, the loads appropriate to the occupancy with comparable use shall be chosen from other occupancies.

9 Regarding loading on lift machine rooms including storage space used for repairing lift machines, designers should go by the recommendations of lift manufacturers for the present. Regarding loading due to false ceiling, the same should be considered as imposed loads on the roof/floor to which it is fixed.

Table 1 Imposed Floor Loads for Different Occupancies
(*Clause* 3.3.1)

Sl No.	**Occupancy Classification**	**Uniformly Distributed Load (UDL)** kN/m^2	**Concentrated Load** kN
(1)	(2)	(3)	(4)
i)	*Residential Buildings*		
	a) Dwelling houses:		
	1) All rooms and kitchens	2.0	1.8
	2) Toilets and bath rooms	2.0	–
	3) Corridors, passages, staircases including fire escapes and store rooms	3.0	4.5
	4) Balconies	3.0	1.5 per metre run concentrated at the outer edge
	b) Dwelling units planned and executed in accordance with [6-1(2)] only:		
	1) Habitable rooms, kitchens, and toilets and bath rooms	2.0	1.8
	2) Corridors, passages and staircases including fire escapes	3.0	4.5

Sl No.	**Occupancy Classification**	**Uniformly Distributed Load (UDL)** kN/m^2	**Concentrated Load** kN
(1)	(2)	(3)	(4)
	3) Balconies	3.0	1.5 per metre run concentrated at the outer edge
	c) Hotels, hostels, boarding houses, lodging houses, dormitories and residential clubs:		
	1) Living rooms, bed rooms and dormitories	2.0	1.8
	2) Kitchen and laundries	3.0	4.5
	3) Billiards room and public lounges	3.0	2.7
	4) Store rooms	5.0	4.5
	5) Dining rooms, cafeterias and restaurants	4.0	2.7
	6) Office rooms	2.5	2.7
	7) Rooms for indoor games	3.0	1.8
	8) Baths and toilets	2.0	–
	9) Corridors, passages staircases including fire escapes and lobbies – as per the floor serviced (excluding stores and the like) but not less than	3.0	4.5
	10) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m²	Concentrated Load kN
(1)	(2)	(3)	(4)
	d) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	e) Garages:		
	1) Garage floors (including parking area and repair workshops for passenger cars and vehicles not exceeding 2.5 tonne gross weight, including access ways and ramps – to be calculated but not less than	2.5	9.0
	2) Garage floors for vehicles not exceeding 4.0 tonne gross weight (including access ways and ramps) – to be calculated but not less than	5.0	9.0
ii)	*Educational Buildings*		
	a) Class rooms and lecture rooms (not used for assembly purposes)	3.0	2.7
	b) Dining rooms, cafeterias and restaurants	3.0[1)]	2.7
	c) Offices, lounges and staff rooms	2.5	2.7
	d) Dormitories	2.0	2.7
	e) Projection rooms	5.0	–
	f) Kitchens	3.0	4.5

Sl No.	**Occupancy Classification**	**Uniformly Distributed Load (UDL)** kN/m^2	**Concentrated Load** kN
(1)	(2)	(3)	(4)
	g) Toilets and bath rooms	2.0	–
	h) Store rooms	5.0	4.5
	j) Libraries and archives:		
	1) Stack room/stack area	6.0 kN/m^2 for a minimum height of 2.2 m + 2.0 kN/m^2 per metre height beyond 2.2 m	4.5
	2) Reading rooms (without separate storage)	4.0	4.5
	3) Reading rooms (with separate storage)	3.0	4.5
	k) Boiler rooms and plant rooms – to be calculated but not less than	4.0	4.5
	m) Corridors, passages, lobbies, staircases including fire escapes – as per the floor serviced (without accounting for storage and projection rooms) but not less than	4.0	4.5
	n) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
iii)	*Institutional Buildings*		
	a) Bed rooms, wards, dressing	2.0	1.8

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m²	Concentrated Load kN
(1)	(2)	(3)	(4)
	rooms, dormitories and lounges		
	b) Kitchens, laundries and laboratories	3.0	4.5
	c) Dining rooms, cafeterias and restaurants	3.0[1)]	2.7
	d) Toilets and bathrooms	2.0	–
	e) X-ray rooms, operating rooms and general storage areas – to be calculated but not less than	3.0	4.5
	f) Office rooms and O.P.D. rooms	2.5	2.7
	g) Corridors, passages, lobbies, staircases including fire escapes – as per the floor serviced (without accounting for storage and projection rooms) but not less than	4.0	4.5
	h) Boiler rooms and plant rooms – to be calculated but not less than	5.0	4.5
	j) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
iv)	*Assembly Buildings*		
	a) Assembly areas :		
	1) With fixed seats[2)]	4.0	–

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
	2) Without fixed seats	5.0	3.6
	b) Restaurants (subject to assembly), museums and art galleries and gymnasia	4.0	4.5
	c) Projection rooms	5.0	–
	d) Stages	5.0	4.5
	e) Office rooms, kitchens and laundries	3.0	4.5
	f) Dressing rooms	2.0	1.8
	g) Lounges and billiards rooms	2.0	2.7
	h) Toilets and bathrooms	2.0	–
	j) Corridors, passages and staircases including fire escapes	4.0	4.5
	k) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
	m) Boiler rooms and plant rooms including weight of machinery	7.5	4.5
	n) Corridors, passages, subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like corridors, staircases and passages in grandstands	5.0	4.5

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
v)	*Business and Office Buildings* (*see also* **3.2.1**)		
	a) Rooms for general use with separate storage	2.5	2.7
	b) Rooms without separate storage	4.0	4.5
	c) Banking halls	3.0	2.7
	d) Business computing machine rooms (with fixed computers or similar equipment)	3.5	4.5
	e) Records/files store rooms and storage space	5.0	4.5
	f) Vaults and strong rooms – to be calculated but not less than	5.0	4.5
	g) Cafeterias and dining rooms	3.0[1)]	2.7
	h) Kitchens	3.0	2.7
	j) Corridors, passages, lobbies, staircases including fire escapes – as per the floor serviced (excluding stores) but not less than	4.0	4.5
	k) Bath and toilets rooms	2.0	–
	m) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m²	Concentrated Load kN
(1)	(2)	(3)	(4)
	n) Stationary stores	4.0 for each metre of storage height	9.0
	p) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	q) Libraries	*See* Sl No. (ii)	
vi)	*Mercantile Buildings*		
	a) Retail shops	4.0	3.6
	b) Wholesale shops – to be calculated but not less than	6.0	4.5
	c) Office rooms	2.5	2.7
	d) Dining rooms, restaurants and cafeterias	3.0[1)]	2.7
	e) Toilets	2.0	–
	f) Kitchens and laundries	3.0	4.5
	g) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	h) Corridors, passages, stair-cases including fire escapes and lobbies	4.0	4.5
	j) Corridors, passages, staircases subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like	5.0	4.5

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m²	Concentrated Load kN
(1)	(2)	(3)	(4)
	k) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
vii)	*Industrial Buildings*[3)]		
	a) Work areas without machinery/ equipment	2.5	4.5
	b) Work areas with machinery/ equipment[4)]		
	1) Light duty } To be calculated but not less than	5.0	4.5
	2) Medium duty } To be calculated but not less than	7.0	4.5
	3) Heavy duty } To be calculated but not less than	10.0	4.5
	c) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	d) Cafeterias and dining rooms	3.0[1)]	2.7
	e) Corridors, passages, stair cases including fire escapes	4.0	4.5
	f) Corridors, passages, lobbies, staircases subject to machine loads and wheeled vehicles – to be calculated but not less than	5.0	4.5
	g) Kitchens	3.0	4.5
	h) Toilets and bathrooms	2.0	–
viii)	*Storage Buildings*[4)]		
	a) Storage rooms (other than cold storage) and warehouses – to be	2.4 kN/m² per metre of storage height with a	7.0

Sl No.		Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)		(2)	(3)	(4)
		calculated based on the bulk density of materials stored but not less than	minimum of 7.5 kN/m^2	
	b)	Cold storage – to be calculated but not less than	5.0 kN/m^2 per metre of storage height with a minimum of 15 kN/m^2	9.0
	c)	Corridors, passages, staircases including fire escapes – as per the floor serviced but not less than	4.0	4.5
	d)	Corridors, passages subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like	5.0	4.5
	e)	Boiler rooms and plant rooms	7.5	4.5

[1)] Where unrestricted assembly of persons is anticipated, the value of UDL should be increased to 4.0 kN/m^2.

[2)] With fixed seats implies that the removal of the seating and the use of the space for other purposes is improbable. The maximum likely load in this case is, therefore, closely controlled.

[3)] The loading in industrial buildings (workshops and factories) varies considerably and so three loadings under the terms 'light', 'medium' and 'heavy' are introduced in order to allow for more economical designs but the terms have no special meaning in themselves other than the imposed load for which the relevant floor is designed. It is, however important particularly in the case of heavy weight loads, to assess the actual loads to ensure that they are not in excess of 10 kN/m^2; in case where they are in excess, the design shall be based on the actual loadings.

[4)] For various mechanical handling equipment which are used to transport goods, as in warehouses, workshops, store rooms, etc, the actual load coming from the use of such equipment shall be ascertained and design should cater to such loads.

3.3.1.1 *Load application*

The uniformly distributed loads specified in Table 1 shall be applied as static loads over the entire floor area under consideration or a portion of the floor area, whichever arrangement produces critical effects on the structural elements as provided in respective design codes.

In the design of floors, the concentrated loads are considered to be applied in the positions which produce the maximum stresses and where deflection is the main criterion in the positions which produce the maximum deflections. Concentrated load, when used for the calculation of bending and shear, are assumed to act at a point. When used for the calculation of local effects such as crushing or punching, they are assumed to act over an actual area of application of 0.3 m x 0.3 m.

3.3.1.2 *Loads due to light partitions*

In office and other buildings, where actual loads due to light partitions cannot be assessed at the time of planning the floors and the supporting structural members shall be designed to carry, in addition to other loads, uniformly distributed loads per square metre of not less than 33.33 percent of weight per metre run of finished partitions, subject to a minimum of 1 kN/m^2, provided total weight of partition walls per m^2 of the wall area does not exceed 1.5 kN/m^2 and the total weight per metre length is not greater than 4.0 kN.

3.3.2 *Reduction in Imposed Loads on Floors*

3.3.2.1 *For members supporting floors*

Except as provided for in **3.3.2.1**(a), the following reductions in assumed total imposed loads on the floors may be made in designing columns, load bearing walls, piers, their supports and foundations:

Number of Floors (Including the Roof) to be Carried by Member Under Consideration	*Reduction in Total Distributed Imposed Load on All Floors to be Carried by the Member Under Consideration* Percent
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

a) No reduction shall be made for any plant or machinery which is specifically allowed for, or for buildings for storage purposes, warehouses and garages.

But, for other buildings, where the floor is designed for an imposed floor load of 5.0 kN/m^2 or more, the reductions shown in **3.3.2.1** may be taken provided that the loading assumed is not less than it would have been, if all the floors had been designed for 5.0 kN/m^2 with no reductions.

NOTE – In case, if the reduced load in the lower floor is lesser than the reduced load in the upper floor, then the reduced load of the upper floor will be adopted.

b) An example is given in Annex B illustrating the reduction of imposed loads in a multi-storeyed building in the design of column members.

3.3.2.2 *For beams in each floor level*

Where a single span of beam, girder or truss supports not less than 50 m^2 of floor at one general level, the imposed floor load may be reduced in the design of the beams, girders or trusses by 5 percent for each 50 m^2 area supported subject to a maximum reduction of 25 percent. But, no reduction shall be made in any of the following types of loads:

a) Any superimposed moving load,
b) Any actual load due to machinery or similar concentrated loads,
c) The additional load in respect of partition walls; and
d) Any impact or vibration.

NOTE – The above reduction does not apply to beams, girders or trusses supporting roof loads.

3.3.3 *Posting of Floor Capacities*

Where a floor or part of a floor of a building has been designed to sustain a uniformly distributed load exceeding 3.0 kN/m^2 and in assembly, business, mercantile, industrial or storage buildings, a permanent notice in the form shown below indicating the actual uniformly distributed and/or concentrated loadings for which the floor has been structurally designed shall be posted in a conspicuous place in a position adjacent to such floor or on such part of a floor.

DESIGNED IMPOSED FLOOR LOADING
Distributed……………..kN/m^2
Concentrated………….kN

Label Indicating Designed Imposed Floor Loading

NOTES

1 The lettering of such notice shall be embossed or cast suitably on a tablet whose least dimension shall not be less than 0.25 m and located not less than 1.5 m above floor level with lettering of a minimum size of 25 mm.

2 If a concentrated load or a bulk load has to occupy a definite position on the floor, the same could also be indicated in the table.

3.4 Imposed Loads on Roofs

3.4.1 *Imposed Loads on Various Types of Roofs*

On flat roofs, sloping roofs and curved roofs, the imposed loads due to use and occupancy of the buildings and the geometry of the types of roofs shall be as given in Table 2.

Table 2 Imposed Loads on Various Types of Roofs
(*Clause* 3.4.1)

Sl No.	Type of Roof	Imposed Load Measured on Plan Area	Minimum Imposed Load Measured on Plan
(1)	(2)	(3)	(4)
i)	Flat, sloping or curved roof with slopes up to and including 10 degrees		
	a) Access provided	1.5 kN/m^2	3.75 kN uniformly distributed over any span of one metre width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall
	b) Access not provided except for maintenance	0.75 kN/m^2	1.9 kN uniformly distributed over any span of one metre width of the roof slab and 4.5 kN uniformly distributed over the span of any beam or truss or wall
ii)	Sloping roof with slope greater than 10 degrees	For roof membrane sheets or purlins – 0.75 kN/m^2 less 0.02 kN/m^2 for every degree increase in slope over 10 degrees	Subject to a minimum of 0.4 kN/m^2

Sl No.	Type of Roof	Imposed Load Measured on Plan Area	Minimum Imposed Load Measured on Plan
(1)	(2)	(3)	(4)
iii)	Curved roof with slope of line obtained by joining springing point to the crown with the horizontal, greater than 10 degrees	$(0.75 – 0.52\ \alpha^2)$ kN/m^2 where $\alpha = h/l$ *h* = height of the highest point of the structure measured from its springing; and *l* = chord width of the roof if singly curved and shorter of the two sides if doubly curved. Alternatively, where structural analysis can be carried out for curved roofs of all slopes in a simple manner applying the laws of statistics, the curved roofs shall be divided into minimum 6 equal segments and for each segment imposed load shall be calculated appropriate to the slope of the chord of each segment as given in (i) and (ii).	Subject to a minimum of 0.4 kN/m^2

NOTES

1 The loads given above do not include loads due to snow, rain, dust collection, etc. The roof shall be designed for imposed loads given above or for snow/rain load, whichever is greater.

2 For special types of roofs with highly permeable and absorbent material, the contingency of roof material increasing in weight due to absorption of moisture shall be provided for.

3.4.1.1 Roofs of buildings used for promenade or incidental to assembly purposes shall be designed for the appropriate imposed floor loads given in Table 1 for the occupancy.

3.4.2 *Concentrated Load on Roof Coverings*

To provide for loads incidental to maintenance, unless otherwise specified by the Engineer-in-Charge, all roof coverings (other than glass or transparent sheets made of fibre glass) shall be capable of carrying an incidental load of 0.90 kN concentrated on an area of 1 250 mm^2 so placed as to produce maximum stresses in the covering. The intensity of the concentrated load may be reduced with the approval of the Engineer-in-Charge, where it is ensured that the roof coverings would not be traversed without suitable aids. In any case, the roof coverings shall be capable of carrying the loads in accordance with **3.4.1**, **3.4.3** and **3.4.4** and wind load.

3.4.3 *Loads Due to Rain*

On surfaces whose positioning, shape and drainage system are such as to make accumulation of rain water possible, loads due to such accumulation of water and the imposed loads for the roof as given in Table 2 shall be considered separately and the more critical of the two shall be adopted in the design.

3.4.4 *Dust Loads*

In areas prone to settlement of dust on roofs (Example, steel plants, cement plants), provision for dust load equivalent to probable thickness of accumulation of dust may be made.

3.4.5 *Loads on Members Supporting Roof Coverings*

Every member of the supporting structure which is directly supporting the roof covering(s) shall be designed to carry the more severe of the following loads except as provided in **3.4.5.1**:

a) The load transmitted to the members from the roof covering(s) in accordance with **3.4.1**, **3.4.3** and **3.4.4**; and
b) An incidental concentrated load of 0.90 kN concentrated over a length of 125 mm placed at the most critical positions on the member.

NOTE – Where it is ensured that the roofs would be traversed only with the aid of planks and ladders capable of distributing the loads on them to two or more supporting members, the intensity of concentrated load indicated in **3.4.5** (b) may be reduced to 0.5 kN with the approval of the Engineer-in-Charge.

3.4.5.1 In case of sloping roofs with slope greater than 10^o, members supporting the roof purlins, such as trusses, beams, girders, etc, may be designed for two-thirds of the imposed load on purlin or roofing sheets.

3.5 Imposed Horizontal Loads on Parapets, Balustrades and Other Appurtenances Fixed to Structure, and on Grandstands

3.5.1 *Parapets, Parapet Walls, Balustrades and other Appurtenances Fixed to the Structure such as Grab Bars, Fixed Ladders and Guardrails in Stilt Parking*

Parapets, parapet walls, balustrades and other appurtenances fixed to the structure such as grab bars, fixed ladders and guardrails in stilt parking, together with the members which give them structural support, shall be designed for the minimum loads given in Table 3. These are expressed as horizontal forces acting at handrail or coping level. These loads shall be considered to act vertically also but not simultaneously with the horizontal forces. The values given in Table 3 are minimum values and where values for actual loadings are available, they shall be used instead.

Table 3 Horizontal Loads on Parapets, Parapet Walls and Balustrades
(*Clause* 3.5.1)

Sl No.	**Usage Area**	**Intensity of Horizontal Load** kN/m Run	**Concentrated load (for each 3m length)** kN
(1)	(2)	(3)	(4)
i)	Light access stairs, gangways and like, not more than 600 mm wide	0.25	–
ii)	Light access stairs, gangways and like, more than 600 mm wide; stairways, landings, balconies and parapet walls (private and part of dwellings)	0.35	–
iii)	a) All other stairways, landings and balconies and all parapets and handrails to roofs [except those subject to overcrowding covered under (iv)]	0.75	0.89
	b) Panel fillers	0.20	0.22

Sl No.	**Usage Area**	**Intensity of Horizontal Load** kN/m Run	**Concentrated load (for each 3m length)** kN
(1)	(2)	(3)	(4)
iv)	Parapets and balustrades:		
	a) In places of assembly, such as restaurants and bars, retail and public areas, not likely to be overcrowded	1.5	1.5
	b) In places of assembly, such as retail areas, theatres, cinemas, bars, auditoria, shopping malls, discothèques, places of worship, likely to be overcrowded	3.0	1.5
v)	Grab bars	–	1.11
vi)	Fixed ladders	–	1.33
vii)	Guardrail systems and handrail assemblies	0.73	0.89

NOTE – In the case of guard parapets on a floor of multi-storeyed car park or crash barriers provided in certain buildings for fire escape, the value of imposed horizontal load (together with impact load) may be determined. For (vii), this load need not be considered for (a) one- and two- family dwellings, and (b) factory, industrial and storage occupancies, in areas that serve occupant load not greater than 22 kN.

3.5.2 *Grandstands and the Like*

Grandstands, stadia, assembly platforms, reviewing stands and the like shall be designed to resist a horizontal force applied to seats of 0.35 kN per linear metre along the line of seats and 0.15 kN per linear metre perpendicular to the line of the seats. These loadings need not be applied simultaneously. Platforms without seats shall be designed to resist a minimum horizontal force of 0.25 kN/m^2 of plan area.

3.6 Loading Effects Due to Impact and Vibration

The crane loads to be considered under imposed loads shall include the vertical loads, eccentricity effects induced by vertical loads, impact factors, lateral and longitudinal braking forces acting across and along the crane rails, respectively.

3.6.1 *Impact Allowance for Lifts, Hoists and Machinery*

The imposed loads specified in **3.3.1** shall be assumed to include adequate allowance for ordinary impact conditions. However, for structures carrying loads which induce impact or vibration, as far as possible, calculations shall be made for increase in the imposed load due to impact or vibration. In the absence of sufficient data for such calculation, the increase in the imposed loads shall be as follows:

	Structures	*Impact Allowance* Percent *Min*
a)	For frames supporting lifts and hoists	100
b)	For foundations, footings and piers supporting lifts and hoisting apparatus	40
c)	For supporting structures and foundations for light machinery, shaft or motor units	20
d)	For supporting structures and foundations for reciprocating machinery or power units	50

3.6.2 *Concentrated Imposed Loads with Impact and Vibration*

Concentrated imposed loads with impact and vibration which may be due to installed machinery shall be considered and provided for in the design. The impact factor shall not be less than 20 percent which is the amount allowable for light machinery.

3.6.2.1 Provision shall also be made for carrying any concentrated equipment loads while the equipment is being installed or moved for servicing and repairing.

3.6.3 *Impact Allowance for Crane Girders*

For crane gantry girders and supporting columns, the impact allowances (given below) shall be deemed to cover all forces set up by vibration, shock from slipping of slings, kinetic action of acceleration, and retardation and impact of wheel loads.

Type of Load	*Additional Load*
a) Vertical loads for electric overhead cranes	25 percent of maximum static loads for crane girders for all class of cranes 25 percent for columns supporting Class III and Class IV cranes 10 percent for columns supporting Class I and Class II cranes No additional load for design of foundations
b) Vertical loads for hand operated cranes	10 percent of maximum wheel loads for crane girders only
c) Horizontal forces transverse to rails:	
1) For electric overhead cranes with trolley having rigid mast for suspension of lifted weight (such as soaker crane, stripper crane, etc)	10 percent of weight of crab and the weight lifted by the cranes, acting on any one crane track rail, acting in either direction and equally distributed amongst all the wheels on one side of rail track For frame analysis, this force, calculated as above, shall be applied on one side of the frame at a time in either direction
2) For all other electric overhead cranes and hand operated cranes	5 percent of weight of crab and the weight lifted by the cranes, acting on any one crane track rail, acting in either direction and equally distributed amongst the wheels on one side of rail track For the frame analysis, the force, calculated as above, shall be applied on one side of the frame at a time in either direction
d) Horizontal traction forces along the rails for overhead cranes, either electrically operated or hand operated	5 percent of all static wheel loads

Forces specified in (c) and (d) shall be considered as acting at the rail level and being appropriately transmitted to the supporting system. Gantry girders and their vertical supports shall be designed on the assumption that either of the horizontal forces in (c) and (d) may act at the same time as the vertical load.

NOTE – *See* [6-1(3)] for classification (Class I to IV) of cranes.

3.6.3.1 *Overloading factors in crane supporting structures*

For all ladle cranes and charging cranes where there is possibility of overloading from production considerations, an overloading factor of 10 percent of the maximum wheel loading shall be taken.

3.6.4 *Crane Load Combinations*

In the absence of any specific indications, the load combinations shall be as indicated below.

3.6.4.1 *Vertical loads*

In an aisle, where more than one crane is in operation or has provision for more than one crane in future, the following load combinations shall be taken for vertical loading:

a) Two adjacent cranes working in tandem with full load and with overloading according to **3.6.3.1**; and

b) For long span gantries, where more than one crane can come in the span, the girder shall be designed for one crane fully loaded with overloading according to **3.6.3.1** plus as many loaded cranes as can be accommodated on the span but without taking into account overloading according to **3.6.3** (a) to give the maximum effect.

3.6.4.2 *Lateral surge*

For design of columns and foundations, supporting crane girders, the following crane combinations shall be considered:

a) *For single bay frames* – Effect of one crane in the bay giving the worst effect shall be considered for calculation of surge force; and

b) *For multi-bay frames* – Effect of two cranes working, one each in any of two bays in the cross section to give the worst effect shall be considered for calculation of surge force.

3.6.4.3 *Tractive force*

3.6.4.3.1 Where one crane is in operation with no provision for future crane, tractive force from only one crane shall be taken.

3.6.4.3.2 Where more than one crane is in operation or there is provision for future crane, tractive force from two cranes giving maximum effect shall be considered.

NOTE – Lateral surge force and longitudinal tractive force acting across and along the crane rail respectively shall not be assumed to act simultaneously. However, if there is only one crane in the bay, the lateral and longitudinal forces may act together simultaneously with vertical loads.

3.7 Rooftop Helipad

Elevated helipad structures shall be designed as per the latest guidelines of Directorate General of Civil Aviation and International Civil Aviation Organization's Heliport Manual.

3.7.1 *Structural Design*

Elevated helipads may be designed for a specific helicopter type though greater operational flexibility will be obtained from a classification system of design. The final approach and take-off area (FATO) should be designed for the largest or heaviest type of helicopter that is anticipated to use the helipad, and account taken of other types of loading such as personnel, freight, snow, refueling equipment, etc. For the purpose of design, it is to be assumed that the helicopter will land on two main wheels, irrespective of the actual number of wheels in the undercarriage, or on two skids, if they are fitted. The loads imposed on the structure should be taken as point loads at the wheel centres as shown below:

Helicopter Category	*Maximum Take-off Mass*		*Point Load for Each Wheel*	*Under-Carriage Wheel Centres*	*Super-imposed Load S_{Ha}*	*Super-imposed Load S_{Hb}*
	kg	kN	kN	M	kN/m^2	kN/m^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	Up to 2 300	Up to 22.6	12.0	1.75	0.5	1.5
2	2 301 - 5 000	22.6 - 49.2	25.0	2.0	0.5	2.0
3	5 001 - 9 000	49.2 - 88.5	45.0	2.5	0.5	2.5
4	9 001 - 13 500	88.5 - 133.0	67.0	3.0	0.5	3.0
5	13 501 - 19 500	133.0 - 192.0	96.0	3.5	0.5	3.0

Helicopter Category	*Maximum Take-off Mass*		*Point Load for Each Wheel*	*Under-Carriage Wheel Centres*	*Super-imposed Load* S_{Ha}	*Super-imposed Load* S_{Hb}
	kg	kN	kN	M	kN/m^2	kN/m^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)
6	19 501 - 27 000	192.0 - 266.0	133.0	4.5	0.5	3.0

3.7.2 The FATO should be designed for the worse condition derived from consideration of the following two cases.

3.7.3 *Case A – Helicopter on Landing*

When designing a FATO on an elevated helipad, and in order to cover the bending and shear stresses that result from a helicopter touching down, the following should be taken into account:

a) *Dynamic load due to impact on touchdown* – This should cover the normal touchdown, with a rate of descent of 1.8 m/s, which equates to the serviceability limit state. The impact load is then equal to 1.55 times the maximum take-off mass of the helicopter.

   The emergency touchdown should also be covered at a rate of descent of 3.6m/s (12 ft/s), which equates to the ultimate limit state. The partial safety factor in this case should be taken as 1.66. Therefore,

   Ultimate design load = 1.66 x service load
   = (1.66 x 1.5) maximum take-off mass
   = 2.5 maximum take-off mass

   The sympathetic response factor discussed at **3.7.3** (b) below should be applied.

b) *Sympathetic response on the FATO* – The dynamic load should be increased by a structural response factor dependent upon the natural frequency of the platform slab when considering the design of supporting beams and columns. This increase in loading will usually apply only to slabs with one or more freely supported edges. It is recommended that the average structural response factor (*R*) of 1.3 should be used in determining the ultimate design load.

c) *Over-all superimposed load on the FATO* (S_{Ha}) – To allow for snow load, personnel, freight and equipment loads, etc, in addition to wheel loads, an allowance of 0.5 KN/m^2 should be included in the design.

d) *Lateral load on the platform supports* – The supports of the platform should be designed to resist a horizontal point load equivalent to 0.5 maximum take-off mass of the helicopter, together with the wind loading [see (*f*)], applied in the direction which will provide the greater bending moments.

e) *Dead load of structural members* – The partial safety factor to be used for the dead load should be taken as 1.4.

f) *Wind loading* – Shall be as per **4**.

g) *Punching shear* – Check for the punching shear of an undercarriage wheel or skid using the ultimate design load with a contact area of 64.5 x 10^3 mm^2.

3.7.4 *Case B – Helicopter at Rest*

When designing a FATO on an elevated helipad, and in order to cover the bending and shear stresses from a helicopter at rest, the following should be taken into account:

a) *Dead load of the helicopter* – Each structural element must be designed to carry the point load, in accordance with the table under **3.7.1**, from the two main wheels or skids applied simultaneously in any position on the FATO so as to produce the worst effect from both bending and shear.
b) *Over-all superimposed load* (S_{Hb}) – In addition to wheel loads, an allowance for over-all superimposed load given in the table under **3.7.1**, over the area of the FATO, should be included in the design.
c) *Dead load on structural members and wind loading* – The same factors should be included in the design for these items as given for Case A.

NOTE – The above design loads for helicopters at rest are summarized below:

Design Load for Helicopter on Landing – Case A

i) Superimposed loads	
a) Helicopter	2.5 L_H distributed as two point loads at the wheel centres for the helicopter category given in the informal table under **3.7.1**. Average value for R = 1.3.
b) Lateral load	1.6 $\frac{L_H}{2}$ applied horizontally in any direction.
c) Over-all superimposed load	Load at platform level together with the maximum wind loading. 1.4 S_{Ha} over the whole area of the platform (S_{Ha} given in the informal table under **3.7.1**.).

ii) Dead load	1.4G
iii) Wind loading	1.4W
iv) Punching shear check	2.5 $L_H R$ load over tyre skid contact area of 64.5 x 10^3 mm^2.

Design load for helicopter at rest - Case B

i) Superimposed loads	
a) Helicopter	1.6 L_H distributed as two point loads at the wheel centres for the helicopter category given in the informal table under **3.7.1**.
b) Over-all superimposed load (Personnel, freight, etc.)	1.6 S_{Hb} over the whole area of the platform. S_{Hb} given in the informal table under **3.7.1**.
ii) Shear check	Check as appropriate

Symbol	*Meaning*
L_H	Maximum take-off mass of helicopter
G	Dead load of structure
W	Wind loading
R	Structural response factor
S_{Ha}	Superimposed load – Case A
S_{Hb}	Superimposed load – Case B

Load Type	*Partial Load Factors*
Dynamic load (ultimate design load)	2.5
Live load	1.6
Dead load	1.4
Wind loading	1.4

3.7.5 Normally, the upper load limit of the helicopter category selected should be used for design purposes except as follows:

> In order to avoid over-design in the platform the upper limit in any band may be exceeded by 10 percent should be the maximum take-off mass of a helicopter fall just into the next higher category.

3.8 Fire Tenders and Emergency Vehicles

Where a structure or portions of a structure are accessed and loaded by fire department access vehicles and other similar emergency vehicles, the structure shall be designed for the greater of the following loads:

a) The actual operational loads, including outrigger reactions and contact areas of the vehicles as stipulated and provided by the building official.
b) The contact area of wheel dimensions to be used considering 45˚ distribution at the reinforcement of slab considering the bear coat on the slab and any soil filling.

or

In absence of any data the bare slab loading may be considered as 15 to 20 kN/m^2.
c) When the fire tender load is combined with gravity loads and the same can be considered as accidental load with the partial load factor of 1.05.
d) Fire tender load need not be combined with lateral loads.

4 WIND LOAD

4.1 General

4.1.1 This clause gives wind forces and their effects (static and dynamic) that should be taken into account when designing buildings, structures and components thereof.

4.1.2 Wind speeds vary randomly both in time and space and hence assessment of wind loads and response predictions are very important in the design of several buildings and structures. A large majority of structures met with in practice do not however, suffer wind induced oscillations and generally do not require to be examined for the dynamic effects of wind. For such normal, short and heavy structures, estimation of loads using static wind analysis has proved to be satisfactory. The details of this method involving important wind characteristics such as the basic wind speeds, terrain categories, modification factors, wind pressure and force coefficients, etc, are given in **4.4** and **4.5**.

4.1.3 Nevertheless, there are various types of structures or their components such as some tall buildings which require investigation of wind induced oscillations. The influence of dynamic velocity fluctuations on the along wind loads (drag loads) for these structures shall be determined using Gust Factor Method, included in **4.8**. A method for calculation of across wind response of tall buildings is included in **4.8.3**.

4.1.4 This Section also applies to buildings or other structures during erection/construction and the same shall be considered carefully during various stages of erection/construction. In locations where the strongest winds and icing may occur simultaneously, loads on structural members, cables and ropes shall be calculated by assuming an ice covering based on climatic and local experience.

4.1.5 Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth's rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards. The wind generally blows horizontal to the ground at high wind speeds. Since vertical components of atmospheric motion are relatively small, the term 'wind' denotes almost exclusively the horizontal wind; vertical winds are always identified as such. The wind speeds are assessed with the aid of anemometers or anemographs which are installed at meteorological observatories at heights generally varying from 10 to 30 m above ground.

4.1.6 Very strong winds (more than 80 km/h) are generally associated with cyclonic storms, thunderstorms, dust storms or vigorous monsoons. A feature of the cyclonic storms over the Indian area is that they rapidly weaken after crossing the coasts and move as depressions/lows inland. The influence of a severe storm after striking the coast does not; in general exceed about 60 km, though sometimes, it may extend even up to 120 km. Very short duration hurricanes of very high wind speeds called Kal Baisaki or Norwesters occur fairly frequently during summer months over North East India.

4.1.7 The wind speeds recorded at any locality are extremely variable and in addition to steady wind at any time, there are effects of gusts which may last for a few seconds. These gusts cause increase in air pressure but their effect on stability of the building may not be so important; often, gusts affect only part of the building and the increased local pressures may be more than balanced by a momentary reduction in the pressure elsewhere. Because of the inertia of the building, short period gusts may not cause any appreciable increase in stress in main components of the building although the walls, roof sheeting and individual cladding units (glass panels) and their supporting members such as purlins, sheeting rails and glazing bars may be more seriously affected. Gusts can also be extremely important for design of structures with high slenderness ratios.

4.1.8 The liability of a building to high wind pressures depends not only upon the geographical location and proximity of other obstructions to air flow but also upon the characteristics of the structure itself.

4.1.9 The effect of wind on the structure as a whole is determined by the combined action of external and internal pressures acting upon it. In all cases, the calculated wind loads act normal to the surface to which they apply.

4.1.10 The stability calculations as a whole shall be done considering the combined effect, as well as separate effects of imposed loads and wind loads on vertical surfaces, roofs and other part of the building above general roof level.

4.1.11 Buildings shall also be designed with due attention to the effects of wind on the comfort of people inside and outside the buildings.

4.1.12 Wind borne debris present in cyclone or high speed wind has been causing a significant amount of damage to building envelopes. The actual in-service performance of fenestration assemblies and impact protective systems in areas prone to severe high speed wind depends on many factors. The main factors that affect the actual loading on building surfaces during a severe windstorm include varying wind direction, duration of the wind event, height above ground, building shape, terrain and surrounding structures. The resistance of fenestration or protective systems assemblies to wind loading after impact depends upon the product design, installation, load magnitude, duration and repetition. The method for determination of performance relating to safety of external building envelope like glazed curtain walls, exterior doors, exterior windows, impact protective systems (shutters, screens), skylight, wall cladding system, etc, impacted by wind borne debris, for buildings and other structures located in geographic regions that are prone to cyclone or extreme wind events shall be in accordance with [6-1(34)].

NOTES

1 This Section does not apply to buildings or structures with unconventional shapes, unusual locations, and abnormal environmental conditions that have not been covered in this Code. Special investigations are necessary in such cases to establish wind loads and their effects. Wind tunnel studies may also be required in such situations.

2 In the case of tall structures with unsymmetrical geometry, the designs may have to be checked for torsional effects due to wind pressure.

3 The provisions given in good practice [6-1(4)] may be suitably referred by the design engineer/planner for reference.

4.2 Notations

The notations to be followed unless otherwise specified in relevant clauses under wind loads are given in Annex C.

4.3 Terminology

4.3.1 For the purpose of wind loads, the following definitions shall apply.

4.3.1.1 *Angle of attack* – An angle between the direction of wind and a reference axis of the structure.

4.3.1.2 *Breadth* – It means horizontal dimension of the building measured normal to the direction of wind.

4.3.1.3 *Depth* – It means the horizontal dimension of the building measured in the direction of the wind.

NOTE – Breadth and depth are dimensions measured in relation to the direction of wind, whereas length and width are dimensions related to the plan.

4.3.1.4 *Developed height* – It is the height of upward penetration of the velocity profile in a new terrain. At large fetch lengths, such penetration reaches the gradient height, above which the wind speed may be taken to be constant. At lesser fetch lengths, a velocity profile of a smaller height but similar to that of the fully developed profile of that terrain category has to be taken, with the additional provision that the velocity at the top of this shorter profile equal to that of the un-penetrated earlier velocity profile at that height.

4.3.1.5 *Effective frontal area* – The projected area of the structure normal to the direction of wind.

4.3.1.6 *Element of surface area* – The area of surface over which the pressure coefficient is taken to be constant.

4.3.1.7 *Force coefficient* – A non-dimensional coefficient such that the total wind force on a body is the product of the force coefficient, the dynamic pressure of the incident design wind speed and the reference area over which the force is required.

NOTE – When the force is in the direction of the incident wind, the non-dimensional coefficient will be called as 'drag coefficient'. When the force is perpendicular to the direction of incident wind, the non-dimensional coefficient will be called as 'lift coefficient'.

4.3.1.8 *Ground roughness* – The nature of the earth's surface as influenced by small scale obstructions such as trees and buildings (as distinct from topography) is called ground roughness.

4.3.1.9 *Gust* – A positive or negative departure of wind speed from its mean value, lasting for not more than, say, 2 min over a specified interval of time.

4.3.1.10 *Peak gust* – A peak gust or peak gust speed is the wind speed associated with the maximum amplitude.

4.3.1.11 *Fetch length* – It is the distance measured along the wind from a boundary at which a change in the type of terrain occurs. When the changes in terrain types are encountered (such as, the boundary of a town or city, forest, etc), the wind profile changes in character but such changes are gradual and start at ground level, spreading or penetrating upwards with increasing fetch length.

4.3.1.12 *Gradient height* – It is the height above the mean ground level at which the gradient wind blows as a result of balance among pressure gradient force, Coriolis force and centrifugal force. For the purpose of this Section, the gradient height is taken as the height above the mean ground level, above which the variation of wind speed with height need not be considered.

4.3.1.13 *Tall building* – A building with a height more than or equal to 50 m or having a height to smaller lateral dimension more than 6.

4.3.1.14 *Low rise building* – A building having its height less than 20 m.

4.3.1.15 *Mean ground level* – The mean ground level is the average horizontal plane of the area enclosed by the boundaries of the structure.

4.3.1.16 *Pressure coefficient* – It is the ratio of the difference between the pressure acting at a point on the surface and the static pressure of the incident wind to the design wind pressure, where the static and design wind pressures are determined at the height of the point considered after taking into account the geographical location, terrain conditions and shielding effect. The pressure coefficient is also equal to [1 - $(V_p/V_z)^2$], where V_p is the actual wind speed at any point on the structure at a height corresponding to that of V_z.

NOTE – Positive sign of the pressure coefficient indicates pressure acting towards the surface and negative sign indicates pressure acting away from the surface.

4.3.1.17 *Return period* – It is the number of years, reciprocal of which gives the probability of extreme wind exceeding a given wind speed in anyone year.

4.3.1.18 *Shielding effect* – Shielding effect or shielding refers to the condition where wind has to pass along some structure(s) or structural element(s) located on the upstream wind side, before meeting the structure or structural element under consideration. A factor called 'shielding factor' is used to account for such effects in estimating the force on the shielded structures.

4.3.1.19 *Suction* – It means pressure less than the atmospheric (static) pressure and is taken to act away from the surface.

4.3.1.20 *Solidity ratio* – It is equal to the effective area (projected area of all the individual elements) of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

NOTE – Solidity ratio is to be calculated for individual frames.

4.3.1.21 *Terrain category* – It means the characteristics of the surface irregularities of an area which arise from natural or constructed features. The categories are numbered in increasing order of roughness.

4.3.1.22 *Topography* – The nature of the earth's surface as influenced by the hill and valley configurations.

4.3.1.23 *Velocity profile* – The variation of the horizontal component of the atmospheric wind speed at different heights above the mean ground level is termed as velocity profile.

4.4 Wind Speed

4.4.1 *Nature of Wind in Atmosphere*

In general, wind speed in the atmospheric boundary layer increases with height from zero at ground level to maximum at a height called the gradient height. There is usually a slight change in direction (Ekman effect) but this is ignored in this Section. The variation with height depends primarily on the terrain conditions. However, the wind speed at any height never remains constant and it has been found convenient to resolve its instantaneous magnitude into an average or mean value and a fluctuating component around this average value. The average value depends on the average time employed in analyzing the meteorological data and this averaging time varies from few seconds to several minutes. The magnitude of fluctuating component of the wind speed, which is called gust, depends on the averaging time. In general, smaller the averaging interval more is the magnitude of the gust speed.

4.4.2 *Basic Wind Speed*

Figure 1 gives basic wind speed map of India, as applicable to 10 m height above mean ground level for different zones of the country. Basic wind speed is based on peak gust velocity averaged over a short time interval of about 3 s and corresponds to mean heights above ground level in an open terrain (Category 2). Basic wind speeds presented in Fig. 1 have been worked out for a 50 year return period. Basic wind speed for some important cities/towns is also given in Annex D.

4.4.3 *Design Wind Speed* (V_z)

The basic wind speed (V_b) for any site shall be obtained from Fig. 1 and shall be modified to include the following effects to get design wind speed, V_z at any height z, for the chosen structure:

a) Risk level,
b) Terrain roughness and height of structure,
c) Local topography, and
d) Importance factor for the cyclonic region.

It can be mathematically expressed as follows:

$$V_z = V_b\, k_1\, k_2\, k_3\, k_4$$

where

V_z = design wind speed at height z, in m/s;
k_1 = probability factor (risk coefficient) (*see* **4.4.3.1**);
k_2 = terrain roughness and height factor (*see* **4.4.3.2**);
k_3 = topography factor (*see* **4.4.3.3**); and
k_4 = importance factor for the cyclonic region (*see* **4.4.3.4**).

NOTE – Wind speed may be taken as constant up to a height of 10 m. However, pressures for buildings less than 10 m high may be reduced by 20 percent for evaluating stability and design of the framing.

4.4.3.1 *Risk coefficient* (k_1 factor)

Figure 1 gives basic wind speeds for terrain Category 2 as applicable at 10 m above ground level based on 50 years mean return period. The suggested life period to be assumed in design and the corresponding k_1 factors for different class of structures for the purpose of design are given in Table 4. In the design of buildings and structures, a regional basic wind speed having a mean return period of 50 years shall be used except as specified in the Note of Table 4.

4.4.3.2 *Terrain, height factor* (k_2 factor)

4.4.3.2.1 *Terrain*

Selection of terrain categories shall be made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Wherever sufficient meteorological information is available about the nature of wind direction, the orientation of any building or structure may be suitably planned.

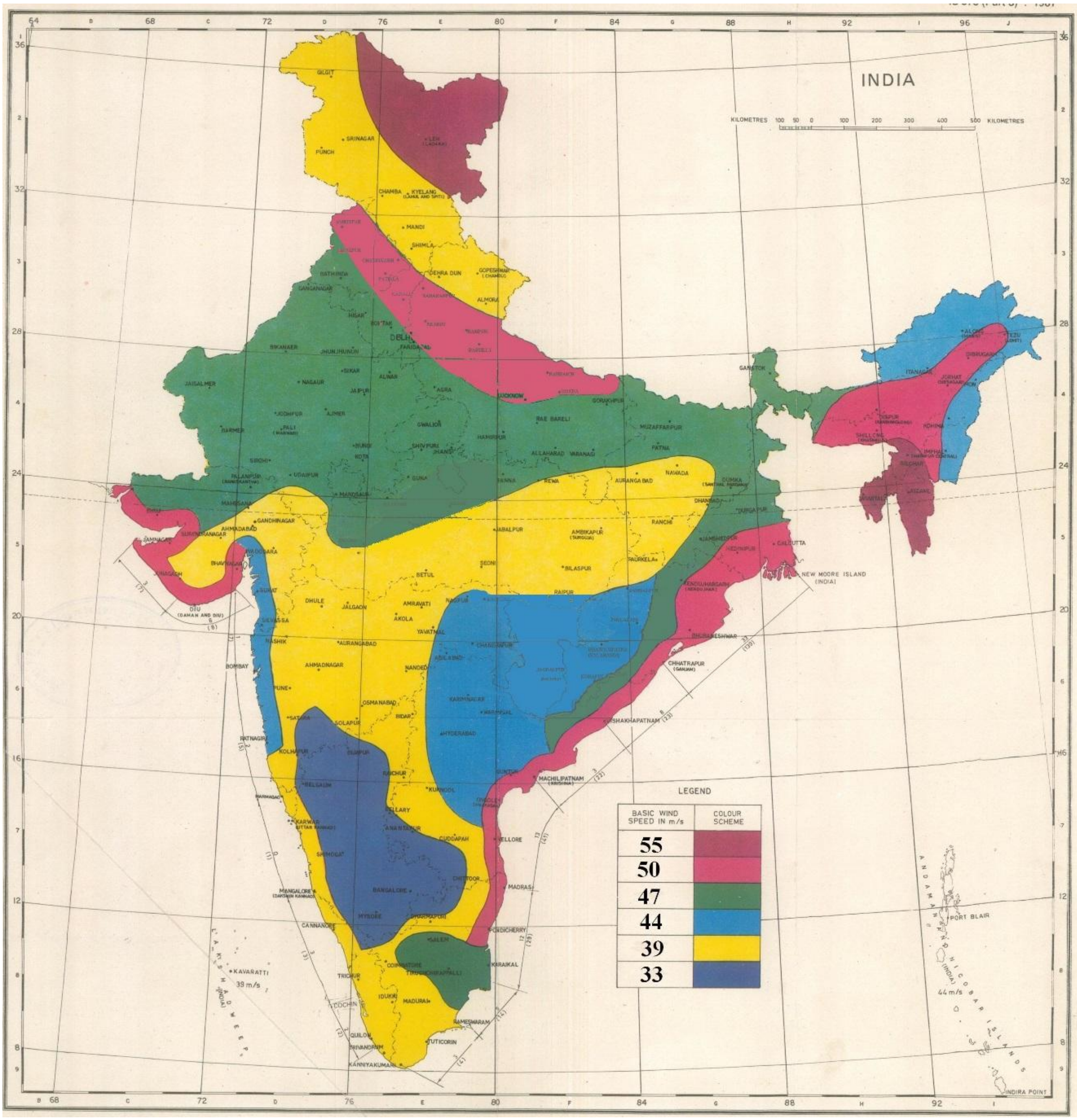
Terrain in which a specific structure stands shall be assessed as being one of the following terrain categories:

a) *Category 1* – Exposed open terrain with few or no obstructions and in which the average height of any object surrounding the structure is less than 1.5 m. The equivalent aerodynamic roughness height, ($z_{0,1}$) for this terrain is 0.002 m. Typically this category represents open sea-coasts and flat plains without trees.

b) *Category 2* – Open terrain with well scattered obstructions having heights generally between 1.5 m and 10 m. The equivalent aerodynamic roughness height, ($z_{0,2}$) for this terrain is 0.02 m. This is the criterion for measurement of regional basic wind speeds and represents airfields, open park lands and undeveloped sparsely built-up outskirts of towns and suburbs. Open land adjacent to sea coast may also be classified as Category 2 due to roughness of large sea waves at high winds.

c) *Category 3* – Terrain with numerous closely spaced obstructions having the size of buildings/structures up to 10 m in height with or without a few isolated tall structures. The equivalent aerodynamic roughness height, ($z_{0,3}$) for this terrain is 0.2 m. This category represents well wooded areas, and shrubs, towns and industrial areas full or partially developed. It is likely that the, next higher category than this will not exist in most design situations and that selection of a more severe category will be deliberate.

d) *Category 4* – Terrain with numerous large high closely spaced obstructions. The equivalent aerodynamic roughness height, ($z_{0,4}$) for this terrain is 2.0 m. This category represents large city centers, generally with obstructions above 25 m and well developed industrial complexes.



NOTES

1 The occurrence of a tornado is possible in virtually any part of India. They are particularly more severe in the northern parts of India. The recorded number of these tornados is too small to assign any frequency. The devastation caused by a tornado is due to exceptionally high winds about its periphery, and the sudden reduction in atmospheric pressure at its centre, resulting in an explosive outward pressure on the elements of the structure. The regional basic wind speeds do not include any specific allowance for tornados. It is not the usual practice to allow for the effect of tornados unless special requirements are called for as in the case of important structures such as, nuclear power reactors and satellite communication towers.

2 The total number of cyclonic storms that have struck different sections of east and west coasts are included in Fig. 1, based on available records for the period from 1877 to 1982. The figures above the lines (between the stations) indicate the total number of severe cyclonic storms with or without a core of hurricane winds (speeds above 87 km/h) and the figures in the brackets below the lines indicate the total number of cyclonic storms. Their effect on land is already reflected in the basic wind speeds specified in Fig. 1. These have been included only as additional information.

FIG. 1 BASIC WIND SPEED IN M/S (BASED ON 50 YEARS RETURN PERIOD)

Table 4 Risk Coefficients for Different Classes of Structures in Different Wind Speed Zones
(*Clause* 4.4.3.1)

Sl No.	**Class of Structure**	**Mean Probable Design Life of Structure in Years**	**k_1 Factor for Basic Wind Speed** (in m/s) **of**					
			33	39	44	47	50	55
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	All general buildings and structures, and ground based free standing tower up to 50 m	50	1.0	1.0	1.0	1.0	1.0	1.0
ii)	Temporary sheds, structures such as those used during construction operations (for example form-work and false work), structures during construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
iii)	Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings	25	0.94	0.92	0.91	0.90	0.90	0.89
iv)	Important buildings and structures, such as hospitals, communication buildings/towers, and power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

NOTE – The factor k_1 is based on statistical concepts which take into account the degree of reliability required and period of time in years during which there will be exposure to wind, that is, life of the structure. Whatever wind speed is adopted for design purposes, there is always a probability (however small) that it may be exceeded in a storm of exceptional violence; more the period of years over which there is exposure to the wind, more is the probability. Larger return periods ranging from 100 to 1 000 years (implying lower risk level) in association with larger periods of exposure may have to be selected for exceptionally important structures, such as, nuclear power reactors and satellite communication towers. Equation given below may be used in such cases to estimate k_1 factors for different periods of exposure and chosen probability of exceedance (risk level). The probability level of 0.63 is normally considered sufficient for design of buildings and structures against wind effects and the values of k_1 corresponding to this risk level are given above.

$$k_1 = \frac{X_{\mathrm{N,P}}}{X_{50,0.63}} = \frac{A - B\left[l_n\left\{-\frac{1}{N}l_n\left(1-P_N\right)\right\}\right]}{A+4B}$$

where

N = mean probable design life of structure in years;
P_N = risk level in N consecutive years (probability that the design wind speed is exceeded at least once in N successive years), nominal value = 0.63;
$X_{N,P}$ = extreme wind speed for given values of N and P_N; and
$X_{50,0.63}$ = extreme wind speed for N = 50 years and P_N = 0.63

A and *B* have the following values for different basic wind speed zones:

Zone m/s	***A**** m/s	***B**** m/s
33	23.1 (83.2)	2.6 (9.2)
39	23.3 (84.0)	3.9 (14.0)
44	24.4 (88.0)	5.0 (18.0)
47	24.4 (88.0)	5.7 (20.5)
50	24.7 (88.8)	6.3 (22.8)
55	25.2 (90.8)	7.6 (27.3)

* Values of *A* and *B* in 'km/h' are given within bracket

4.4.3.2.2 *Variation of wind speed with height in different terrains (k_2)*

Table 5 gives multiplying factors (k_2) by which the basic wind speed given in Fig. 1 shall be multiplied to obtain the wind speed at different heights, in each terrain category.

Table 5 Factors to Obtain Design Wind Speed Variation with Height in Different Terrains
(*Clause* 4.4.3.2.2)

Sl No.	**Height (z)** m	**Terrain and Height Multiplier (k_2)** Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
i)	10	1.05	1.00	0.91	0.80
ii)	15	1.09	1.05	0.97	0.80
iii)	20	1.12	1.07	1.01	0.80
iv)	30	1.15	1.12	1.06	0.97
v)	50	1.20	1.17	1.12	1.10
vi)	100	1.26	1.24	1.20	1.20
vii)	150	1.30	1.28	1.24	1.24
viii)	200	1.32	1.30	1.27	1.27
ix)	250	1.34	1.32	1.29	1.28
x)	300	1.35	1.34	1.31	1.30
xi)	350	1.35	1.35	1.32	1.31
xii)	400	1.35	1.35	1.34	1.32

Sl No.	Height (z) m	Terrain and Height Multiplier (k_2)			
		Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
xiii)	450	1.35	1.35	1.35	1.33
xiv)	500	1.35	1.35	1.35	1.34

NOTE – For intermediate values of height *z* in a given terrain category, use linear interpolation.

4.4.3.2.3 *Terrain categories in relation to the direction of wind*

The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Where sufficient meteorological information is available, the basic wind speed may be varied for specific wind direction.

4.4.3.2.4 *Changes in terrain categories*

The velocity profile for a given terrain category does not develop to full height immediately with the commencement of that terrain category but develop gradually to height (h_x) which increases with the fetch or upwind distance (*x*).

a) *Fetch and Developed Height Relationship* – The relation between the developed height (h_x) and the fetch (*x*) for wind-flow over each of the four terrain categories may be taken as given in Table 6.
b) For structures of heights more than the developed height (h_x) in Table 6, the velocity profile may be determined in accordance with the following:
    1) The less or least rough terrain, or
    2) The method described in Annex E.

Table 6 Fetch and Developed Height Relationship
(*Clause* 4.4.3.2.4)

Sl No.	Fetch (x) km	Developed Height (h_x) m			
		Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
i)	0.2	12	20	35	60
ii)	0.5	20	30	35	95
iii)	1	25	45	80	130
iv)	2	35	65	110	190
v)	5	60	100	170	300
vi)	10	80	140	250	450
vii)	20	120	200	350	500
viii)	50	180	300	400	500

4.4.3.3 *Topography (k_3 factor)*

The basic wind speed V_b given in Fig. 1 takes into account the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliffs, steep escarpments, or ridges.

The effect of topography shall be significant at a site when the upwind slope (θ) is more than about 3°, and below that, the value of k_3 may be taken to be equal to 1.0. The value of k_3 is confined in the range of 1.0 to 1.36 for slopes more than 3°. A method of evaluating the value of k_3 for values more than 1.0 is given in Annex F. It may be noted that the value of k_3 varies with height above ground level, with a maximum near the ground, and reducing to 1.0 at higher levels.

4.4.3.4 *Importance factor for cyclonic region (k_4 factor)*

The east coast of India is relatively more vulnerable for occurrences of severe cyclones. On the west coast, Gujarat is vulnerable for severe cyclones. Studies of wind speed and damage to buildings and structures point to the fact that the speeds given in the basic wind speed map are often exceeded during the cyclones. The effect of cyclonic storms is largely felt in a belt of approximately 60 km width at the coast. In order to ensure better safety of structures in this region (60 km wide on the east coast as well as on the Gujarat coast), the values of k_4, as recommended in good practice [6-1(32)] are stipulated as applicable according to the importance of the structure:

	k_4
Structures of post-cyclone importance for emergency services (such as cyclone shelters, hospitals, schools, communication towers, etc)	1.30
Industrial structures	1.15
All other structures	1.00

4.4.4 *Hourly Mean Wind Speed*

The hourly mean wind speed at height *z*, for different terrains can be obtained as:

$$\bar{V}_{z,H} = \bar{k}_{2,i} V_b$$

where

$\bar{k}_{2,i}$ = hourly mean wind speed factor for terrain category 'i'

$$= 0.1423 \left[\ln\left(\frac{z}{z_{0,i}} \right) \right] \left(z_{0,i} \right)^{0.0706}$$

The design hourly mean wind speed at height z can be obtained as:

$$\bar{V}_{z,d} = \bar{V}_{z,H} k_1 k_3 k_4$$

$$= V_b k_1 \overline{k}_{2,i} k_3 k_4$$

4.4.5 *Turbulence Intensity*

The turbulence intensity variations with height for different terrains can be obtained using the relations given below:

a) *Terrain category 1*

$$I_{z,1} = 0.350\,7 - 0.053\,5 \log_{10}\left(\frac{z}{z_{0,1}}\right)$$

b) *Terrain category 2*

$$I_{z,2} = I_{z,1} + \frac{1}{7}\left(I_{z,4} - I_{z,1}\right)$$

c) *Terrain category 3*

$$I_{z,3} = I_{z,1} + \frac{3}{7}\left(I_{z,4} - I_{z,1}\right)$$

d) *Terrain category 4*

$$I_{z,4} = 0.466 - 0.135\,8 \log_{10}\left(\frac{z}{z_{0,4}}\right)$$

4.4.6 *Off-shore Wind Velocity*

Cyclonic storms form far away from the sea coast and gradually reduce in speed as they approach the sea coast. Cyclonic storms generally extend up to about 60 km inland after striking the coast. Their effect on land is already reflected in basic wind speeds specified in Fig. 1. The influence of wind speed off the coast up to a distance of about 200 km may be taken as 1.15 times the value on the nearest coast in the absence of any definite wind data. The factor 1.15 shall be used in addition to k_4.

4.5 Wind Pressures and Forces on Buildings/Structures

4.5.1 *General*

The wind load on a building/structure shall be calculated for,

a) The building/structure as a whole;
b) Individual structural elements as roofs and walls; and
c) Individual cladding units including glazing and their fixings.

4.5.2 *Design Wind Pressure*

The wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind speed:

$$p_z = 0.6 V_z^2$$

where

p_z = wind pressure at height z, in N/m^2; and

V_z = design wind speed at height z, in m/s.

The design wind pressure p_d can be obtained as,

$$p_d = K_d\, K_a\, K_c\, p_z$$

where

K_d = wind directionality factor,
K_a = area averaging factor, and
K_c = combination factor [*see* **4.5.3.3.13**].

The value of p_d, however shall not be taken as less than 0.70 p_z.

NOTES

1 The coefficient 0.6 (in SI units) in the above formula depends on a number of factors and mainly on the atmospheric pressure and air temperature. The value chosen corresponds to the average Indian atmospheric conditions.

2 K_d should be taken as 1.0 when considering local pressure coefficients.

4.5.2.1 *Wind directionality factor, K_d*

Considering the randomness in the directionality of wind and recognizing the fact that pressure or force coefficients are determined for specific wind directions, it is specified that for buildings, solid signs, open signs, lattice frameworks, and trussed towers (triangular, square, rectangular) a factor of 0.90 may be used on the design wind pressure. For circular or near-circular forms this factor may be taken as 1.0.

For the cyclone affected regions also the factor K_d shall be taken as 1.0.

4.5.2.2 *Area averaging factor, K_a*

Pressure coefficients given in **4.5.3** are a result of averaging the measured pressure values over a given area. As the area becomes larger, the correlation of measured values decrease and *vice-versa*. The decrease in pressures due to larger areas may be taken into account as given in Table 7.

Table 7 Area Averaging Factor (K_a)
(*Clause* 4.5.2.2)

Sl No.	**Tributary Area (A)** m^2	**Area Averaging Factor (K_a)**[1)]
(1)	(2)	(3)
i)	$\leq$ 10	1.0
ii)	25	0.9
iii)	$\geq$ 100	0.8

[1)] Linear interpolation for intermediate values of tributary area, A is permitted.

4.5.2.2.1 *Tributary area*

a) *Overall structure* – For evaluating loads on frames the tributary area shall be taken as the centre to centre distances between frames multiplied by the individual panel dimension in the other direction together with overall pressure coefficients.

b) *Individual elements* – For beam type elements, purlins, etc, the tributary area shall be taken as effective span multiplied by spacing. The effective span is the actual span for mid span and cantilever load effects; and half the sum of adjacent spans for support moments and reactions.

c) For plate type elements, the area of individual plates between supports is taken as the tributary area.

d) For glass cladding, individual pane area of glass is the tributary area.

4.5.3 *Pressure Coefficients*

The pressure coefficients are always given for a particular surface or part of the surface of a building/structure. The wind load acting normal to a surface is obtained by multiplying the area of that surface or its appropriate portion by the pressure coefficient (C_p) and the design wind pressure at the height of the surface from the ground. The average values of these pressure coefficients for some building shapes are given in **4.5.3.2** and **4.5.3.3**.

Average values of pressure coefficients are given for critical wind directions in one or more quadrants. In order to determine the maximum wind load on the building/structure, the total load should be calculated for each of the critical directions shown from all quadrants. Where considerable variation of pressure occurs over a surface, it has been sub-divided and mean pressure coefficients given for each of its several parts.

In addition, areas of high local suction (negative pressure concentration) frequently occurring near the edges of walls and roofs are separately shown. Coefficients for the local effects should only be used for calculation of forces on these local areas affecting roof sheeting, glass panels, and individual cladding units including their fixtures. They should not be used for calculating force on entire structural elements such as roof, walls or structure as a whole.

NOTES

1 The pressure coefficients given in different tables have been obtained mainly from measurements on models in wind tunnels, and the great majority of data available has been obtained in conditions of relatively smooth flow. Where sufficient field data exists as in the case of rectangular buildings, values have been obtained to allow for turbulent flow.

2 In recent years, wall glazing and cladding design has been a source of major concern.

Although of less consequence than the collapse of main structures, damage to glass can be hazardous and cause considerable financial losses.

3 For pressure coefficients for structures not covered herein, reference may be made to specialist literature on the subject or advice may be sought from specialists in the subject.

4.5.3.1 *Wind load on individual members*

When calculating the wind load on individual structural elements such as roofs and walls, and individual cladding units and their fittings, it is essential to take account of the pressure difference between opposite faces of such elements or units. For clad structures, it is, therefore, necessary to know the internal pressure as well as the external pressure. Then the wind load, F (in N), acting in a direction normal to the individual structural element or cladding unit is:

$$F = (C_{pe} - C_{pi})\, A\, p_d$$

where

C_{pe}	=	external pressure coefficient;
C_{pi}	=	internal pressure coefficient;
A	=	surface area of structural element or cladding unit, in m^2; and
p_d	=	design wind pressure, in N/ m^2.

NOTES

1 If the surface design pressure varies with height, the surface areas of the structural element may be sub-divided so that the specified pressures are taken over appropriate areas.

2 Positive wind load indicates the force acting towards the structural element and negative away from it.

4.5.3.2 *Internal pressure coefficients*

Internal air pressure in a building depends upon the degree of permeability of cladding to the flow of air. The internal air pressure may be positive or negative depending on the direction of flow of air in relation to openings in the buildings.

a) In the case of buildings where the claddings permit the flow of air with openings not more than about 5 percent of the wall area but where there are no large openings, it is necessary to consider the possibility of the internal pressure being positive or negative. Two design conditions shall be examined, one with an internal pressure coefficient of +0.2 and another with an internal pressure coefficient of -0.2.

The internal pressure coefficient is algebraically added to the external pressure coefficient and the analysis which indicates greater distress of the member shall be adopted. In most situations a simple inspection of the sign of external pressure will at once indicate the proper sign of the internal pressure coefficient to be taken for design.

NOTE ─ The term normal permeability relates to the flow of air commonly afforded by claddings not only through open windows and doors, but also through the slits round the closed windows and doors and through chimneys, ventilators and through the joints between roof coverings, the total open area being less than 5 percent of area of the walls having the openings.

b) *Buildings/structures with medium and large openings* — Buildings/structures with medium and large openings may also exhibit either positive or negative internal pressure depending upon the direction of wind. Buildings/structures with medium openings between about 5 to 20 percent of wall area shall be examined for an internal pressure coefficient of +0.5 and later with an internal pressure coefficient of -0.5, and the analysis which produces greater distress of the member shall be adopted. Buildings/structures with large openings, that is, openings larger than 20 percent of the wall area shall be examined once with an internal pressure coefficient of +0.7 and again with an internal pressure coefficient of -0.7, and the analysis which produces greater distress of the member shall be adopted.

Buildings/structures with one open side or opening exceeding 20 percent of wall area may be assumed to be subjected to internal positive pressure or suction similar to those of buildings with large openings. A few examples of buildings with one side openings are shown in Fig. 2 indicating values of internal pressure coefficients with respect to the direction of wind.

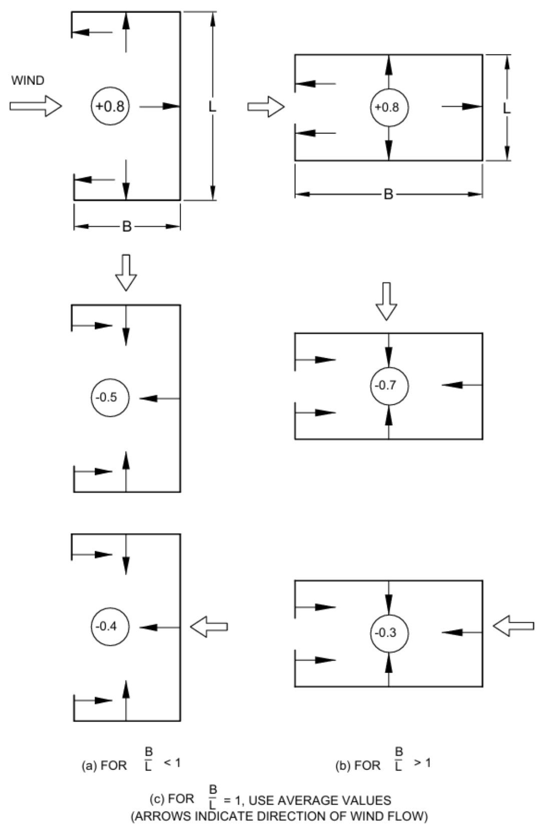


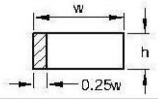
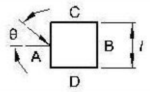
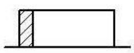
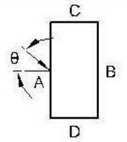
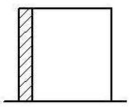
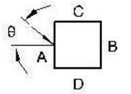
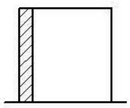
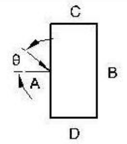
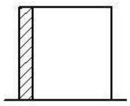
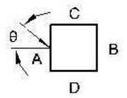
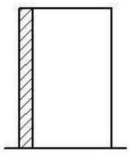
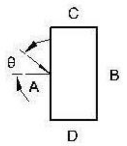
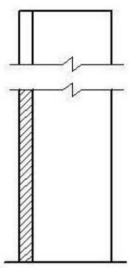
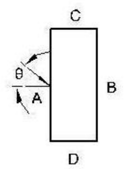
FIG. 2 VALUES OF INTERNAL PRESSURE COEFFICIENTS WITH RESPECT TO THE DIRECTION OF WIND FOR BUILDINGS WITH ONE SIDE OPENINGS

4.5.3.3 *External pressure coefficients*

4.5.3.3.1 *Walls*

The average external pressure coefficient for the walls of clad buildings of rectangular plan shall be as given in Table 8. In addition, local pressure concentration coefficients are also given.

Table 8 External Pressure Coefficients (C_{pe}) for Walls of Rectangular Clad Buildings
(*Clause* 4.5.3.3.1)

BUILDING HEIGHT RATIO	BUILDING PLAN RATIO	ELEVATION	PLAN	WIND ANGLE θ	C_{pe} FOR SURFACE A	B	C	D	LOCAL C_{pe} (hatched)
				Degrees					
$\frac{h}{w} \le \frac{1}{2}$	$1 < \frac{l}{w} \le \frac{3}{2}$			0 90	+ 0.7 - 0.5	- 0.2 - 0.5	- 0.5 + 0.7	- 0.5 - 0.2	- 0.8
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+ 0.7 - 0.5	- 0.25 - 0.5	- 0.6 + 0.7	- 0.6 - 0.1	- 1.0
$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2}$	$1 \le \frac{l}{w} \le \frac{3}{2}$			0 90	+ 0.7 - 0.6	- 0.25 - 0.6	- 0.6 + 0.7	- 0.6 - 0.25	- 1.1
	$\frac{3}{2} \le \frac{l}{w} < 4$			0 90	+ 0.7 - 0.5	- 0.3 - 0.5	- 0.7 + 0.7	- 0.7 - 0.1	- 1.1
$\frac{3}{2} < \frac{h}{w} < 6$	$1 < \frac{l}{w} \le \frac{3}{2}$			0 90	+ 0.8 - 0.8	- 0.25 - 0.8	- 0.8 + 0.8	- 0.8 - 0.25	- 1.2
	$\frac{3}{2} \le \frac{l}{w} < 4$			0 90	+ 0.7 - 0.5	- 0.4 - 0.5	- 0.7 + 0.8	- 0.7 - 0.1	- 1.2
$\frac{h}{w} \ge 6$	$\frac{l}{w} = \frac{3}{2}$			0 90	+ 0.95 - 0.8	- 1.85 - 0.8	- 0.9 + 0.9	- 0.9 - 0.85	- 1.25
	$\frac{l}{w} = 1.0$			0 90	+ 0.95 - 0.7	- 1.25 - 0.7	- 0.7 + 0.95	- 0.7 - 1.25	- 1.25
	$\frac{l}{w} = 2$			0 90	+ 0.85 - 0.75	- 0.75 - 0.75	- 0.75 + 0.85	- 0.75 - 0.75	- 1.25

NOTE : h is the height to eaves or parapet, l is the greater horizontal dimension of a building and w is the lesser horizontal dimension of a building.

4.5.3.3.2 *Pitched, hipped and monoslope roofs of clad buildings*

The average external pressure coefficients and pressure concentration coefficients for pitched roofs of rectangular clad building shall be as given in Table 9. Where no pressure concentration coefficients are given, the average coefficients shall apply. The pressure coefficients on the underside of any overhanging roof shall be taken in accordance with **4.5.3.3.5**.

NOTES

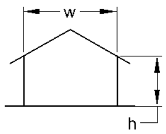
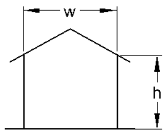
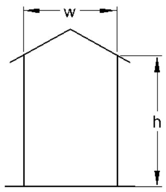
1 The pressure concentration shall be assumed to act outward (suction pressure) at the ridges, eaves, cornices and 90° corners of roofs.

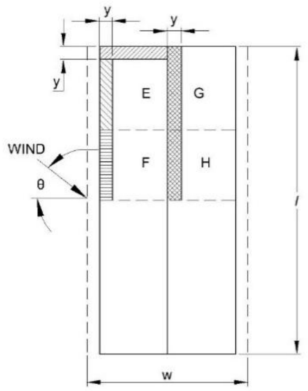
2 The pressure concentration shall not be included with the net external pressure when computing overall loads.

3 For hipped roofs, pressure coefficients (including local values) may be taken on all the four slopes, as appropriate from Table 9, and be reduced by 20 percent for the hip slope.

Table 9 External Pressure Coefficients (C_{pe}) for Pitched Roofs of Rectangular Clad Buildings

(*Clause* 4.5.3.3.2)

BUILDING HEIGHT RATIO		ROOF ANGLE α	WIND ANGLE θ 0°		WIND ANGLE θ 90°		LOCAL COEFFICIENTS			
			EF	*GH*	*EG*	*FH*				
		Degrees								
$\frac{h}{w} \leq \frac{1}{2}$		0	- 0.8	- 0.4	- 0.8	- 0.4	- 2.0	- 2.0	- 2.0	-----
		5	- 0.9	- 0.4	- 0.8	- 0.4	- 1.4	- 1.2	- 1.2	- 1.0
		10	- 1.2	- 0.4	- 0.8	- 0.6	- 1.4	- 1.4		- 1.2
		20	- 0.4	- 0.4	- 0.7	- 0.6	- 1.0			- 1.2
		30	0	- 0.4	- 0.7	- 0.6	- 0.8			- 1.1
		45	+ 0.3	- 0.5	- 0.7	- 0.6				- 1.1
		60	+ 0.7	- 0.6	- 0.7	- 0.6				- 1.1
$\frac{1}{2} \leq \frac{h}{w} \leq \frac{3}{2}$		0	- 0.8	- 0.6	- 1.0	- 0.6	- 2.0	- 2.0	- 2.0	-----
		5	- 0.9	- 0.6	- 0.9	- 0.6	- 2.0	- 2.0	- 1.5	- 1.0
		10	- 1.1	- 0.6	- 0.8	- 0.6	- 2.0	- 2.0	- 1.5	- 1.2
		20	- 0.7	- 0.5	- 0.8	- 0.6	- 1.5	- 1.5	- 1.5	- 1.0
		30	- 0.2	- 0.5	- 0.8	- 0.8	- 1.0			- 1.0
		45	+ 0.2	- 0.5	- 0.8	- 0.8				
		60	+ 0.6	- 0.5	- 0.8	- 0.8				
$\frac{3}{2} < \frac{h}{w} < 6$		0	- 0.7	- 0.6	- 0.9	- 0.7	- 2.0	- 2.0	- 2.0	-----
		5	- 0.7	- 0.6	- 0.8	- 0.8	- 2.0	- 2.0	- 1.5	- 1.0
		10	- 0.7	- 0.6	- 0.8	- 0.8	- 2.0	- 2.0	- 1.5	- 1.2
		20	- 0.8	- 0.6	- 0.8	- 0.8	- 1.5	- 1.5	- 1.5	- 1.2
		30	- 1.0	- 0.5	- 0.8	- 0.7	- 1.5			
		40	- 0.2	- 0.5	- 0.8	- 0.7	- 1.0			
		45	+ 0.2	- 0.5	- 0.8	- 0.7				
		60	+ 0.5	- 0.5	- 0.8	- 0.7				



KEY PLAN

y = *h* or 0.15 *w*
Whichever is the lesser.

NOTE - 1 *h* is the height to eaves or parapet and *w* is the lesser horizontal dimension of a building.

NOTE - 2 Where no local coefficients are given , the overall coefficient apply.

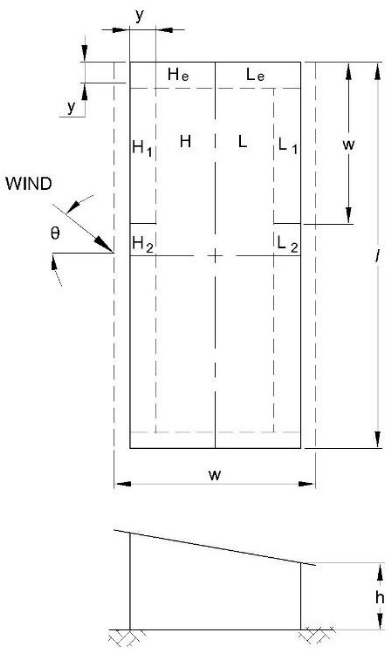
NOTE - 3 For hipped roofs the local coefficient for the hip ridge may be conservatively taken as the the appropriate ridge value.

NOTE - 4 *w* and *l* are dimensions between the walls excluding overhangs.

For mono slope roofs of rectangular clad buildings, the average pressure coefficient and pressure concentration coefficient for mono slope (lean-to) roofs of rectangular clad buildings shall be as given in Table 10.

Table 10 External Pressure Coefficients (C_{pe}) for Monoslope Roofs for Rectangular Clad Buildings $\frac{h}{w} < 2$

(*Clause* 4.5.3.3.2)



$y = h$ or $0.15\ w$
Whichever is the lesser.

NOTE :- Area *H* and area *L* refer to the whole quadrant.

ROOF ANGLE α	WIND ANGLE θ										LOCAL COEFFICIENTS (C_{pe})					
	0°		45°		90°		135°		180°							
	H	*L*	*H*	*L*	*H & L* *	*H & L* **	*H*	*L*	*H*	*L*	H_1	H_2	L_1	L_2	H_e	L_e
Degrees																
5	- 1.0	- 0.5	- 1.0	- 0.9	- 1.0	- 0.5	- 0.9	- 1.0	- 0.5	- 1.0	- 2.0	- 1.5	- 2.0	- 1.5	- 2.0	- 2.0
10	- 1.0	- 0.5	- 1.0	- 0.8	- 1.0	- 0.5	- 0.8	- 1.0	- 0.4	- 1.0	- 2.0	- 1.5	- 2.0	- 1.5	- 2.0	- 2.0
15	- 0.9	- 0.5	- 1.0	- 0.7	- 1.0	- 0.5	- 0.6	- 1.0	- 0.3	- 1.0	- 1.8	- 0.9	- 1.8	- 1.4	- 2.0	- 2.0
20	- 0.8	- 0.5	- 1.0	- 0.6	- 0.9	- 0.5	- 0.5	- 1.0	- 0.2	- 1.0	- 1.8	- 0.8	- 1.8	- 1.4	- 2.0	- 2.0
25	- 0.7	- 0.5	- 1.0	- 0.6	- 0.8	- 0.5	- 0.3	- 0.9	- 0.1	- 0.9	- 1.8	- 0.7	- 0.9	- 0.9	- 2.0	- 2.0
30	- 0.5	- 0.5	- 1.0	- 0.6	- 0.8	- 0.5	- 0.1	- 0.6	0	- 0.6	- 1.8	- 0.5	- 0.5	- 0.5	- 2.0	- 2.0

* Applied to length *w / 2* from wind-ward end.

** Applies to remainder

NOTE :- 1 *h* is the height of eaves at lower side, *l* is the greater horizontal dimensions of a building and *w* is the lesser horizontal dimension of a building.

NOTE :- 2 l and w are overall length and width including overhangs,

4.5.3.3.3 *Canopy roofs with* (1/4 < *h*/*w* < 1 *and* 1 < *L*/*w*< 3)

The pressure coefficients are given in Tables 11 and 12 separately for mono pitch and double pitch canopy roofs such as open-air parking garages, shelter areas, outdoor areas, railway platforms, stadiums and theatres. The coefficients take into account of the combined effect of the wind exerted on and under the roof for all wind directions; the resultant is to be taken normal to the canopy. Where the local coefficients overlap, the greater of the two given values should be taken. However, the effect of partial closures of one side and or both sides, such as those due to trains, buses and stored materials shall be foreseen and taken into account.

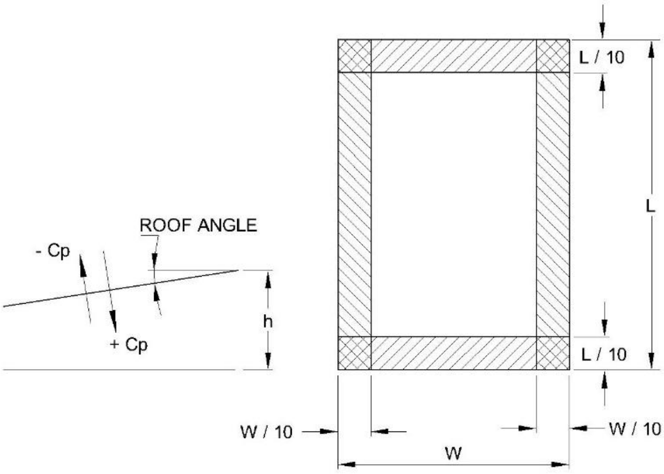
The solidity ratio, ϕ, is equal to the area of obstructions under the canopy divided by the gross area under the canopy, both areas normal to the wind direction. ϕ = 0 represents a canopy with no obstructions underneath. ϕ = 1 represents the canopy fully blocked with contents to the downwind eaves. Values of C_p for intermediate solidities may be linearly interpolated between these two extremes, and apply upwind of the position of maximum blockage only. For downwind of the position of maximum blockage, the coefficients for ϕ = 0 may be used.

In addition to the forces due to the pressures normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia and to friction over the surface of the canopy. For any wind direction, only the greater of these two forces need to be taken into account. Fascia loads should be calculated on the area of the surface facing the wind, using a force coefficient of 1.3. Frictional drag should be calculated using the coefficients given in **4.5.4.1**.

NOTE – Tables 13 to 18 may be used to get internal and external pressure coefficients for pitches and troughed free roofs for some specific cases for which aspect ratios and roof slopes have been specified. However, while using Tables 13 to 18 any significant departure from it should be investigated carefully. No increase shall be made for local effects except as indicated.

Table 11 Pressure Coefficients for Monoslope Free Roofs

(*Clause* 4.5.3.3.3)



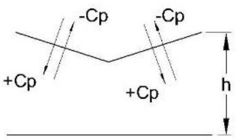
ROOF ANGLE (Degrees) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS			
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS		
0 5 10 15 20 25 30	All values of Φ	+ 0.2 + 0.4 + 0.5 + 0.7 + 0.8 + 1.0 + 1.2	+ 0.5 + 0.8 + 1.2 + 1.4 + 1.7 + 2.0 + 2.2	+ 1.8 + 2.1 + 2.4 + 2.7 + 2.9 + 3.1 + 3.2	+ 1.1 + 1.3 + 1.6 + 1.8 + 2.1 + 2.3 + 2.4
0	Φ = 0	- 0.5	- 0.6	- 1.3	- 1.4
	Φ = 1	- 1.0	- 1.2	- 1.8	- 1.9
5	Φ = 0	- 0.7	- 1.1	- 1.7	- 1.8
	Φ = 1	- 1.1	- 1.6	- 2.2	- 2.3
10	Φ = 0	- 0.9	- 1.5	- 2.0	- 2.1
	Φ = 1	- 1.3	- 2.1	- 2.6	- 2.7
15	Φ = 0	- 1.1	- 1.8	- 2.4	- 2.5
	Φ = 1	- 1.4	- 2.3	- 2.9	- 3.0
20	Φ = 0	- 1.3	- 2.2	- 2.8	- 2.9
	Φ = 1	- 1.5	- 2.6	- 3.1	- 3.2
25	Φ = 0	- 1.6	- 2.6	- 3.2	- 3.2
	Φ = 1	- 1.7	- 2.8	- 3.5	- 3.5
30	Φ = 0	- 1.8	- 3.0	- 3.8	- 3.6
	Φ = 1	- 1.8	- 3.0	- 3.8	- 3.6

NOTE 1: For monopitch canopies the centre of pressure should be taken to act at 0.3 w from the windward edge.

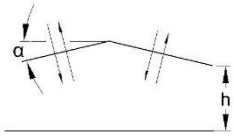
NOTE 2: W and L are overall width and length including overhangs,

Table 12 Pressure Coefficients for Free Standing Double Sloped Roofs

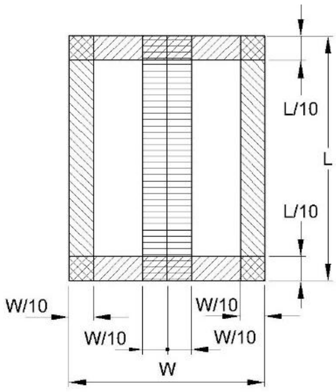
(*Clause* 4.5.3.3.3)



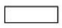
- ve ROOF ANGLE



+ ve ROOF ANGLE



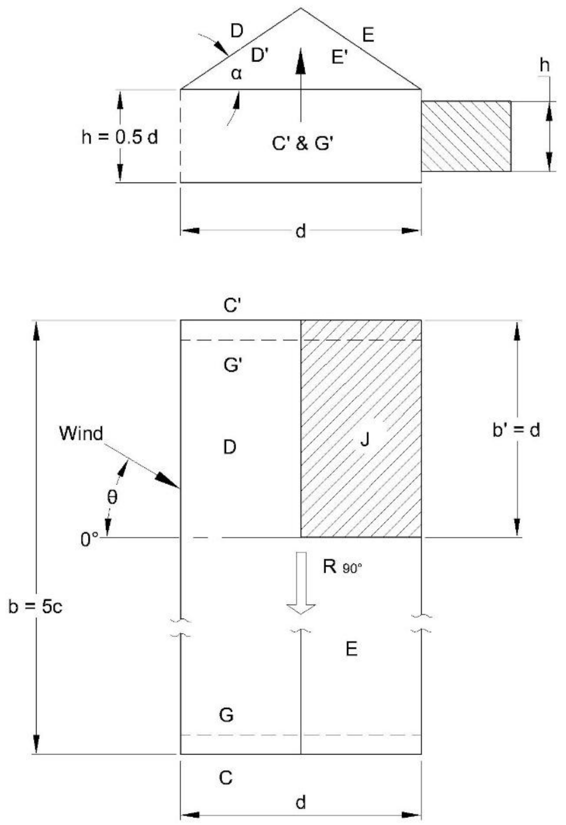
KEY PLAN

ROOF ANGLE (Degrees) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS				
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS			
						
- 20	All values of Φ	+ 0.7	+ 0.8	+ 1.6	+ 0.6	+ 1.7
- 15		+ 0.5	+ 0.6	+ 1.5	+ 0.7	+ 1.4
- 10		+ 0.4	+ 0.6	+ 1.4	+ 0.8	+ 1.1
- 5		+ 0.3	+ 0.5	+ 1.5	+ 0.8	+ 0.8
+ 5		+ 0.3	+ 0.6	+ 1.8	+ 1.3	+ 0.4
+ 10		+ 0.4	+ 0.7	+ 1.8	+ 1.4	+ 0.4
+ 15		+ 0.4	+ 0.9	+ 1.9	+ 1.4	+ 0.4
+ 20		+ 0.6	+ 1.1	+ 1.9	+ 1.5	+ 0.4
+ 25		+ 0.7	+ 1.2	+ 1.9	+ 1.6	+ 0.5
+ 30		+ 0.9	+ 1.3	+ 1.9	+ 1.6	+ 0.7
- 20	Φ = 0	- 0.7	- 0.9	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.9	- 1.2	- 1.7	- 1.9	- 1.2
- 15	Φ = 0	- 0.6	- 0.8	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.8	- 1.1	- 1.7	- 1.9	- 1.2
- 10	Φ = 0	- 0.6	- 0.8	- 1.3	- 1.5	- 0.6
	Φ = 1	- 0.8	- 1.1	- 1.7	- 1.9	- 1.3
- 5	Φ = 0	- 0.5	- 0.7	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.8	- 1.5	- 1.7	- 1.9	- 1.4
+ 5	Φ = 0	- 0.6	- 0.6	- 1.4	- 1.4	- 1.1
	Φ = 1	- 0.9	- 1.3	- 1.8	- 1.8	- 2.1
+ 10	Φ = 0	- 0.7	- 0.7	- 1.5	- 1.4	- 1.4
	Φ = 1	- 1.1	- 1.4	- 2.0	- 1.8	- 2.4
+ 15	Φ = 0	- 0.8	- 0.9	- 1.7	- 1.4	- 1.8
	Φ = 1	- 1.2	- 1.5	- 2.2	- 1.9	- 2.8
+ 20	Φ = 0	- 0.9	- 1.2	- 1.8	- 1.4	- 2.0
	Φ = 1	- 1.3	- 1.7	- 2.3	- 1.9	- 3.0
+ 25	Φ = 0	- 1.0	- 1.4	- 1.9	- 1.4	- 2.0
	Φ = 1	- 1.4	- 1.9	- 2.4	- 2.1	- 3.0
+ 30	Φ = 0	- 1.0	- 1.4	- 1.9	- 1.4	- 2.0
	Φ = 1	- 1.4	- 2.1	- 2.6	- 2.2	- 3.0

Each slope of a duopitch canopy should be able to withstand forces using both the maximum and the minimum coefficients, and the whole canopy should be able to support forces using one slope at the maximum coefficient with the other slope at the minimum coefficient. For duoptich canopies the centre of pressure should be taken to act at the centre of each slope.

Note :- W and L are overall width and length including overhangs

Table 13 Pressure Coefficients (Top and Bottom) for Pitched Roofs, Roof Slope, $\alpha = 30^\circ$
(*Clause* 4.5.3.3.3)

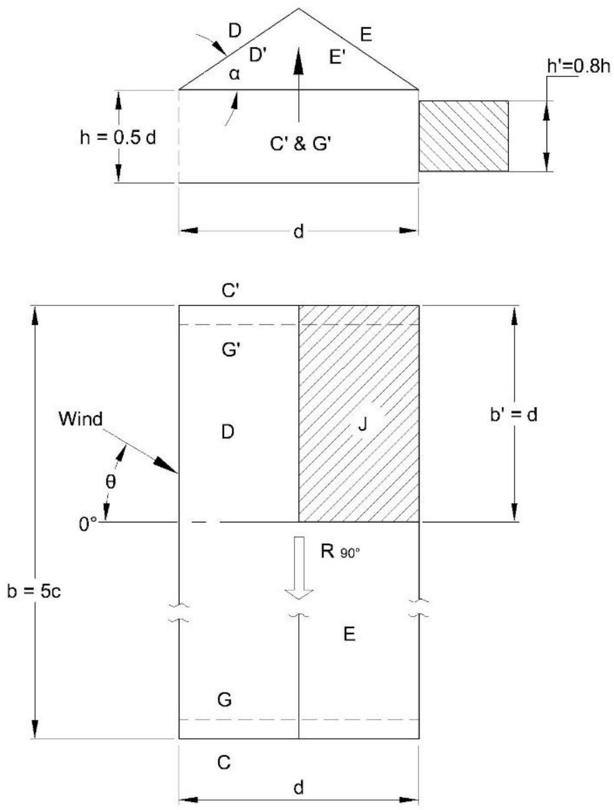


Roof slope α = 30°
θ = 0°- 45°, *D, D', E, E'* full length.
θ = 90°, *D, D', E, E'* part length.
b', thereafter C_p = 0

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D'*	*E*	*E'*	END SURFACES			
					C	*C'*	*G*	*G'*
0°	+ 0.6	- 1.0	- 0.5	- 0.9				
45°	+ 0.1	- 0.3	- 0.6	- 0.3				
90°	- 0.3	- 0.4	- 0.3	- 0.4	- 0.3	- 0.8	- 0.3	- 0.4
For all value of θ	For J : C_p Top = 1.0, C_p bottom = - 0.2 Tangentially acting friction : $R_{90^\circ} = 0.05\, p_d\, bd$							

Table 14 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 30^\circ$ With Effects of Train or Stored Materials

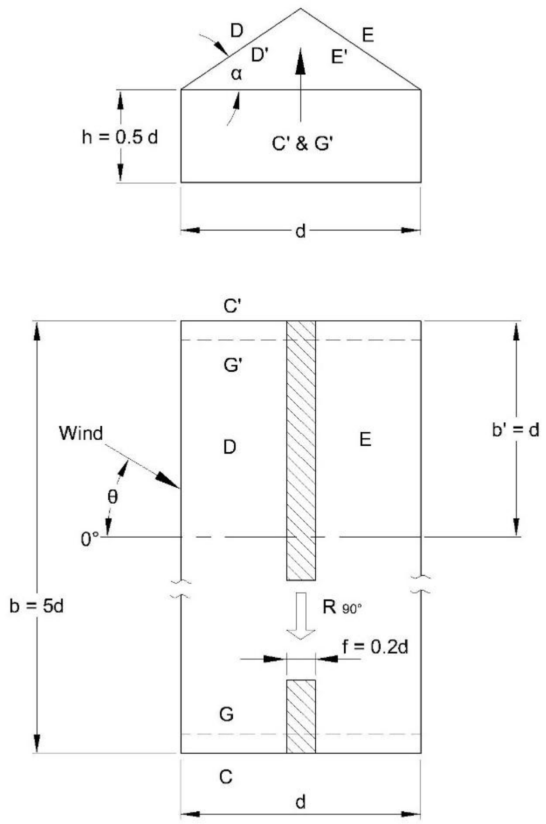
(*Clause* 4.5.3.3.3)



Roof slope α = 30°
Effects of trains or stored materials.
θ = 0°- 45°, or 135° - 180°, *D, D', E, E'* full length.
θ = 90°, *D, D', E, E'* part length.
b', thereafter C_p = 0

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D′*	*E*	*E′*	END SURFACES			
					C	*C′*	*G*	*G′*
0°	+ 0.1	+ 0.8	- 0.7	+ 0.9				
45°	- 0.1	+ 0.5	- 0.8	+ 0.5				
90°	- 0.4	- 0.5	- 0.4	- 0.5	- 0.3	+ 0.8	+ 0.3	- 0.4
180°	- 0.3	- 0.6	+ 0.4	- 0.6				
For all value of θ	For J : C_p Top = -1.5, C_p bottom = 0.5 Tangentially acting friction : $R_{90^\circ} = 0.05\, p_d\, bd$							

Table 15 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 10^\circ$
(*Clause* 4.5.3.3.3)

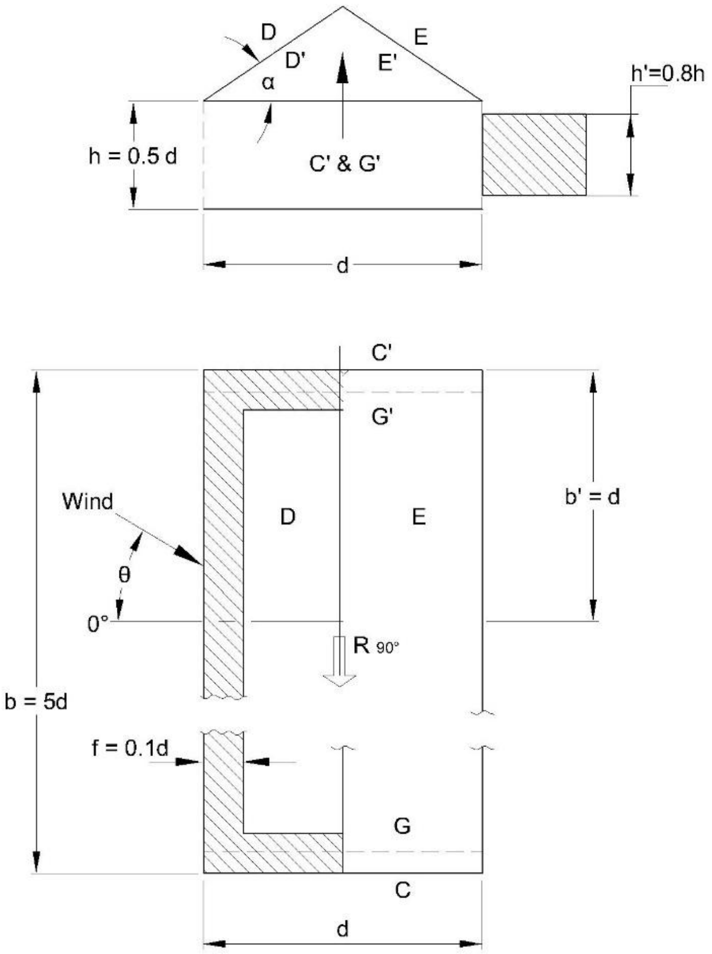


Roof slope α = 10°
θ = 0°- 45°, *D, D′, E, E′* full length.
θ = 90°, *D, D′, E, E′* part length.
b′, thereafter C_p = 0

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D′*	*E*	*E′*	END SURFACES			
					C	*C′*	*G*	*G′*
0°	- 1.0	+ 0.3	- 0.5	+ 0.2				
45°	- 0.3	+ 0.1	- 0.3	+ 0.1				
90°	- 0.3	0	- 0.3	0	- 0.4	+ 0.8	+ 0.3	- 0.6
For all value of θ	For *f* : C_p Top = - 1.0, C_p bottom = 0.4 Tangentially acting friction : $R_{90^\circ} = 0.1\, p_d\, bd$							

Table 16 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 10^\circ$ With Effects of Train or Stored Materials

(*Clause* 4.5.3.3.3)

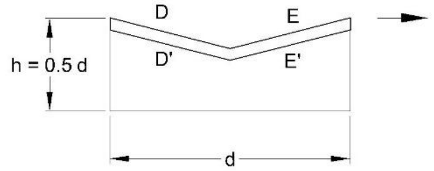


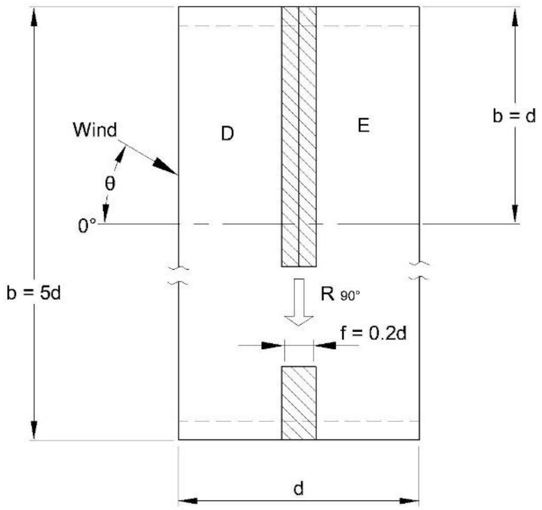
Roof slope α = 10°
Effects of trains or stored materials.
θ = 0°- 45°, or 135° - 180°, *D, D', E, E'* full length.
θ = 90°, *D, D', E, E'* part length.
b', thereafter C_p = 0

θ	PRESSURE COEFFICIENTS, C_p							
					END SURFACES			
	D	*D′*	*E*	*E′*	*C*	*C′*	*G*	*G′*
0°	- 1.3	+ 0.8	- 0.6	0.7	- 0.4	+ 0.8	+ 0.3	- 0.6
45°	- 0.5	+ 0.4	- 0.3	+ 0.3				
90°	- 0.3	0	- 0.3	0				
180°	- 0.4	- 0.3	- 0.6	- 0.3				
For all value of θ	For *f* : C_p Top = -1.6, C_p bottom = - 0.9 Tangentially acting friction : $R_{90^\circ} = 0.1\,p_d\,bd$							

Table 17 Pressure Coefficients for Troughed Free Roofs, $\alpha = 10^\circ$

(*Clause* 4.5.3.3.3)





Roof slope $\alpha = 10^\circ$

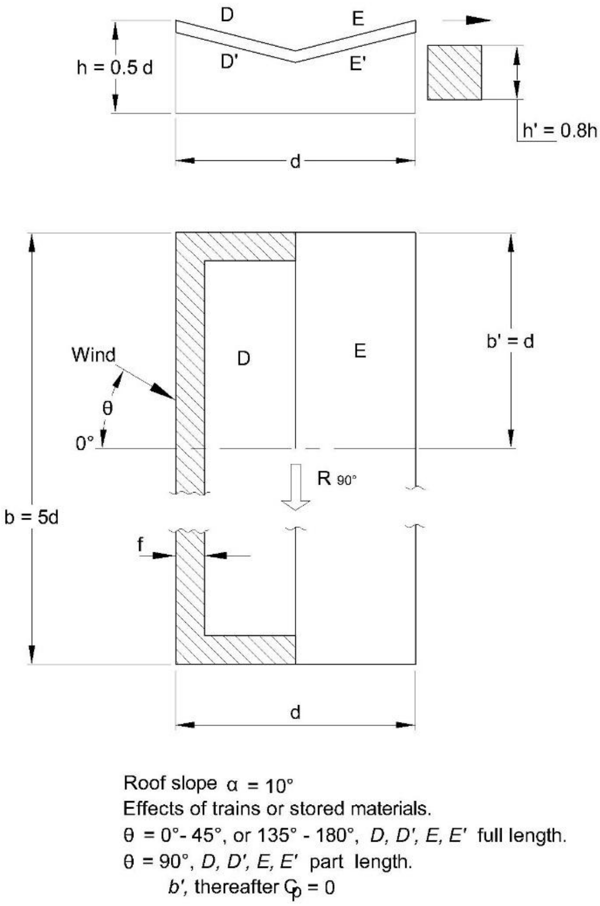
$\theta = 0^\circ - 45^\circ$, *D, D′, E, E′* full length.

$\theta = 90^\circ$, *D, D′, E, E′* part length.

b′, thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p			
	D	*D′*	*E*	*E′*
0°	+ 0.3	- 0.7	+ 0.2	- 0.9
45°	0	- 0.2	+ 0.1	- 0.3
90°	- 0.1	0.1	- 0.1	+ 0.1
For all value of θ	For *f* : C_p Top = 0.4, C_p bottom = - 1.5 Tangentially acting friction : $R_{90^\circ} = 0.1\, p_d\, bd$			

Table 18 Pressure Coefficients (Top and Bottom) for Troughed Free Roofs, $\alpha = 10^\circ$ With Effects of Train or Stored Material
(*Clause* 4.5.3.3.3)

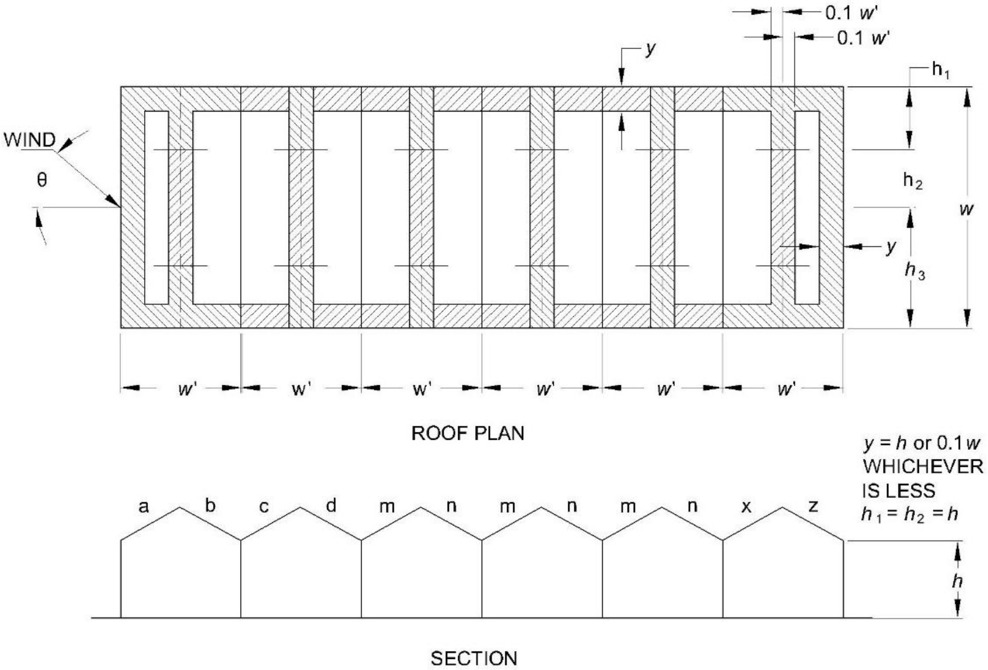


θ	PRESSURE COEFFICIENTS, C_p			
	D	*D'*	*E*	*E'*
0°	- 0.7	+ 0.8	- 0.6	+ 0.6
45°	- 0.4	+ 0.3	- 0.2	+ 0.2
90°	- 0.1	+ 0.1	- 0.1	+ 0.1
180°	- 0.4	- 0.2	- 0.6	- 0.3
For all value of θ	For *f* : C_p Top = -1.1, C_p bottom = 0.9 Tangentially acting friction : $R_{90^\circ} = 0.1\, p_d\, bd$			

4.5.3.3.4 *Pitched and saw-tooth roofs of multi-span buildings*

For pitched and saw-tooth roofs of multi-span buildings, the external average pressure coefficients shall be as given in Tables 19 and 20, respectively provided that all the spans shall be equal and the height to the eaves shall not exceed the span.

Table 19 External Pressure Coefficients (C_{pe}) for Pitched Roofs of Multi-span Buildings (All Spans Equal) with $h < w'$
(*Clause* 4.5.3.3.4)

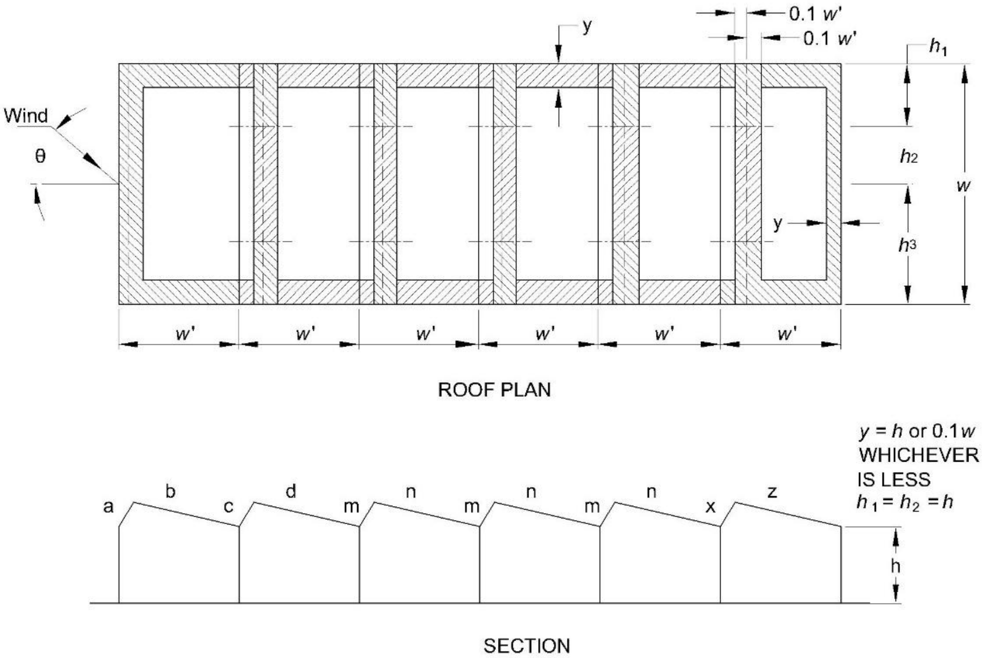


ROOF ANGLE	WIND ANGLE	FIRST SPAN		FIRST INTERME - DIATE SPAN		OTHER INTERME - DIATE SPANS		END SPAN		LOCAL COEFFICIENT	
α Degrees	θ Degrees	a	b	c	d	m	n	x	z	(hatch ╲)	(hatch ╱)
5	0	- 0.9	- 0.6	- 0.4	- 0.3	- 0.3	- 0.3	- 0.3	- 0.3		
10		- 1.1	- 0.6	- 0.4	- 0.3	- 0.3	- 0.3	- 0.3	- 0.4		
20		- 0.7	- 0.6	- 0.4	- 0.3	- 0.3	- 0.3	- 0.3	- 0.5	- 2.0	- 1.5
30		- 0.2	- 0.6	- 0.4	- 0.3	- 0.2	- 0.3	- 0.2	- 0.5		
45		+ 0.3	- 0.6	- 0.6	- 0.4	- 0.2	- 0.4	- 0.2	- 0.5		
DISTANCE											
ROOF ANGLE α DEGREES		WIND ANGLE θ DEGREES		h_1		h_2		h_3			
UP TO 45		90		- 0.8		- 0.6		- 0.2			

Frictional drag : When wind angle θ = 0°, horizontal forces due to frictional drag are allowed for in the above values, and
When wind angle θ = 90˚, allow for frictional drag in accordance with 4.5.4.1

NOTE – Evidence on these buildings is fragmentary and any departure from the cases given should be investigated separately.

Table 20 External Pressure Coefficients (C_{pe}) for Saw Tooth Roofs of Multi-span Buildings (All Spans Equal) with $h < w'$
(*Clause* 4.5.3.3.4)



WIND ANGLE	FIRST SPAN		FIRST INTERME - DIATE SPAN		OTHER INTERME - DIATE SPANS		END SPAN		LOCAL COEFFICIENT	
θ Degrees	a	b	c	d	m	n	x	z	(hatched zone)	(hatched zone)
0	+ 0.6	- 0.7	- 0.7	- 0.4	- 0.3	- 0.2	- 0.1	- 0.3		
180	- 0.5	- 0.3	- 0.3	- 0.3	- 0.4	- 0.6	- 0.6	- 0.1	- 2.0	- 1.5

DISTANCE

WIND ANGLE θ DEGREES	h_1	h_2	h_3
90	- 0.8	- 0.6	- 0.2
270	Similar to 90°, h_1, h_2, h_3, are needed to be reckoned from the windword edge in the same order		

Frictional drag : When wing angle θ = 0°, horizontal forces due to frictional drag are allowed for in the above values, and

When wind angle θ = 90˚, allow for frictional drag in accordance with 4.5.4.1

NOTE - Evidence on these buildings is fragmentary and any departure from the cases given should be investigated separately.

4.5.3.3.5 *Pressure coefficients on overhangs from roofs*

The pressure coefficients on the top overhanging portion of the roofs shall be taken to be the same as that of the nearest top portion of the non-overhanging portion of the roofs. The pressure coefficients for the underside surface of the overhanging portions shall be taken as follows and shall be taken as positive, if the overhanging portion is on the windward side:

1) 1.25, if the overhanging slopes downwards;
2) 1.00, if the overhanging is horizontal; and
3) 0.75, if the overhanging slopes upwards.

For overhanging portions on sides other than windward side, the average pressure coefficients on adjoining walls may be used.

4.5.3.3.6 *Curved roofs*

For curved roofs the external pressure coefficients shall be as given in Table 21. Allowance for local effects shall be made in accordance with Table 9. Two values of C_2 have been given for elevated curved roofs. Both the load cases have to be analyzed, and critical load effects are to be considered in design.

4.5.3.3.7 *Cylindrical structures*

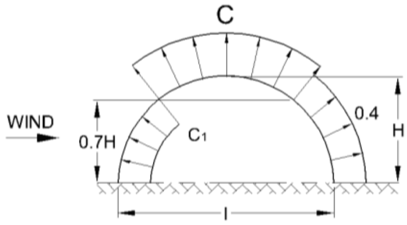
For the purpose of calculating the wind pressure distribution around a cylindrical structure of circular cross section, the value of external pressure coefficients given in Table 22 may be used, provided that the Reynolds number is more than 10 000. They may be used for wind blowing normal to the axis of cylinders having axis normal to the ground plane and cylinders having their axis parallel to the ground plane. '*h'* is height of a vertical cylinder or length of a horizontal cylinder. Where there is a free flow of air around both ends, *h* is to be taken as half the length when calculating *h/D* ratio.

In the calculation of resultant load on the periphery of the cylinder, the value of C_{pi} shall be taken into account. For open ended cylinders, C_{pi} shall be taken as follows:

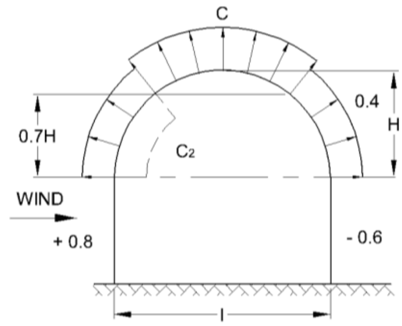
a) - 0.8, where *h*/*D* is more than or equal to 0.3; and

b) - 0.5, where *h*/*D* is less than 0.3.

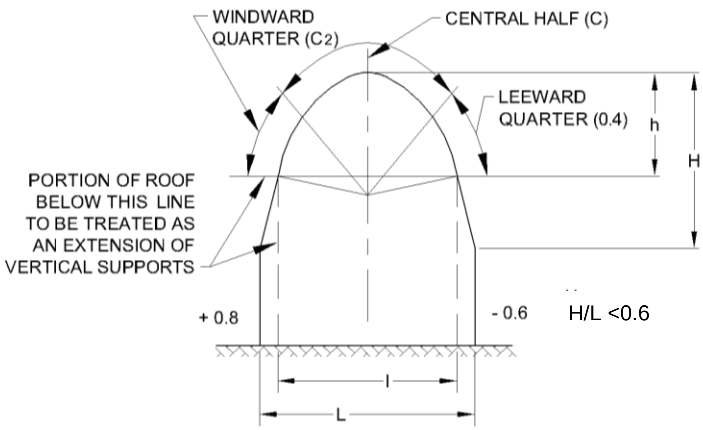
Table 21 External Pressure Coefficients (C_{pe}) for Curved Roofs
(*Clause* 4.5.3.3.6)



a) ROOF SPRINGING FROM GROUND LEVEL



b) ROOF ON ELEVATED STRUCTURE



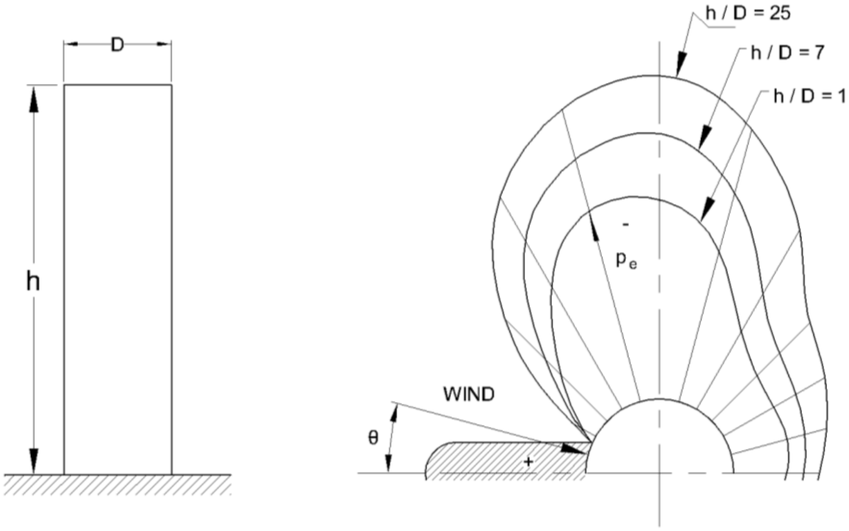
c) DOUBLE CURVED ROOFS

VALUES OF C, C_1 and C_2

H/l	C	C_1	C_2	C_2
0.1	- 0.8	+ 0.1	- 0.8	+ 0.05
0.2	- 0.9	+ 0.3	- 0.7	+ 0.1
0.3	- 1.0	+ 0.4	- 0.3	+ 0.15
0.4	- 1.1	+ 0.6	+ 0.4	-
0.5	- 1.2	+ 0.7	+ 0.7	-

Note :- When the wind is blowing normal to gable ends, C_{pe} may be taken as equal to - 0.7 for the full width of the roof over a length of $l/2$ from the gable ends and - 0.5 for the remaining portion.

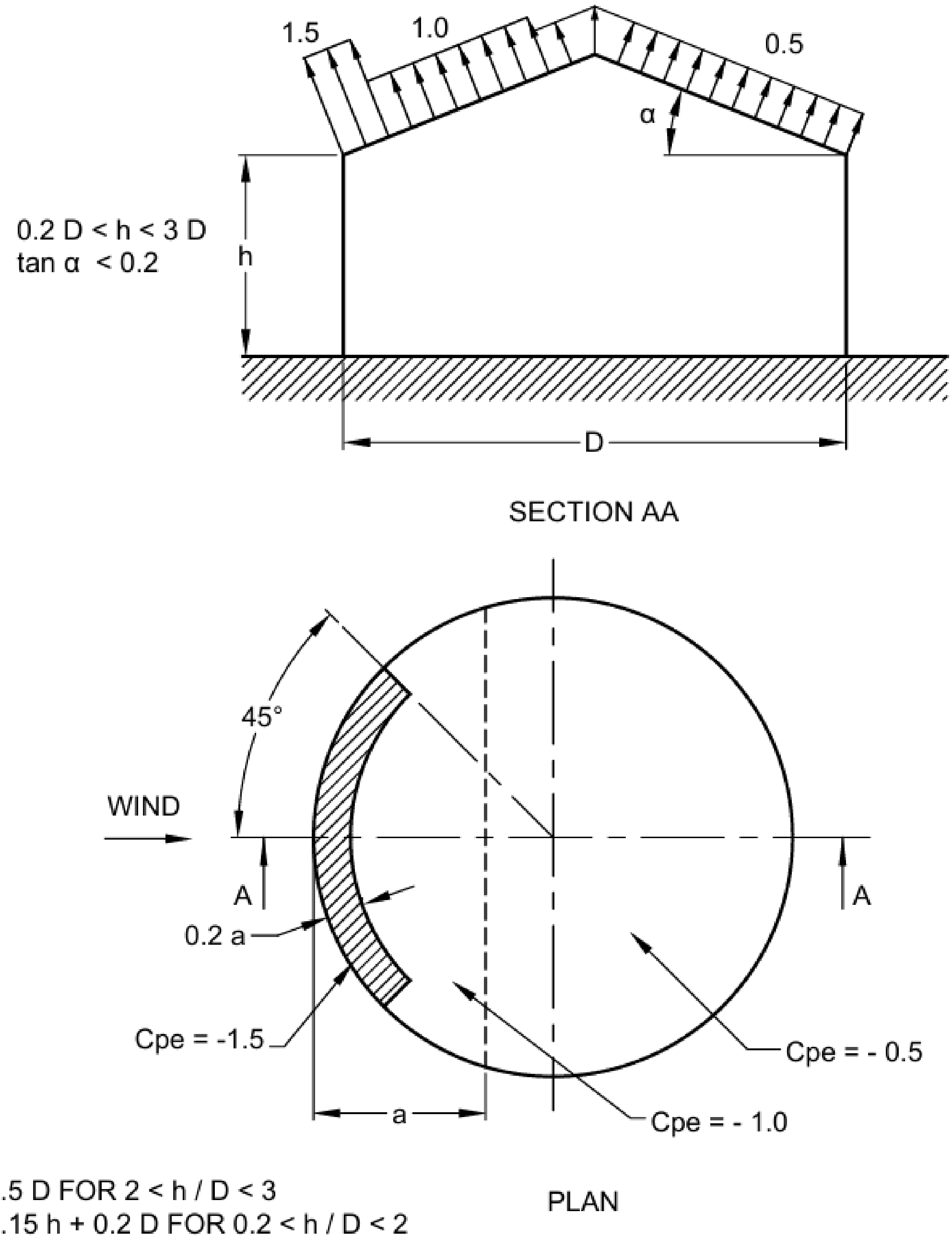
Table 22 External Pressure Coefficients (C_{pe}) around Cylindrical Structures
(*Clause* 4.5.3.3.7)



POSITION OF PERIPHERY, θ IN DEGREES	PRESSURE COEFFICIENTS C_{pe}		
	h / *D* = 25	*h* / *D* = 7	*h* / *D* = 1
0	1.0	1.0	1.0
15	0.8	0.8	0.8
30	0.1	0.1	0.1
45	- 0.9	- 0.8	- 0.7
60	- 1.9	- 1.7	- 1.2
75	- 2.5	- 2.2	- 1.6
90	- 2.6	- 2.2	- 1.7
105	- 1.9	- 1.7	- 1.2
120	- 0.9	- 0.8	- 0.7
135	- 0.7	- 0.6	- 0.5
150	- 0.6	- 0.5	- 0.4
165	- 0.6	- 0.5	- 0.4
180	- 0.6	- 0.5	- 0.4

4.5.3.3.8 *Roofs and bottoms of cylindrical elevated structures*

The external pressure coefficients for roofs and bottoms of cylindrical elevated structures shall be as given in Table 23. Alternately, the pressure distribution given in Fig. 3 can be used together with the force coefficients given in Table 28 for the cylindrical portion.

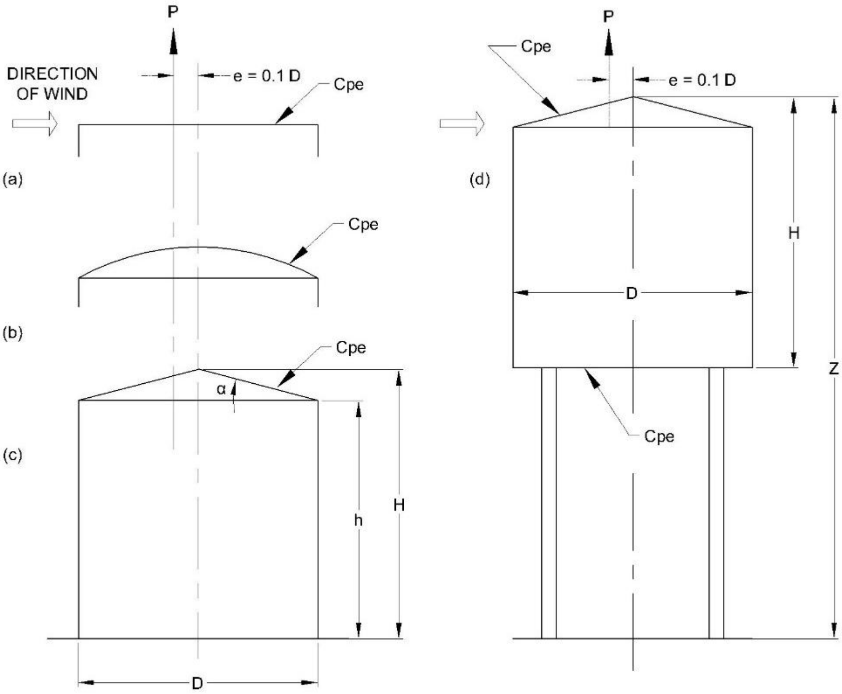


0.5 D FOR 2 < h / D < 3
0.15 h + 0.2 D FOR 0.2 < h / D < 2

(For force coefficient corresponding to shell portion)
(*See* Table 28)

FIG. 3 EXTERNAL PRESSURE COEFFICIENTS ON THE UPPER ROOF SURFACE OF CYLINDRICAL STRUCTURES STANDING ON THE GROUND

Table 23 External Pressure Coefficients (C_{pe}) for Roofs and Bottoms of Cylindrical Structures
(*Clause* 4.5.3.3.8)



COEFFICIENTS OF EXTERNAL PRESSURE C_{pe}				
STRUCTURE ACCORDING TO SHAPE				
a, b and c		d		
H / D	ROOF	(z / H) - 1	ROOF	BOTTOM
0.5	- 0.65	1.00	- 0.75	- 0.8
1.00	- 1.00	1.25	- 0.75	- 0.7
2.00	- 1.00	1.50	- 0.75	- 0.6

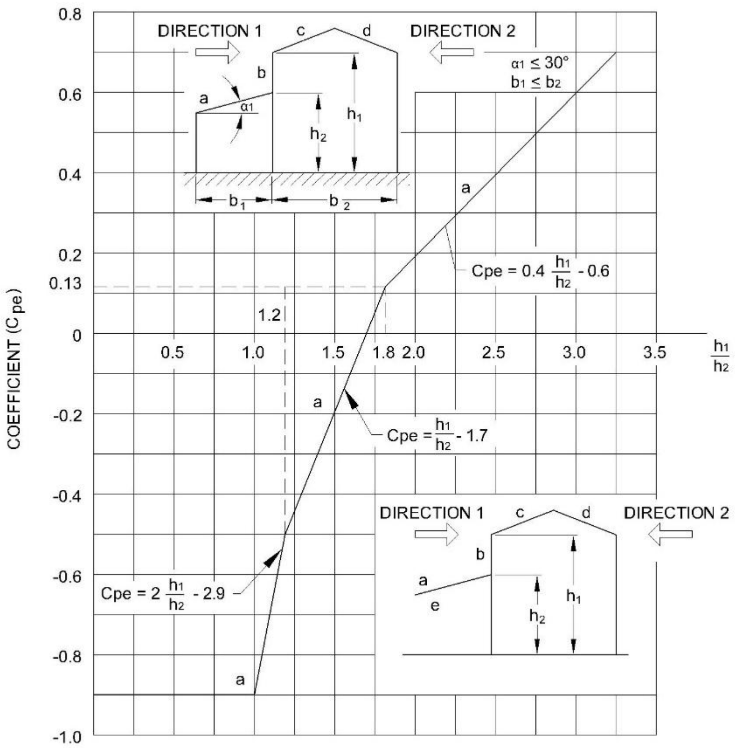
Total force acting on the roof of the structure, $P = 0.785\ D^2\ (C_{pi} - C_{pe})\ p_d$

The resultant of *P* lies eccentrically, e = 0.1D

4.5.3.3.9 *Combined roofs*

The average external pressure coefficients for combined roofs are shown in Table 24.

Table 24 External Pressure Coefficients (C_{pe}) for Combined Roofs
(*Clause* 4.5.3.3.9)

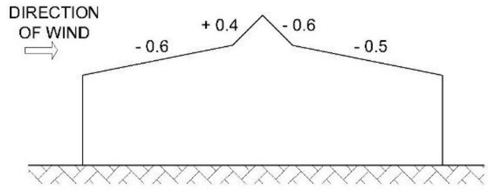


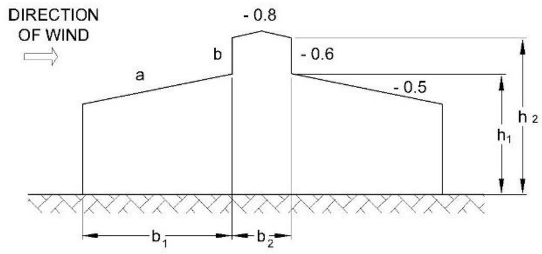
VALUES OF COEFFICIENTS (C_{pe})		
PORTION	DIRECTION 1	DIRECTION 2
a	FROM THE DIAGRAM	- 0.4
b	$C_{pe} = -0.5,\ \frac{h_1}{h_2} \le 1.5$ $C_{pe} = +0.7;\ \frac{h_1}{h_2} > 1.5$	
c and d	See Table 9	
e	See Clause 4.5.3.3	

4.5.3.3.10 *Roofs with skylight*

The average external pressure coefficients for roofs with skylight are shown in Table 25.

Table 25 External Pressure Coefficients (C_{pe}) for Roofs with a Sky Light
[*Clause* 4.5.3.3.10]





b) ROOFS WITH A SKY LIGHT

VALUES OF COEFFICIENTS (C_{pe})			
PORTION	$b_1 > b_2$		$b_1 \leq b_2$
	a	*b*	*a* and *b*
C_{pe}	- 0.6	+ 0.7	See Table for combined roofs

4.5.3.3.11 *Grandstands*

The pressure coefficients on the roof (top and bottom) and rear wall of a typical grandstand roof which is open on three sides are given in Table 26. The pressure coefficients are valid for a particular ratio of dimensions as specified in Table 24 but may be used for deviations up to 20 percent. In general, the maximum wind load occurs when the wind is blowing into the open front of the stand, causing positive pressure under the roof and negative pressure on the roof.

4.5.3.3.12 *Spheres*

The external pressure coefficients for spheres shall be as given in Table 27.

4.5.3.3.13 *Frames*

When taking wind loads on frames of clad buildings it is reasonable to assume that the pressures or suctions inside and outside the structure shall not be fully correlated.

Therefore, when taking the combined effect of wind loads on the frame, a reduction factor of K_c = 0.90 may be used over the building envelope when roof is subjected to pressure and internal pressure is suction, or *vice-versa*.

4.5.4 *Force Coefficients*

The value of force coefficients (C_f) apply to a building or structure as a whole, and when multiplied by the effective frontal area (A_e) of the building or structure and design wind pressure, p_d gives the total wind load (F) on that particular building or structure, expressed as:

$$F = C_f A_e p_d$$

where F is the force acting in a direction specified in the respective tables and C_f is the force coefficient for the building.

NOTES

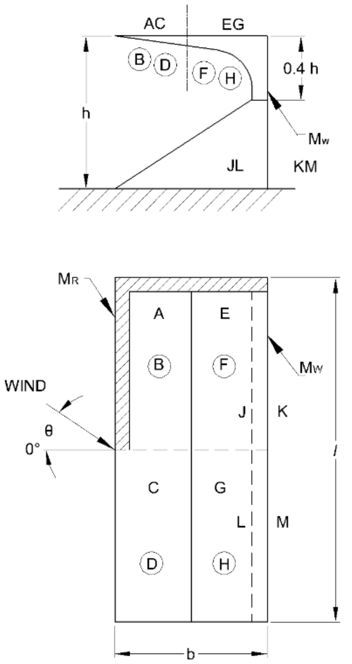
1 The value of the force coefficient differs for the wind acting on different faces of a building or structure. In order to determine the critical load, the total wind load should be calculated for each wind direction.

2 If surface design pressure varies with height, the surface area of the building/structure may be sub-divided so that specified pressures are taken over appropriate areas.

3 In tapered buildings/structures, the force coefficients shall be applied after sub-dividing the building/structure into suitable number of strips and the load on each strip calculated individually, taking the area of each strip as A_e.

4 For force coefficients for structures not covered herein, reference may be made to specialist literature on the subject or advice may be sought from specialist in the subject.

Table 26 Pressure Coefficients at Top and Bottom Roof of Grand Stands Open Three Sides (Roof Angle Up to 3°)
(*Clause* 4.5.3.3.11)

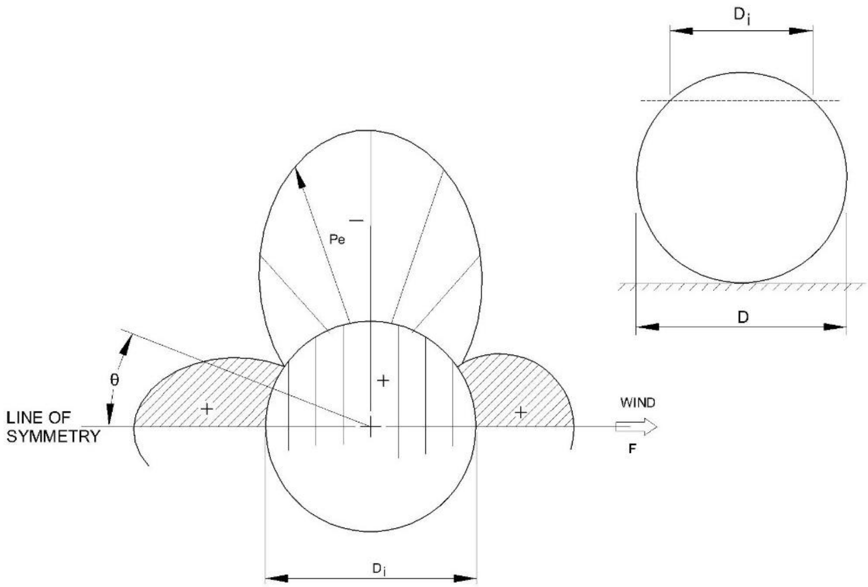


(Shaded area to scale)

FRONT AND BACK OF WALL				
θ	*J*	*K*	*L*	*M*
0°	+ 0.9	- 0.5	+ 0.9	- 0.5
45°	+ 0.8	- 0.6	+ 0.4	- 0.4
135°	- 1.1	+ 0.6	- 1.0	+ 0.4
180°	- 0.3	+ 0.9	- 0.3	+ 0.9
60°		*Mw* - C_p of K = - 1.0		
60°		*Mw* - C_p of J = + 1.0		

TOP AND BOTTOM ROOF								
θ	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*
0°	- 1.0	+ 0.9	- 1.0	+ 0.9	- 0.7	+ 0.9	+ 0.7	+ 0.9
45°	- 1.0	+ 0.7	- 0.7	+ 0.4	- 0.5	+ 0.8	- 0.5	+ 0.3
135°	- 0.4	- 1.1	- 0.7	- 1.0	- 0.9	- 1.1	- 0.9	- 1.0
180°	- 0.6	- 0.3	- 0.6	- 0.3	- 0.6	- 0.3	- 0.6	- 0.3
45°	'M_R' - C_p (top) = - 2.0							
45°	'M_R' - C_p (bottom) = + 1.0							

Table 27 External Pressure Coefficients (C_{pe}) Around Spherical Structures
(*Clause* 4.5.3.3.12)



POSITION OF PERIPHERY, θ IN DEGREES	C_{pe}
0	+ 1.0
15	+ 0.9
30	+ 0.5
45	- 0.1
60	- 0.7
75	- 1.1
90	- 1.2
105	- 1.0
120	- 0.6
135	- 0.2
150	+ 0.1
165	+ 0.3
180	+ 0.4

4.5.4.1 *Frictional drag*

In certain buildings of special shape, a force due to frictional drag shall be taken into account in addition to those loads specified in **4.5.3**. For rectangular clad buildings, this addition is necessary only where the ratio *d/h* or *d/b* is more than 4. The frictional drag force, *F',* in the direction of the wind is given by the following formulae:

If $h \le b$, $F' = C_f' (d - 4h)\, b\, p_d + C'_f (d - 4h)\, 2h\, p_d$, and

If $h > b$, $F' = C_f' (d - 4b)\, b\, p_d + C'_f (d - 4b)\, 2h\, p_d$

The first term in each case gives the drag on the roof and the second on the walls. The C_f' has the following values:

a) 0.01 for smooth surfaces without corrugations or ribs across the wind direction,
b) 0.02 for surfaces with corrugations across the wind direction, and

c) 0.04 for surfaces with ribs across the wind direction.

For other buildings, the frictional drag has been indicated, where necessary, in the tables of pressure coefficients and force coefficients.

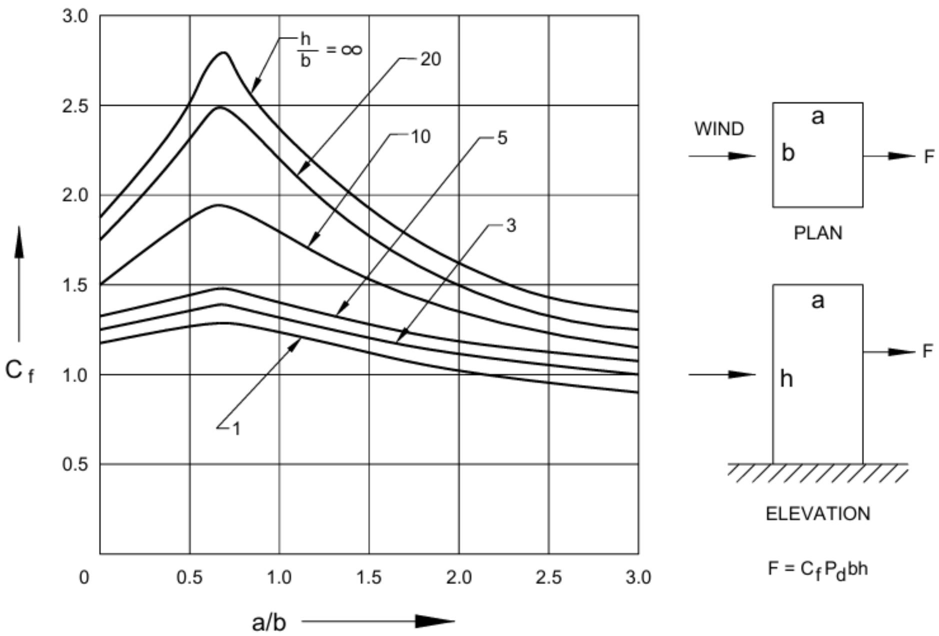
4.5.4.2 *Force coefficients for clad buildings*

4.5.4.2.1 *Clad buildings of uniform section*

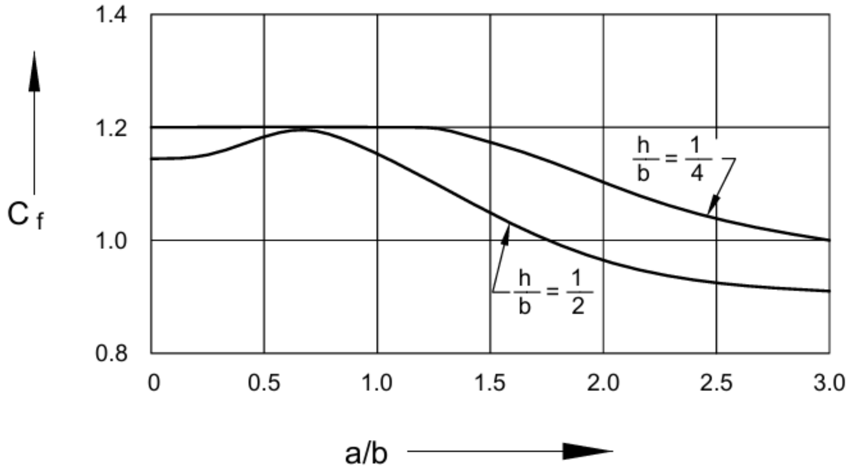
The overall force coefficients for rectangular clad buildings of uniform section with flat roofs in uniform flow shall be as given in Fig. 4 and for other clad buildings of uniform section (without projections, except where otherwise shown) shall be as given in Table 28.

NOTE – Structures that are in the supercritical flow regime, because of their size and design wind velocity, may need further calculation to ensure that the greatest loads do not occur at some wind speed below the maximum, when the flow will be sub-critical.

The coefficients are for buildings without projections, except where otherwise shown.
In Table 28, $\bar{V}_d b$ is used as an indication of the airflow regime.



4A VALUES OF C_f VERSUS a / b for h / b ≥ 1



4B VALUES OF C_f VERSUS a/b FOR h/b < 1

Fig. 4 FORCE COEFFICIENT FOR RECTANGULAR CLAD BUILDING IN UNIFORM FLOW

Table 28 Force Coefficients, C_f for Clad Buildings of Uniform Section (Acting in the Direction of Wind)
(*Clause* 4.5.4.2.1)

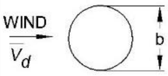
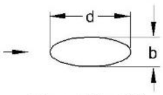
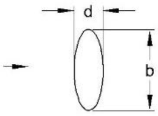
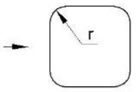
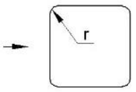
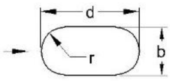
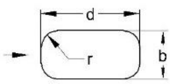
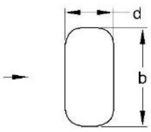
PLAN SHAPE	$\frac{\overline{V}_d\, b}{m^2/s}$	C_f FOR HEIGHT / BREADTH RATIO						
		UPTO 1/2	1	2	5	10	20	∞
 See also Appendix D	ALL SURFACES < 6 ROUGH or WITH PROJECTION ≥ 6	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	SMOOTH ≥ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
 Ellipse b/d = 1/2	< 10	0.5	0.5	0.5	0.5	0.6	0.6	0.7
	≥ 10	0.2	0.2	0.2	0.2	0.2	0.2	0.2
 Ellipse b/d = 2	< 8	0.8	0.8	0.9	1.0	1.1	1.3	1.7
	≥ 8	0.8	0.8	0.9	1.0	1.1	1.3	1.5
 b/d = 1 r/b = 1/3	< 4	0.6	0.6	0.6	0.7	0.8	0.8	1.0
	≥ 4	0.4	0.4	0.4	0.4	0.5	0.5	0.5
 b/d = 1 r/b = 1/6	< 10	0.7	0.8	0.8	0.9	1.0	1.0	1.3
	≥ 10	0.5	0.5	0.5	0.5	0.6	0.6	0.6
 b/d = 1/2 r/b = 1/2	< 3	0.3	0.3	0.3	0.3	0.3	0.3	0.4
	≥ 3	0.2	0.2	0.2	0.2	0.3	0.3	0.3
 b/d = 1/2 r/b = 1/6	All values	0.5	0.5	0.5	0.5	0.6	0.6	0.7
 b/d = 2 r/b = 1/12	All values	0.9	0.9	1.0	1.1	1.2	1.5	1.9

Table 28 — *Continued*

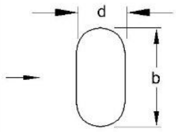
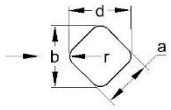
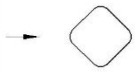
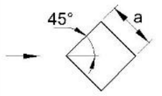
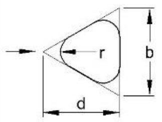
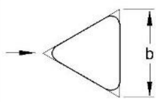
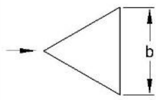
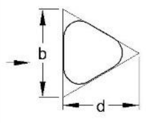
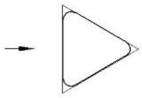
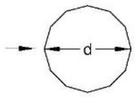
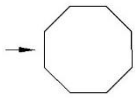
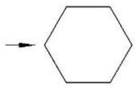
PLAN SHAPE	$\frac{\bar{V}_d b}{m^2/s}$	C_f FOR HEIGHT / BREADTH RATIO						
		UPTO 1/2	1	2	5	10	20	∞
 b/d = 2, r/b = 1/4	< 6	0.7	0.8	0.8	0.9	1.0	1.2	1.6
	≥ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
 r/a = 1/3	< 10	0.8	0.8	0.9	1.0	1.1	1.3	1.5
	≥ 10	0.5	0.5	0.5	0.5	0.5	0.6	0.6
 r/a = 1/12	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
 r/a = 1/48	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
 r/b = 1/4	< 11	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	≥ 11	0.4	0.4	0.4	0.4	0.5	0.5	0.5
 r/b = 1/12	All values	0.8	0.8	0.8	1.0	1.1	1.2	1.4
 r/b = 1/48	All values	0.7	0.7	0.8	0.9	1.0	1.1	1.3
 r/b = 1/4	< 8	0.7	0.7	0.8	0.9	1.0	1.1	1.3
	≤ 8	0.4	0.4	0.4	0.4	0.5	0.5	0.5

Table 28 — *Concluded*

PLAN SHAPE	$\bar{V}_d b$ m^2/s	C_f FOR HEIGHT / BREADTH RATIO						
		UPTO 1/2	1	2	5	10	20	∞
 $1/48 < r/b < 1/12$	All values	1.2	1.2	1.2	1.4	1.6	1.7	2.1
 12 SIDED POLYGON	< 12	0.7	0.7	0.8	0.9	1.0	1.1	1.3
	≥ 12	0.7	0.7	0.7	0.7	0.8	0.9	1.1
 OCTAGON	All values	1.0	1.0	1.1	1.2	1.2	1.3	1.4
 HEXAGON	All values	1.0	1.1	1.2	1.3	1.4	1.4	1.5

4.5.4.2.2 *Buildings of circular shapes*

Force coefficients for buildings of circular cross section shapes shall be as given in Table 28. However, more precise estimation of force coefficients for circular shapes of infinite length can be obtained from Fig. 5 taking into account the average height of surface roughness ε. When the length is finite the values obtained from Fig. 5 shall be reduced by the multiplication factor K (*see also* Table 31 and Annex G).

4.5.4.2.3 *Free standing walls and hoardings*

Force coefficients for free standing walls and hoardings shall be as given in Table 29.

To allow for oblique winds, the design shall also be checked for net pressure normal to the surface varying linearly from a maximum of 1.7 C_f at the windward edge to 0.44 C_f at the leeward edge.

The wind load on appurtenances and supports for hoardings shall be accounted for separately by using the appropriate net pressure coefficients. Allowance shall be made for shielding effects of one element on another.

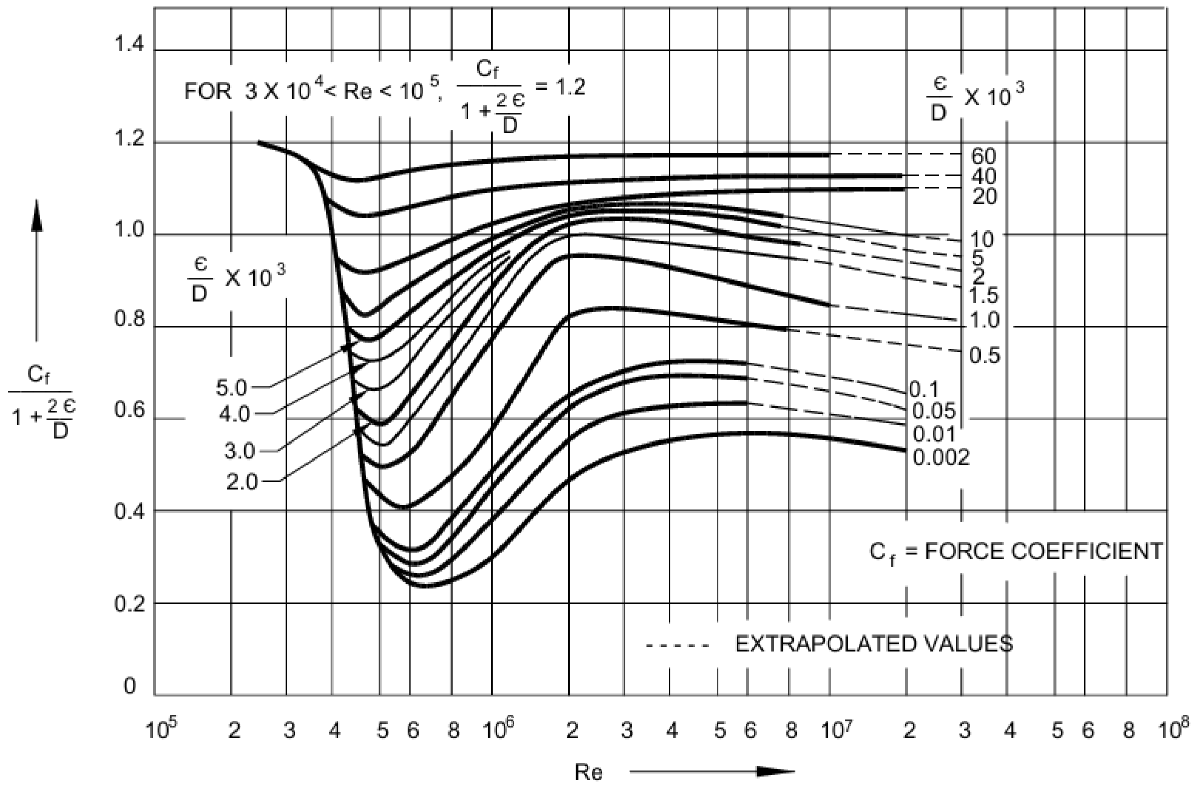
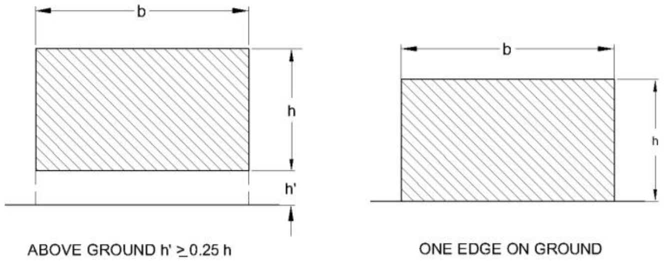


Fig. 5 VARIATION OF $\frac{C_f}{1+\frac{2\epsilon}{D}}$ WITH Re < 3 x 10^4 FOR CIRCULAR SECTIONS

4.5.4.2.4 *Solid circular shapes mounted on a surface*

The force coefficients for solid circular shapes mounted on a surface shall be as given in Table 30.

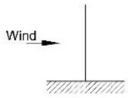
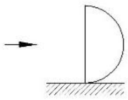
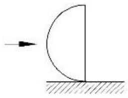
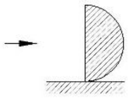
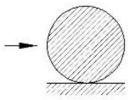
Table 29 Force Coefficients, C_f for Low Walls or Hoardings (<15m High)
[*Clause* 4.5.4.2.3]



FORCE COEFFICIENT

WIDTH TO HEIGHT RATIO, *b/h*		DRAG COEFFICIENT C_f
WALL ABOVE GROUND	WALL ON GROUND	
FROM 0.5 TO 6	FROM 1 TO 12	1.2
10	20	1.3
16	32	1.4
20	40	1.5
40	80	1.75
60	120	1.8
80 OR MORE	160 OR MORE	2.0

Table 30 Force Coefficients, C_f for Solid Shapes Mounted on a Surface
(*Clause* 4.5.4.2.4)

SIDE ELEVATION	DESCRIPTION OF SHAPE	C_f
	CIRCULAR DISC	1.2
	HEMISPHERICAL BOWL	1.4
	HEMISPHERICAL BOWL	0.4
	HEMISPHERICAL SOLID	1.2
	SPHERICAL SOLID	0.5 FOR V_z D < 7 0.2 FOR V_z D ≥ 7

4.5.4.3 *Force coefficients for unclad buildings*

4.5.4.3.1 This section applies to permanently unclad buildings and to frameworks of buildings while temporarily unclad. In the case of buildings whose surfaces are well-rounded, such as those with elliptic, circular or oval cross sections, the total force can be more at a wind speed much less than maximum due to transition in the nature of boundary layer on them. Although this phenomenon is well known in the case of circular cylinders, the same phenomenon exists in the case of many other well-rounded structures, and this possibility must be checked.

4.5.4.3.2 *Individual members*

a) The force coefficient given in Table 32 refers to members of infinite length. For members of finite length, the coefficients should be multiplied by a factor *K* that depends on the ratio *l*/*b* where *l* is the length of the member and *b* is the width across the direction of wind. Table 31 gives the required values of *K*. The following special cases must be noted while estimating *K*:

   1) when any member abuts on to a plate or wall in such a way that free flow of air around that end of the member is prevented, then the ratio of *l*/*b* shall be doubled for the purpose of determining *K*; and

   2) when both ends of a member are so obstructed, the ratio shall be taken as infinity for the purpose of determining *K*.

b) *Flat-sided members* – Force coefficients for wind normal to the longitudinal axis of flat-sided structural members shall be as given in Table 32.

The force coefficients are given for two mutually perpendicular directions relative to a reference axis on the structural member. They are denoted by C_{fn} and C_{ft} and give the forces normal and transverse, respectively to the reference plane as shown in Table 32.

Normal force, $F_n = (C_{fn}\ p_d\ K) / b$
Transverse force, $F_t = (C_{ft}\ p_d\ K) / b$

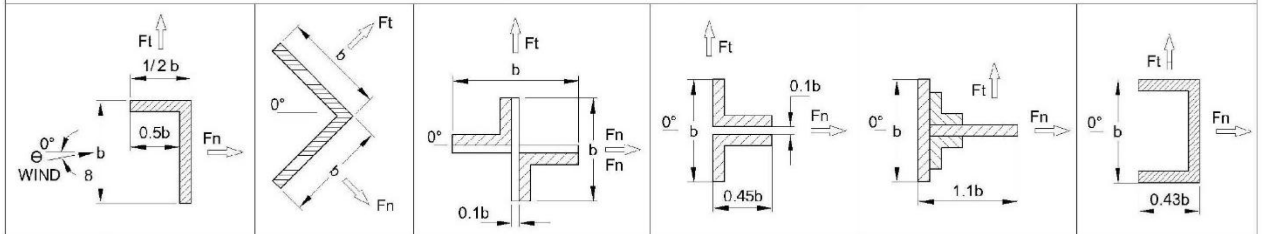
c) *Circular sections* – Force coefficients for members of circular section shall be as given in Table 28 (*see also* Annex G).

d) Force coefficients for wires and cables shall be as given in Table 33 according to the diameter (D), the design wind speed (V_d) and the surface roughness.

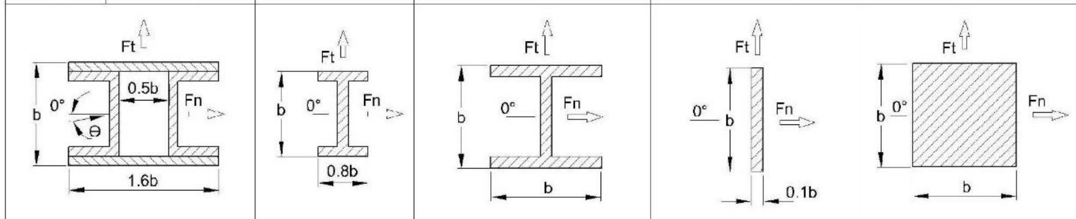
Table 31 Reduction Factor *K* for Individual Members
[*Clauses* 4.5.4.2.2 *and* 4.5.4.3.2(a)]

Sl No.	***l/b* or *l/D***	**2**	**5**	**10**	**20**	**40**	**50**	**100**	**∞**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	Circular cylinder, sub-critical flow	0.58	0.62	0.68	0.74	0.82	0.87	0.98	1.00
ii)	Circular cylinder, supercritical flow ($D\overline{V}_d \geq 6$ m^2/s)	0.80	0.80	0.82	0.90	0.98	0.99	1.00	1.00
iii)	For plate perpendicular to wind ($b\overline{V}_d \geq 6$ m^2/s)	0.62	0.66	0.69	0.81	0.87	0.90	0.95	1.00

Table 32 Force Coefficients, C_f for Individual Structural Members of Infinite Length
[*Clause* 4.5.4.3.2(b)]



θ DEGREE	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
0	+ 1.9	+ 0.95	+ 1.8	+ 1.8	+ 1.75	+ 0.1	+ 1.6	0	+ 2.0	0	+ 2.05	0
45	+ 1.8	+ 0.8	+ 2.1	+ 1.8	+ 0.85	+ 0.85	+ 1.5	- 0.1	+ 1.2	+ 0.9	+ 1.85	+ 0.6
90	+ 2.0	+ 1.7	- 1.9	- 1.0	+ 0.1	+ 1.75	- 0.95	+ 0.7	- 1.6	+ 2.15	0	+ 0.6
135	- 1.8	- 0.1	- 2.0	+ 0.3	- 0.75	+ 0.75	- 0.5	+ 1.05	- 1.1	+ 2.4	- 1.6	+ 0.4
180	- 2.0	+ 0.1	- 1.4	- 1.4	- 1.75	- 0.1	- 1.5	0	- 1.7	+ 2.1	- 1.8	0



θ DEGREE	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
0	+ 1.4	0	+ 2.05	0	+ 1.6	0	+ 2.0	0	+ 2.0	0
45	+ 1.2	+ 1.6	+ 1.95	+ 0.6	+ 1.5	+ 1.5	+ 1.8	+ 0.1	+ 1.55	+ 1.55
90	0	+ 2.2	+ 0.5	+ 0.9	0	+ 1.9	0	+ 0.1	0	+ 2.0

Table 33 Force Coefficients, C_f for Wires and Cables (l/D = 100)
[*Clause* 4.5.4.3.2(b)]

Sl No.	Flow Regime	Force Coefficient, C_f for			
		Smooth Surface	Moderately Smooth Wire (Galvanized or Painted)	Fine Stranded Cables	Thick Stranded Cables
(1)	(2)	(3)	(4)	(5)	(6)
i)	$D\bar{V}_d$ < 6 m²/s	1.2	1.2	1.2	1.3
ii)	$D\bar{V}_d$ ≥ 6 m²/s	0.5	0.7	0.9	1.1

4.5.4.3.3 *Single frames*

Force coefficients for a single frame having either,

a) all flat sided members; or

b) all circular members in which all the members of the frame have either,

   1) $D\bar{V}_d$ less than 6 m²/s; or

   2) $D\bar{V}_d$ more than or equal to 6 m²/s.

shall be as given in Table 31 according to the type of the member, the diameter (*D*), the design hourly mean wind speed ($\bar{V}_d$) and the solidity ratio (Φ).

Force coefficients for a single frame not complying with the above requirements shall be calculated as follows:

$$C_f = \gamma\, C_{f\,super} + (1-\gamma)\frac{A_{circ\,sub}}{A_{sub}} C_{f\,sub} + (1-\gamma)\frac{A_{flat}}{A_{sub}} C_{f\,flat}$$

where

$C_{f\ super}$ = force coefficient for the supercritical circular members as given in Table 34 or Annex G,

$C_{f\ sub}$ = force coefficient for subcritical circular members as given in Table 34 or Annex G,

$C_{f\ flat}$ = force coefficient for the flat sided members as given in Table 34,

$A_{circ\ sub}$ = effective area of subcritical circular members,

A_{flat} = effective area of flat-sided members,

A_{sub} = $A_{\text{circ sub}} + A_{\text{flat}}$,

γ = $\dfrac{\text{Area of the frame in a supercritical flow}}{A_e}$, and

A_e = Effective frontal area

Table 34 Force Coefficients for Single Frames
(*Clause* 4.5.4.3.3)

Sl No.	Solidity Ratio, Φ	Force Coefficient, C_t For		
		Flat Sided Members	Circular Sections	
			Subcritical Flow ($D\bar{V}_d < 6$ m²/s)	Super Critical Flow ($D\bar{V}_d \geq 6$ m²/s)
(1)	(2)	(3)	(4)	(5)
i)	0.1	1.9	1.2	0.7
ii)	0.2	1.8	1.2	0.8
iii)	0.3	1.7	1.2	0.8
iv)	0.4	1.7	1.1	0.8
v)	0.5	1.6	1.1	0.8
vi)	0.75	1.6	1.5	1.4
vii)	1.00	2.0	2.0	2.0

NOTE ─ Linear interpolation between the values is permitted.

4.5.4.3.4 *Multiple frame buildings*

This Section applies to structures having two or more parallel frames where the windward frames may have a shielding effect upon the frames to leeward side. The windward frame and any unshielded parts of other frames shall be calculated in accordance with **4.5.4.3.3**, but the wind load on the parts of frames that are sheltered should be multiplied by a shielding factor which is dependent upon the solidity ratio of the windward frame, the types of members comprising the frame and the spacing ratio of the frames. The values of the shielding factors are given in Table 35.

Where there are more than two frames of similar geometry and spacing, the wind load on the third and subsequent frames should be taken as equal to that on the second frame. The loads on the various frames shall be added to obtain total load on the structure.

a) The frame spacing ratio is equal to the centre to centre distance between the frames, beams or girders divided by the least overall dimension of the frames, beam or girder measured in a direction normal to the direction of wind. For triangular framed structures or rectangular framed structures diagonal to the wind, the spacing ratio should be calculated from the mean distance between the frames in the direction of the wind.

b) Effective solidity ratio, Φ_e:

$\Phi_e = \Phi$ for flat-sided members.

Φ_e is to be obtained from Fig. 6 for members of circular cross sections.

Table 35 Shielding Factor, η for Multiple Frames
(*Clause* 4.5.4.3.4)

Sl No.	Effective Solidity Ratio, Φ_e	Frame Spacing Ratio				
		< 0.5	1.0	2.0	4.0	> 8.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	0	1.0	1.0	1.0	1.0	1.0
ii)	0.1	0.9	1.0	1.0	1.0	1.0
iii)	0.2	0.8	0.9	1.0	1.0	1.0
iv)	0.3	0.7	0.8	1.0	1.0	1.0
v)	0.4	0.6	0.7	1.0	1.0	1.0
vi)	0.5	0.5	0.6	0.9	1.0	1.0
vii)	0.7	0.3	0.6	0.8	0.9	1.0
viii)	1.0	0.3	0.6	0.6	0.8	1.0

NOTE ─ Linear interpolation between the values is permitted.

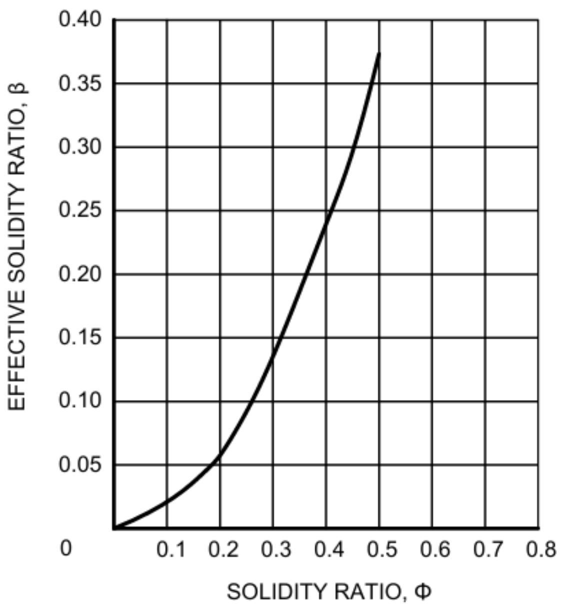


FIG. 6 EFFECTIVE SOLIDITY RATIO FOR CIRCULAR SECTION MEMBERS

4.5.4.3.5 *Lattice towers*

a) Force coefficient for lattice towers of square or equilateral triangle section with flat sided members for wind blowing against any face shall be as given in Table 36.

b) For square lattice towers with flat-sided members the maximum load, which occurs when the wind blows into a corner, shall be taken as 1.2 times the load for the wind blowing against a face.

c) For equilateral triangle lattice towers with flat-sided members, the load may be assumed to be constant for any inclination of wind to a face.

d) Force coefficients for lattice towers of square section with circular members, all in the same flow regime, may be as given in Table 37.

e) Force coefficients for lattice towers of equilateral-triangle section with circular members all in the same flow regime may be as given in Table 38.

4.5.4.3.6 *Tower appurtenances*

The wind loading on tower appurtenances, such as ladders, conduits, lights, elevators, etc, shall be calculated using appropriate net pressure coefficients for these elements. Allowance may be made for shielding effect from other elements.

Table 36 Overall Force Coefficients for Towers Composed of Flat Sided Members
[*Clause* 4.5.4.3.5(a)]

Sl No.	**Solidity Ratio, Φ**	**Force Coefficient For**	
		Square Towers	Equilateral Triangular Towers
(1)	(2)	(3)	(4)
i)	< 0.1	3.8	3.1
ii)	0.2	3.3	2.7
iii)	0.3	2.8	2.3
iv)	0.4	2.3	1.9
v)	0.5	2.1	1.5

Table 37 Overall Force Coefficients for Square Towers Composed of Circular Members
[*Clause* 4.5.4.3.5 (d)]

Sl No.	Solidity Ratio of Front Face Φ	Force Coefficient For			
		Subcritical Flow ($D\bar{V}_d < 6$ m^2/s)		Super Critical Flow ($D\bar{V}_d \geq 6$ m^2/s)	
		Onto Face	Onto Corner	Onto Face	Onto Corner
(1)	(2)	(3)	(4)	(5)	(6)
i)	< 0.05	2.4	2.5	1.1	1.2
ii)	0.1	2.2	2.3	1.2	1.3
iii)	0.2	1.9	2.1	1.3	1.6
iv)	0.3	1.7	1.9	1.4	1.6
v)	0.4	1.6	1.9	1.4	1.6
vi)	0.5	1.4	1.9	1.4	1.6

Table 38 Overall Force Coefficients for Equilateral Triangular Towers Composed of Circular Members
[Clause 4.5.4.3.5(e)]

Sl No.	Solidity Ratio of Front Face Φ	Force Coefficient For	
		Subcritical flow ($D\bar{V}_d < 6$ m^2/s)	Super critical flow ($D\bar{V}_d \geq 6$ m^2/s)
		All wind directions	All wind directions
(1)	(2)	(3)	(4)
i)	< 0.05	1.8	0.8
ii)	0.1	1.7	0.8
iii)	0.2	1.6	1.1
iv)	0.3	1.5	1.1
v)	0.4	1.5	1.1
vi)	0.5	1.4	1.2

4.6 Interference Effects

4.6.1 *General*

Wind interference is caused by modification in the wind characteristics produced by the obstruction caused by an object or a building/structure in the path of the wind. If such wind strikes another structure, the wind pressures usually get enhanced, though there can also be some shielding effect between two very closely spaced buildings/structures. The actual phenomenon is too complex to justify generalization of the wind forces/pressures produced due to interference which can only be ascertained by detailed wind tunnel/CFD studies. However, some guidance can be provided for the purpose of preliminary design. To account for the effect of interference, a wind interference factor (IF) has been introduced as a multiplying factor to be applied to the design wind pressure/force. Interference effects can be more significant for tall buildings. The interference factor is defined as the ratio between the enhanced pressure/force in the grouped configuration to the corresponding pressure/force in isolated configuration. Since the values of IF can vary considerably based on building geometry and location, the given values of IF are a kind of median values and are meant only for preliminary design estimates. The designer is advised that for assigning values of IF for final design particularly for tall buildings, specialist literature be consulted or a wind tunnel study carried out.

4.6.2 *Roof of Low Rise Buildings*

Maximum increase in wind force on the roof due to interference from similar buildings in case of closely spaced low rise buildings with flat roofs may be up to 25 percent for c/c distance (*x*) between the buildings of 5 times the dimension (*b*) of the interfering building normal to the direction of wind (*see* Fig. 7). Interference effect beyond 20*b* may be considered to be negligible. For intermediate spacing linear interpolation may be used.

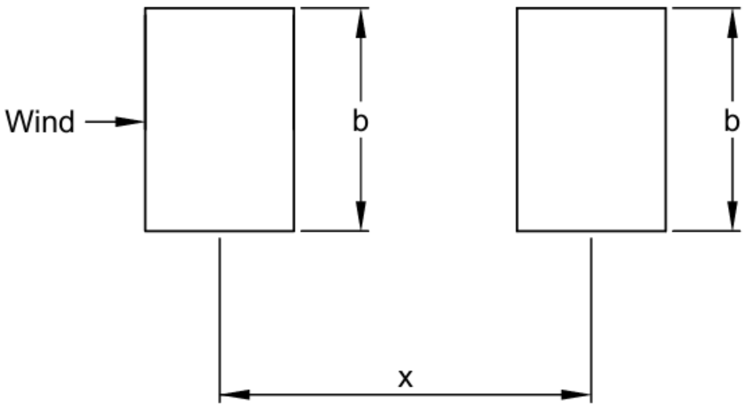


FIG. 7 LOW-RISE BUILDING IN TANDEM CAUSING INTERFERENCE EFFECT

4.6.3 *Tall Buildings*

Based on studies on tall rectangular buildings, Fig. 8 gives various zones of interference. The interference factor (IF), which needs to be considered as a multiplication factor for wind loads corresponding to isolated building, may be assumed as follows for preliminary estimate of the wind loads under interference caused by another interfering tall building of same or more height located at different zones Z1 to Z4 as shown in Fig. 8:

Zone	Z1	Z2	Z3	Z4
IF	1.35	1.25	1.15	1.07

The interference effect due to buildings of height less than one-third of the height of the building under consideration may be considered to be negligible while for interference from a building of intermediate height, linear interpolation may be used between one-third and full height.

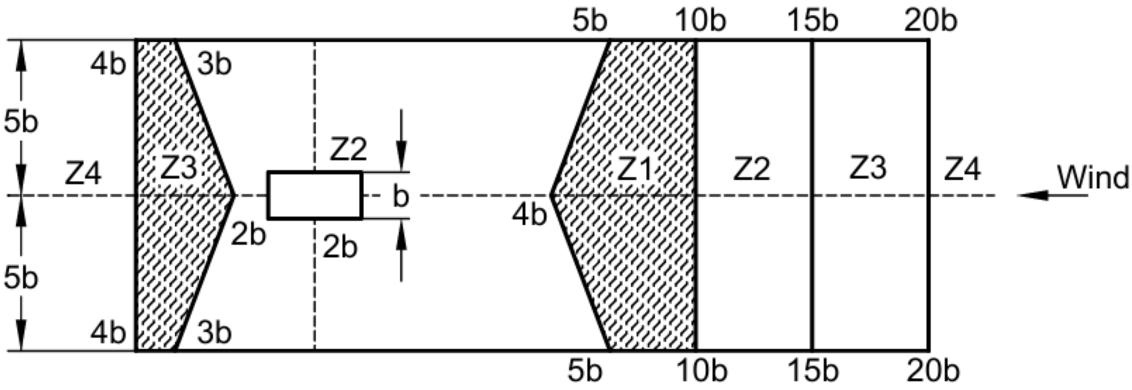


FIG. 8 INTERFERENCE ZONES FOR TALL RECTANGULAR BUILDINGS OF SAME OR GREATER HEIGHT

4.7 Dynamic Effects

4.7.1 *General*

Flexible slender structures and structural elements shall be investigated to ascertain the importance of wind induced oscillations or excitations in along wind and across wind directions.

In general, the following guidelines may be used for examining the problems of wind-induced oscillations:

a) Buildings and closed structures with a height to minimum lateral dimension ratio of more than about 5.0, or

b) Buildings and structures whose natural frequency in the first mode is less than 1.0 Hz.

Any building or structure which satisfies either of the above two criteria shall be examined for dynamic effects of wind.

NOTES

1 The fundamental time period (T), in second, may either be established by experimental observations on similar buildings or calculated by any rational method of analysis. In the absence of such data, T may be determined as follows for multi-storied buildings:

a) For moment resistant frames without bracings or shear walls resisting the lateral loads,

$T = 0.1\,n$

where
n = number of storeys, including basement storeys.

b) For all others,

$$T = \frac{0.09H}{\sqrt{d}}$$

where

H = total height of the main structures of the building, in m; and

d = maximum base dimension of building in meters in a direction parallel to the applied wind force.

2 If preliminary studies indicate that wind-induced oscillations are likely to be significant, investigations should be pursued with the aid of analytical methods or if necessary, by means of wind tunnel tests on models.

3 Across-wind motions may be due to lateral gustiness of the wind, unsteady wake flow (for example, vortex shedding), negative aerodynamic damping or due to a combination of these effects. These cross-wind motions may become critical in the design of tall buildings / structures.

4 Motions in the direction of wind (also known as buffeting) are caused by fluctuating wind force associated with gust. The excitation depends on gust energy available at the resonant frequency.

5 The eddies shed from an upstream body may intensify motion in the direction of the wind and may also affect cross-wind motion.

6 The designer should also be aware of the following three forms of wind-induced motion which are characterized by increasing amplitude of oscillation with the increase of wind speed.

a) *Galloping* – Galloping is transverse oscillations of some structures due to the development of aerodynamic forces which are in phase with the motion. It is characterized by the progressively increasing amplitude of transverse vibration with increase of wind speed. The cross sections which are particularly prone to this type of excitation include the following:

1) All structures with non-circular cross sections, such as triangular, square, polygons, as well as angles, crosses, and T sections.

2) Twisted cables and cables with ice encrustations.

b) *Flutter* – Flutter is unstable oscillatory motion of a structure due to coupling between aerodynamic force and elastic deformation of the structure. Perhaps the most common form is oscillatory motion due to combined bending and torsion. Although oscillatory motion in each degree of freedom may be damped, instability can set in due to energy transfer from one mode of oscillation to another and the structure is seen to execute sustained or divergent oscillations with a type of motion which is a combination of the individual modes of vibration. Such energy transfer takes place when the natural frequencies of modes taken individually are close to each other (ratio being typically less than 2.0). Flutter can set in at wind speeds much less than those required for exciting the individual modes of motion. Long span suspension bridge decks or any member of a structure with large values of *d*/*t* (where *d* is the length of the member and *t* is its dimension parallel to wind stream) are prone to low speed flutter. Wind tunnel testing is required to determine critical flutter speeds and the likely structural response. Other types of flutter are single degree of freedom stall flutter, torsional flutter, etc.

c) *Ovalling* – Thin walled structures with open ends at one or both ends such as oil storage tanks and natural draught cooling towers in which the ratio of the diameter or minimum lateral dimension to the wall thickness is of the order of 100 or more are prone to ovalling oscillations. These oscillations are characterized by periodic radial deformation of the hollow structure.

7 Buildings and structures that may be subjected to significant wind excited oscillations require careful investigations. It is to be noted that wind induced oscillations may occur at wind speeds lower than the design wind speed.

8 Analytical methods for the evaluation of response of dynamic structures to wind loading can be found in the special publications.

9 In assessing wind loads due to such dynamic phenomenon as galloping, flutter and ovalling, in the absence of the required information either in the special publications or other literature, expert advice should be sought including experiments on models in boundary layer wind tunnels.

4.7.2 *Motion due to Vortex Shedding*

4.7.2.1 *Slender structures*

For a structure, the vortex shedding frequency, *f*s, shall be determined by the following formula:

$$fs = \frac{S_t \overline{V}_{z,H}}{b}$$

where

S_t	=	Strouhal number
$\overline{V}_{z,H}$	=	hourly mean wind speed at height *z*, and
b	=	breadth of a structure or structural member normal to the wind direction in the horizontal plane.

a) *Circular structures* – For structures of circular cross section:

S_t = 0.20 for $D\overline{V}_{z,H}$ less than 6 m^2/s, and
= 0.25 for $D\overline{V}_{z,H}$ more than or equal to 6 m^2/s.

b) *Rectangular structures* – For structures of rectangular cross section:

$S_t = 0.10$

NOTES

1 Significant cross wind motions may be produced by vortex shedding, if the natural frequency of the structure or structural element is equal to the frequency of the vortex shedding within the range of expected wind speeds. In such cases, further analysis should be carried out on the basis of special publications.

2 Unlined welded steel cylindrical structures are prone to excitations by vortex shedding.

3 Intensification of the effects of periodic vortex shedding has been reported in cases where two or more similar structures are located in close proximity, for example at less than 20*b* apart, where *b* is the dimension of the structure normal to the wind.

4 The formulae given in **4.7.2.1**(a) is valid for infinitely long cylindrical structures. The value of S_t decreases slowly as the ratio of length to maximum transverse width decreases, the reduction being up to about half the value, if the structure is only three times higher than its width. Vortex shedding need not be considered, if the ratio of length to maximum transverse dimension is less than 2.0.

4.8 Dynamic Wind Response

4.8.1 *General*

Tall buildings/structures which are 'wind sensitive' shall be designed for dynamic wind loads. Hourly mean wind speed is used as a reference wind speed to be used in dynamic wind analysis. For calculation of along wind loads and response (bending moments, shear forces, or tip deflections) the Gust Factor (GF) method is used as specified in **4.8.2**. The across wind design peak base overturning moment and tip deflection shall be calculated using **4.8.3**.

4.8.2 *Along Wind Response*

For calculation of along-wind load effects at a level s on a building/structure, the design hourly mean wind pressure at height *z* shall be multiplied by the Gust Factor (*G*). This factor is dependent on both the overall height *h* and the level *s* under consideration (*see* Fig. 9). For calculation of base bending moment and deflection at the top of the building/structure, '*s*' should be taken as zero.

Note that $0 < s < h$, and $s < z < h$.

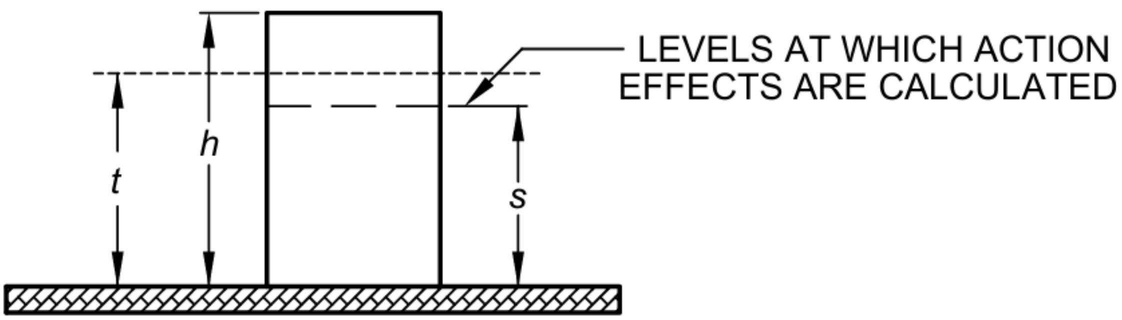


FIG. 9 NOTATIONS FOR HEIGHTS

The design peak along wind base bending moment (M_a) shall be obtained by summing the moments resulting from design peak along wind loads acting at different heights, z, along the height of the building/structure and can be obtained from,

$$M_a = \sum F_z \, z$$

$$F_z = C_{f,z} A_z \overline{p}_d G$$

where

F_z = design peak along wind load on the building/structure at any height *z;*
A_z = the effective frontal area of the building/structure at any height *z*, in m^2;
$\overline{p}_d$ = design hourly mean wind pressure corresponding to $\overline{V}_{z,d}$ and obtained as $0.6\overline{V}_{z,d}^2$ (N/m^2);
$\overline{V}_{z,d}$ = design hourly mean wind speed at height *z*, in m/s (*see* **4.4.4**);

$C_{f,z}$ = the drag force coefficient of the building/structure corresponding to the area A_z;

G = Gust Factor and is given by:

$$= 1 + r\sqrt{\left[g_v^2 B_s (1+\phi)^2 + \frac{H_s g_R^2 SE}{\beta}\right]}$$

where

r = roughness factor which is twice the longitudinal turbulence intensity, $I_{h,i}$ (*see* **4.4.5**);

g_v = peak factor for upwind velocity fluctuation
= 3.0 for category 1 and 2 terrains; and
= 4.0 for category 3 and 4 terrains;

B_s = background factor indicating the measure of slowly varying component of fluctuating wind load caused by the lower frequency wind speed variations

$$= \frac{1}{\left[1 + \frac{\sqrt{0.26(h-s)^2 + 0.46 b_{sh}^2}}{L_h}\right]}$$

where

b_{sh} = average breadth of the building/structure between heights *s* and *h*;

L_h = measure of effective turbulence length scale at the height, *h*, in m;

$= 85\left(\frac{h}{10}\right)^{0.25}$ for terrain category 1 to 3, and

$= 70\left(\frac{h}{10}\right)^{0.25}$ for terrain category 4

ϕ = factor to account for the second order turbulence intensity;

$$= \frac{g_v I_{h,i} \sqrt{B_s}}{2}$$

$I_{h,i}$ = turbulence intensity at height *h* in terrain category *i*;
H_s = height factor for resonance response;

$$= 1+\left(\frac{s}{h}\right)^2$$

S = size reduction factor given by

$$= \frac{1}{\left[1+\frac{3.5 f_a h}{\overline{V}_{h,d}}\right]\left[1+\frac{4 f_a b_{0h}}{\overline{V}_{h,d}}\right]}$$

where

b_{0h} = average breadth of the building/structure between 0 and *h*;

E = spectrum of turbulence in the approaching wind stream

$$= \frac{\pi N}{\left(1+70.8N^2\right)^{5/6}}$$

where

N = an effective reduced frequency

$$= \frac{f_a L_h}{\overline{V}_{h,d}}$$

f_a = first mode natural frequency of the building/structure in along wind direction, in Hz
$\overline{V}_{h,d}$ = design hourly mean wind speed at height, *h* in m/s (*see* **4.4.4**);
β = damping coefficient of the building/structure (*see* Table 39); and
g_R = peak factor for resonant response.

$$= \sqrt{\left[2\ln(3\,600\, f_a)\right]}$$

Table 39 Suggested Values of Structural Damping Coefficients
(*Clause* 4.8.2)

Sl No. (1)	**Kind of Structure** (2)	**Damping Coefficient, *β*** (3)
i)	Welded steel structures	0.010
ii)	Bolted steel structures / RCC structures	0.020
iii)	Prestressed concrete structures	0.016

4.8.2.1 *Peak acceleration in along wind direction*

The peak acceleration at the top of the building/structure in along wind direction ($\hat{\ddot{x}}$ in m/s^2) is given by the following equation:

$$\hat{\ddot{x}} = (2\pi f_a)^2 \bar{x}\, g_R\, r \sqrt{\frac{SE}{\beta}}$$

where,

$\bar{x}$ = mean deflection at the position where the acceleration is required. Other notations are same as given in **9.2**.

For computing the peak acceleration in the along wind direction, a mean wind speed at the height of the building / structure, $\bar{V}_h$ corresponding to a 5 year mean return period shall be used. A reduced value of 0.011 is also suggested for the structural damping, *β* for reinforced concrete structures.

4.8.3 *Across Wind Response*

This section gives method for determining equivalent static wind load and base overturning moment in the across wind direction for tall enclosed buildings and towers of rectangular cross section. Calculation of across wind response is not required for lattice towers.

The across wind design peak base bending moment M_c for enclosed buildings and towers shall be determined as follows:

$$M_c = 0.5 g_h \bar{p}_h b h^2 (1.06 - 0.06k)\sqrt{\frac{\pi C_{fs}}{\beta}}$$

where

g_h = a peak factor;
= $\sqrt{[2\ln(3\,600\, f_c)]}$ in cross wind direction;

$\bar{p}_h$ = hourly mean wind pressure at height *h*, in Pa;

b = breadth of the structure normal to the wind, in m;

h = height of the structure, in m;

k = a mode shape power exponent for representation of the fundamental mode shape as represented by:

$$\psi(z) = \left(\frac{z}{h}\right)^k$$; and

f_c = first mode natural frequency of the building/structure in across wind direction, in Hz.

The across wind load distribution on the building/structure can be obtained from M_c using linear distribution of loads as given below:

$$F_{z,c} = \left(\frac{3M_c}{h^2}\right)\left(\frac{z}{h}\right)$$

where $F_{z,c}$ = across wind load per unit height at height *z.*

4.8.3.1 *Peak acceleration in across wind direction*

The peak acceleration at the top of the building/structure in across-wind direction ($\hat{\ddot{y}}$ in m/s^2) with approximately constant mass per unit height shall be determined as follows:

$$\hat{\ddot{y}} = 1.5 \frac{g_h \, \bar{p}_h \, b}{m_0} (0.76 + 0.24k) \sqrt{\left(\frac{\pi C_{fs}}{\beta}\right)}$$

Typical values of the mode shape power exponent, *k* are as follows:

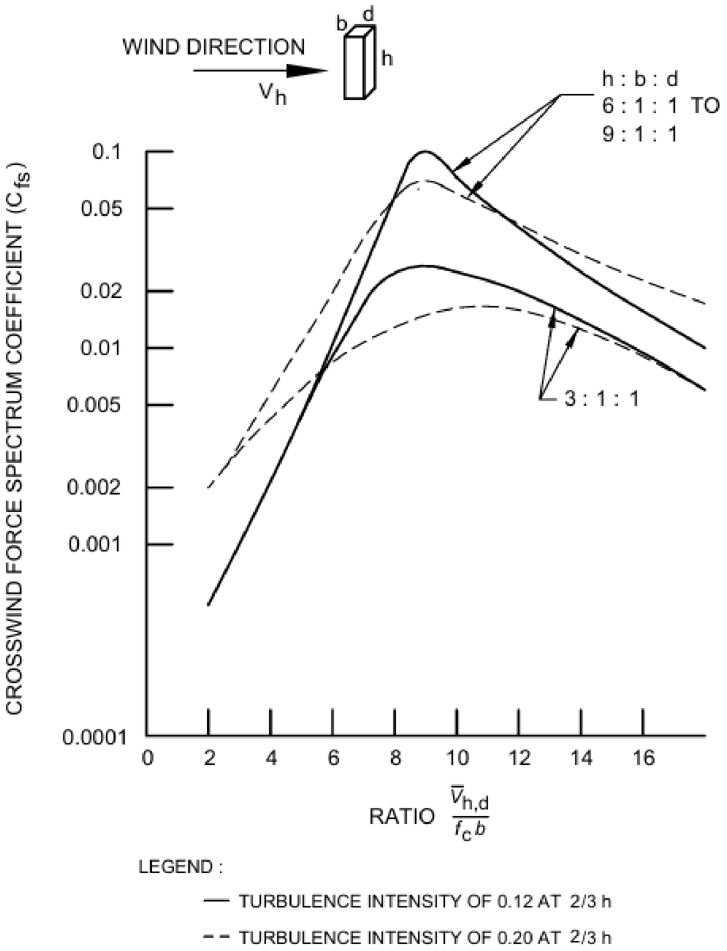
a) uniform cantilever, *k* = 1.5
b) slender framed structure (moment resisting), *k* = 0.5
c) building with a central core and moment resisting façade, *k* = 1.0
d) lattice tower decreasing in stiffness with height, or a tower with a large mass at the top, *k* = 2.3

C_{fs} = across wind force spectrum coefficient generalized for a linear mode, (*see* Fig. 10);
β = damping coefficient of the building/structure (*see* Table 39); and
m_0 = the average mass per unit height of the structure, in kg/m.

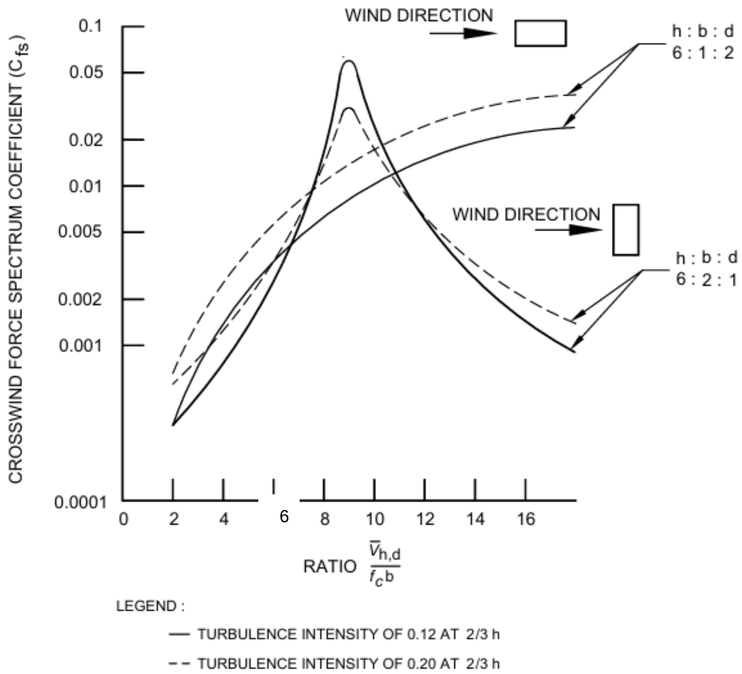
4.8.4 *Combination of Along Wind and Across Wind Load Effects*

The along wind and across wind loads have to be applied simultaneously on the building/structure during design.

FIGURE 10A VALUES OF THE CROSS WIND FORCE SPECTRUM COEFFICIENT FOR SQUARE SECTION BUILDINGS



` WIND FORCE SPECTRUM COEFFICIENT (*Continued*)



10B VALUES OF THE CROSS WIND FORCE SPECTRUM COEFFICIENT FOR 2:1 AND 1:2 RECTANGULAR SECTION BUILDINGS

FIG. 10 WIND FORCE SPECTRUM COEFFICIENTS

5 EARTHQUAKE EFFECTS

The provision of this clause will be added suitably after the finalization/publication of Part 1 and 2 of 'Criteria for Earthquake Resistant Design of Structures - Part 1 General Provisions and Part 2 Buildings [(Second Revision of IS 1893 (Part 1)].

5.1	**GENERAL**
5.2	**TERMINOLOGY, SYMBOLS AND ABBREVIATIONS**
5.3	**GENERAL PRINCIPLES**
	5.3.1 Ground Motion
	5.3.2 Forces for Design of Structures
	5.3.3 Inelastic Behaviour in Structures
	5.3.4 Design for Ductile Response of Structures
	5.3.5 Soil-Structure Interaction
	5.3.6 Floor Response Spectrum
	5.3.7 Additions, Appendages and Alterations to Existing Structures
	5.3.8 Change in Occupancy
	5.3.9 Assumptions
5.4	**EARTHQUAKE HAZARD**
	5.4.1 Earthquake Ground Shaking
	5.4.1.1 Horizontal and Vertical Shaking Effects
	5.4.2 Design Earthquake Hazard
	5.4.2.1 Earthquake Zones
	5.4.2.2 Design Earthquake Peak Ground Acceleration
	5.4.2.2.1 Categories of Structures
	5.4.2.2.2 Return Periods for each Category of Structure
	5.4.2.2.3 Earthquake Zone Factor
	5.4.2.3 Importance Factor
	5.4.2.4 Elastic Response Pseudo-Acceleration, Pseudo-Velocity and Displacement Spectra
	5.4.2.4.1 Site Class
	5.4.2.4.2 Normalized Elastic Maximum Horizontal and Vertical PSA, PSV, and SD for damping ratio of 5% of critical damping
	5.4.2.4.3 Normalized Elastic Maximum Horizontal and Vertical PSA, PSV and SD for damping ratio other than 5% of critical damping
	5.4.2.4.4 Design Elastic Maximum Horizontal and Vertical PSA, PSV and SD
	5.4.3 Site-Specific Earthquake Hazard
	5.4.3.1 Lower Bound when Site-Specific Spectrum is used
	5.4.3.2 Procedure
5.5	**PRINCIPLES OF DESIGN OF STRUCTURES**
	5.5.1 Performance Expectation
	5.5.1.1 Damage
	5.5.1.2 Integrity
	5.5.1.2.1 Desirable Collapse Mechanism
	5..1.2.2 Key Elements
	5.5.2 Attributes

	5.5.2.1 Structural Configuration
	5.5.2.2 Lateral Stiffness
	5.5.2.3 Lateral Strength
	5.5.2.4 Deformability and Ductility
	5.5.2.5 Energy Dissipation and Desirable Collapse Mechanism
	5.5.3 Design Requirements
	5.5.4 Design Earthquake Force
	5.5.4.1 Seismic Weight (W)
	5.5.4.2 Earthquake Forces for Strength Design
	5.5.4.2.1 Minimum Design Earthquake Forces
	5.5.4.3 Designing for Effects of Earthquake Shaking
	5.5.4.4 Revision in Design Earthquake Forces
	5.5.5 Load Combinations
	5.5.5.1 Basic Combinations
	5.5.5.2 Multi-directional Earthquake Shaking
	5.5.5.2.1 One Horizontal Plan Direction at a time
	5.5.5.2.2 Two Mutually Orthogonal Plan Directions at a time
	5.5.5.2.3 Three Mutually Orthogonal Directions at a time
	5.5.6 Earthquake Analysis
	5.5.5.6.1 Modelling
	5.5.6.2 Methods of Analysis
	5.5.6.2.1 Linear Equivalent Static Analysis
	5.5.6.2.2 Linear Dynamic Analysis
	5.5.6.2.3 Protection of Base Shear Force from Dynamic Method
	5.5.7 Earthquake Resistant Design
	5.5.7.1 Structural Elements in Superstructures and Foundations Elements
	5.5.7.2 Soils and Earthen Embankments
	5.5.7.3 Overstrength-based Requirement for Foundation Elements and Soil
	5.5.8 Geotechnical Aspects
	5.5.8.1 Soil Properties
	5.5.8.1.1 Basic Input
	5.5.8.1.2 Soil Flexibility
	5.5.8.1.3 Soil Strength
	5.5.8.1.4 Soil Damping
	5.5.8.2 Liquefaction
	5.5.8.2.1 Estimation of Cyclic Stress Ratio
	5.5.8.2.2 Estimation of Cyclic Resistance Ratio CRR
	5.5.8.2.3 Assessing Liquefaction Potential of a Site
5.6	**ARCHITECTURAL ELEMENTS AND UTILITIES**
	5.6.1 Classification of *AEUs*
	5.6.2 Protection of *AEUs*
	5.6.3 Load Effects for Design of System to Protect *AEUs*
	5.6.3.1 Acceleration-sensitive A-*AEUs*
	5.6.3.2 Displacement-sensitive D-*AEUs*
	5.6.3.3 Acceleration-cum-Displacement-sensitive A-D *AEUs*

	5.6.4 Earthquake Analysis
	5.6.5 Earthquake Demands on *AEUs*
	5.6.5.1 Acceleration-Sensitive *AEUs*
	5.6.5.2 Displacement-Sensitive *AEUs*
	5.6.5.3 Acceleration-cum-Displacement-Sensitive *AEUs*
5.7	**ADDITIONAL CRITERIA OF BULDINGS**
	5.7.1 For All Buildings 5.7.1.1 General Provisions 5.7.1.1.1 Irregularities 5.7.1.1.1.1 Geometry 5.7.1.1.1.2 Mass 5.7.1.1.1.3 Stiffness 5.7.1.1.1.4 Strength 5.7.1.1.1.5 Behaviour 5.7.1.1.2 Design Requirements 5.7.1.1.2.1 Stiffness Requirements 5.7.1.1.2.2 Strength Requirements 5.7.1.1.2.3 Deformability Requirements 5.7.1.1.3 Structural Analysis 5.7.1.1.3.1 Methods of Analysis 5.7.1.1.3.2 Modelling 5.7.1.1.4 Miscellaneous 5.7.1.1.4.1 Buildings on Sloping Ground 5.7.1.1.4.2 Foundations 5.7.1.1.4.3 Cantilever Projections 5.7.1.1.4.4 Compound Walls 5.7.1.1.4.5 Architectural Elements and Utilities **5.7.2 Earthquake Demand on Masonry Buildings** 5.7.2.1 Structural Systems 5.7.2.2 Design Earthquake Force 5.7.2.3 Structural Analysis 5.7.2.4 Special Categories of Buildings **5.7.3 Earthquake Demand on Concrete Buildings** 5.7.3.1 Structural Analysis 5.7.3.2 Special Categories of Buildings **5.7.4 Earthquake Demand on Concrete Buildings** 5.7.4.1 Structural Analysis 5.7.4.2 Special Categories of Buildings
ANNEX H	**SYMBOLS**
ANNEXJ	**MSK 1964 INTENSITY SCALE**
ANNEX K	**EARTHQUAKE HAZARD ASSESSMENT FOR MACRO-ZONING AND SITE-SPECIFIC STUDIES**
ANNEX L	**EARTHQUAKE ZONES OF SELECT TOWNS AS PER 2011 CENSUS OF INDIA**

6 SNOW LOAD

6.1 This clause deals with snow loads on roofs of buildings. Roofs should be designed for the actual load due to snow or for the imposed loads specified in **3**, whichever is more severe.

NOTE — Mountainous regions in northern and eastern parts of India are subjected to snow-fall. In northern India, parts of union territory of Jammu and Kashmir (J&K), union territory of Ladakh, Himachal Pradesh, Uttarakhand, and in eastern India, parts of Arunachal and Sikkim experience snow-fall of varying depths two to three times in a year.

6.2 Notations

a) μ (Dimensionless) – Nominal values of the shape coefficients, taking into account snow drifts, sliding snow, etc, with subscripts, if necessary.

b) l_{i} (m) – Horizontal dimension with numerical subscripts, if necessary.

c) h_{i} (m) – Vertical dimensions with numerical subscripts, if necessary.

d) β_{i} (degree) – Roof and other surface slope.

e) s_0 (kPa) – Characteristic value of snow load on the ground with a specified annual exceedance probability.

f) s (kPa) – Snow load on roofs other surfaces.

g) A (m) – Site altitude above sea level

6.3 Snow Load in Roof(s)

6.3.1 The minimum design snow load on a roof area or any other area above ground which is subjected to snow accumulation is obtained by multiplying the snow load on ground, s_o by the shape coefficient μ, as applicable to the particular roof area considered, expressed as:

$$s = \mu \, s_o$$

where

s = design snow load on plan area of roof, in Pa;
μ = shape coefficient (*see* **6.4**); and
s_o = ground snow load in Pa (1 Pa=1 N/m^2).

NOTE – Ground snow load at any place depends on the critical combination of the maximum depth of undisturbed aggregate cumulative snow fall and its average density. In due course the characteristic snow load on ground for different regions will be included based on studies. Till such time the users of this code are advised to contact either Defence Geoinformatics

Research Establishment (DGRE), Chandigarh of Defence Research and Development Organization (DRDO), or India Meteorological Department (IMD), Pune in the absence of any specific information for any location.

6.3.2 *Characteristic Ground Snow Load* (s_0)

The characteristic ground snow load (s_0) at any site is defined based on an annual probability of exceedance of 0.034 and corresponds to a mean return period of 30 years. The value of s_0 (kPa) for any site in snow-bound regions of Indian Himalayas should be obtained using the procedure described in Annex M.

NOTE — Annex M gives the characteristic ground snow load map for union territory of Jammu and Kashmir, union territory of Ladakh, Himachal Pradesh, Uttarakhand and Sikkim resulting from studies conducted by DGRE, Chandigarh.

6.3.3 *Partial Loading Due to Melting, Sliding, Snow Redistribution and Snow Removal*

A loading corresponding to severe imbalances resulting from snow removal, redistribution, sliding, melting, etc, (for example, zero snow load on specific parts of the roof) should always be considered. Such considerations are particularly important for structures which are sensitive to unbalance loading (for example, curved roofs, arches, domes, collar beam roofs, continuous beam systems). Water load due to ponding in flat roof as well as valley of pitched roof shall be taken as not less than 0.24 kPa.

6.3.4 *Ponding Instability*

6.3.4.1 Roofs shall be designed to preclude ponding.

6.3.4.2 For flat roofs (or with a small slope), roof deflections caused by snow loads shall be investigated when determining the likelihood of ponding from rain-on-snow or from snow meltwater. The angle for snow melting is 15º.

6.3 Shape Coefficients

6.4.1 *General Principles*

In perfectly calm weather, falling snow would cover roofs and the ground with a uniform blanket of snow, and the design snow load could be considered as a uniformly distributed load. Truly uniform loading conditions, however, are rare and have usually only been observed in areas that are sheltered on all sides by high trees, buildings, etc. In such a case, the shape coefficient would be equal to unity.

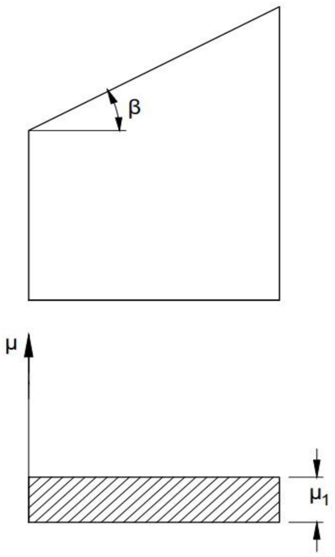
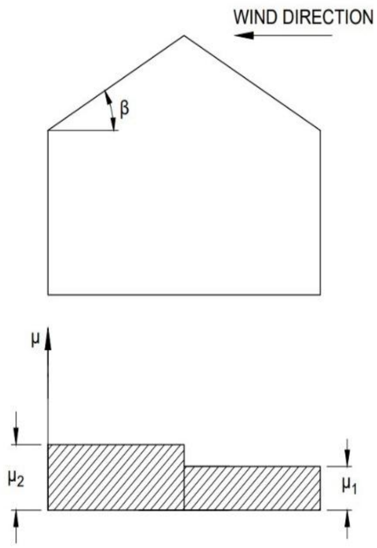
In most regions, snow-falls are accompanied or followed by winds. The winds will redistribute the snow, and on some roofs especially multilevel roofs, the accumulated drift load may reach a multiple of the ground load. Roofs which are sheltered by other buildings, vegetation, etc, may collect more snow load than the ground level. The phenomenon is of the same nature as that illustrated for multilevel roofs in **6.4.2.4**.

So far, sufficient data are not available to determine the shape coefficient on a statistical basis. Therefore, a nominal value is given. A representative sample of roofs is shown in **6.4.2**. However, in special cases such as strip loading, cleaning of the roof periodically by deliberate heating of the roof, etc, have to be treated separately. However, no reduction in load of snow on roof having underside insulation and/or internal heating system shall be done.

The distribution of snow in the direction parallel to the caves is assumed to be uniform.

6.4.2 *Shape Coefficients for Selected Types of Roofs*

6.4.2.1 *Monopitched and simple pitched roofs*

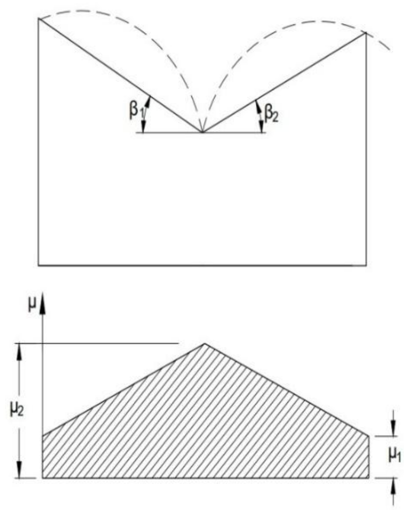
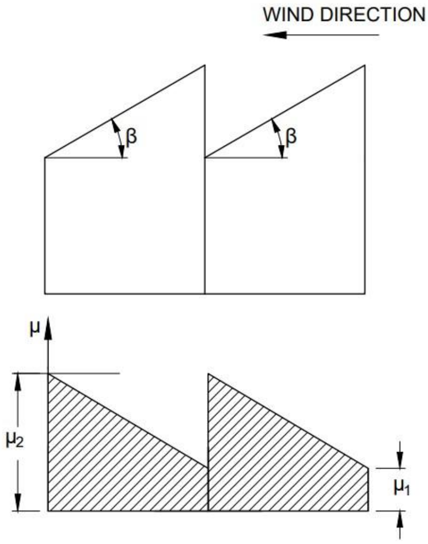
	 Simple Flat and Monopitch Roofs	 Simple Pitched Roofs (Positive Roof Slope)*
$0° < \beta \leq 15°$	$\mu_1 = 0.80$	$\mu_2 = \mu_1 = 0.80$
$15° < \beta \leq 30°$		$\mu_2 = 0.80 + 0.40\left(\frac{\beta - 15}{15}\right)$ $\mu_1 = 0.80$
$30° < \beta \leq 60°$	$\mu_1 = 0.80\left(\frac{60 - \beta}{30}\right)$	$\mu_2 = 1.20\left(\frac{60 - \beta}{30}\right)$ $\mu_1 = 0.80\left(\frac{60 - \beta}{30}\right)$
$\beta > 60°$	$\mu_1 = 0$	$\mu_2 = \mu_1 = 0$

*For asymmetrical simple pitched roofs, each side of the roof shall be treated as one half of corresponding symmetrical roofs.

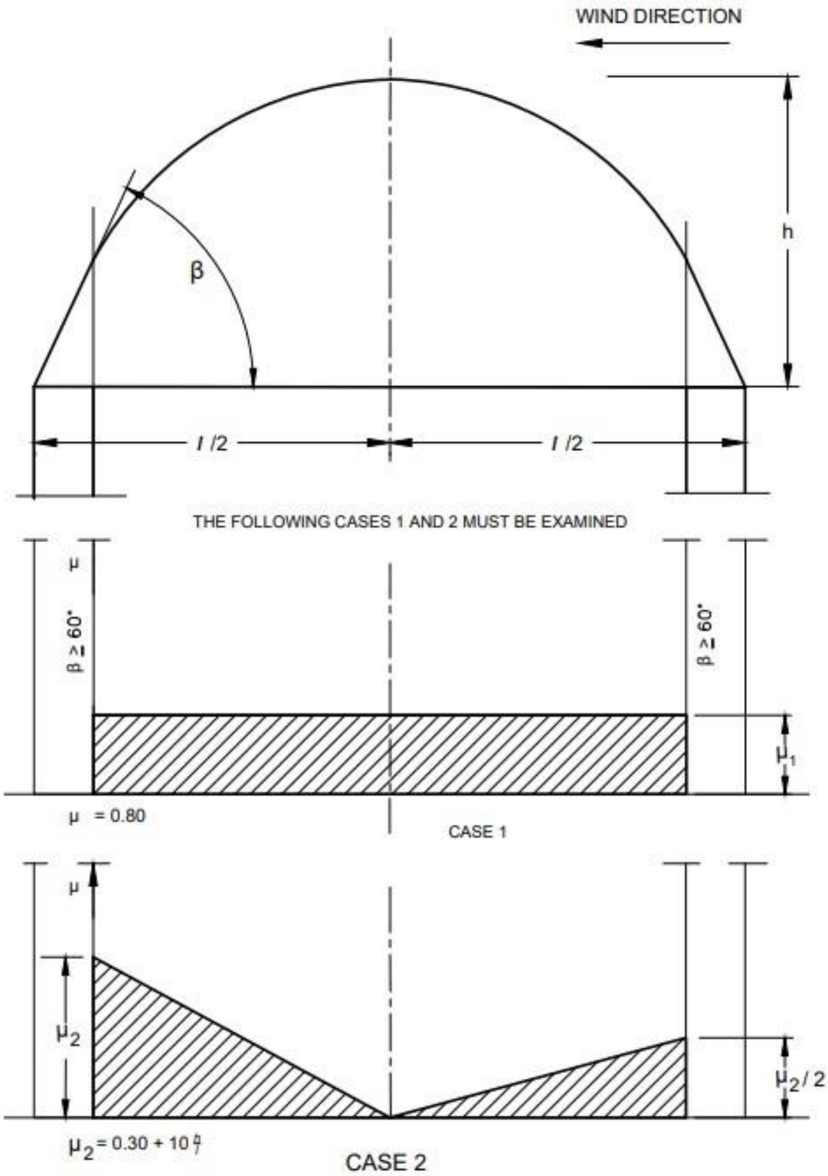
NOTES

1 For monopitch roof, wind direction will not affect the value of *μ*.
2 The values of *μ*1 and *μ*2 shall be reversed in case the wind direction changes from left to right for simple pitched roof.

6.4.2.2 *Multiple pitched roofs*

	 Simple on Multiple Pitched Roofs (Negative Roof Slope)	 Two – Span or Multispan Roofs
$0° < \beta \leq 30°$	$\mu_2 = 0.80\left(\frac{30+\beta}{30}\right)$ $\mu_1 = 0.80$	$\mu_2 = 0.80\left(\frac{30+\beta}{30}\right)$ $\mu_1 = 0.80$
$30° < \beta \leq 60°$	$\mu_1 = 1.60$ $\mu_1 = 0.80\left(\frac{60-\beta}{30}\right)$	$\mu_2 = 1.60$ $\mu_1 = 0.80\left(\frac{60-\beta}{30}\right)$
$\beta > 60°$	$\mu_1 = 1.60$ $\mu_1 = 0$	$\mu_2 = 1.60$ $\mu_1 = 0$

6.4.2.3 *Simple curved roofs*

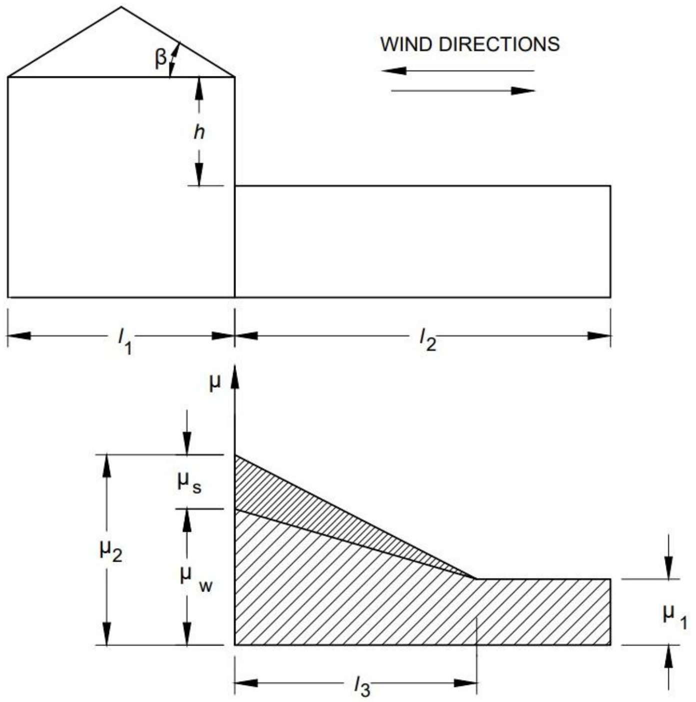


Restrictions:

$\mu_2 \leq 2.3$

$\mu = 0 \; if \; \beta \geq 60^o$

6.4.2.4 *Multilevel roofs*[1)]



$\mu_1 = 0.80$
$\mu_2 = \mu_s + \mu_w$

where

μ_s = due to sliding
μ_w = due to wind

$l_3 = 2h$ 2) but is restricted as follows:

$5\ m \leq l_3 \leq 15\ m$
$\mu_w = \frac{l_1+l_2}{2h} \leq \frac{kh}{s_0}$ with the restriction $0.80 \leq \mu_w \leq 4.0$

where

h is in meters
s_0 is in kPa $(\mathrm{kN/m^2})$
$k = 2\,\mathrm{kN/m^2}$

NOTES

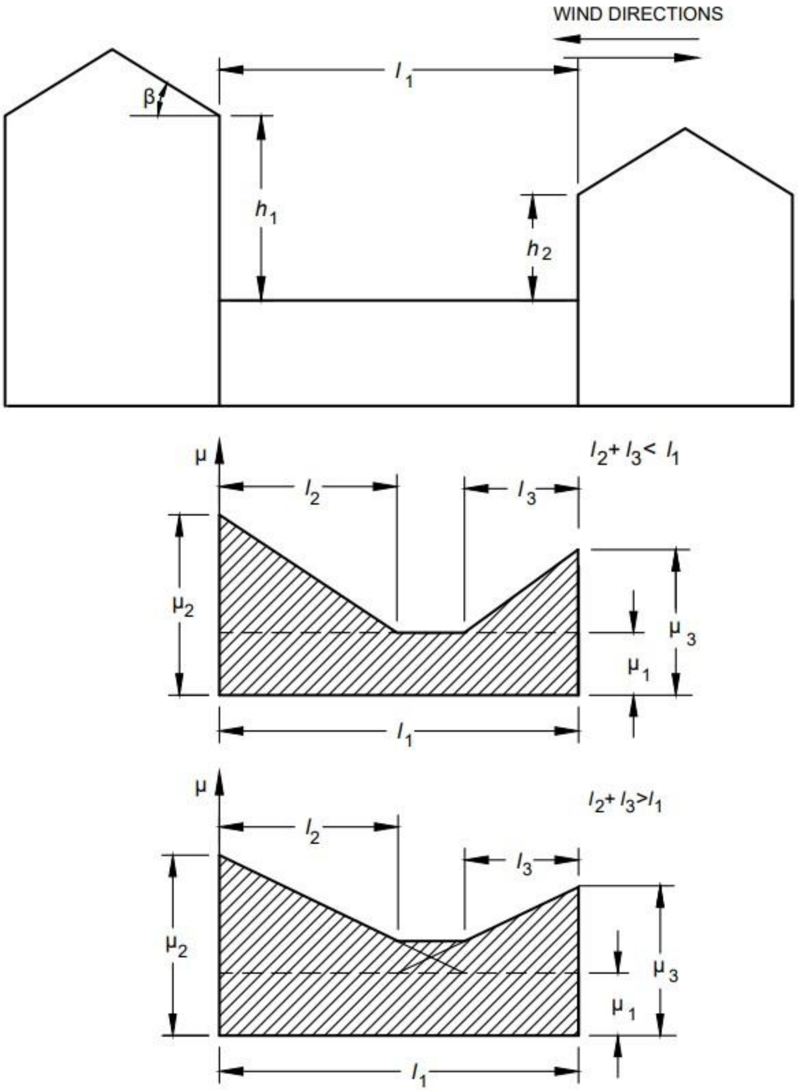
1 A more extensive formula for μ_w is described in Annex N.

2 If $l_2 < l_3$, the coefficient μ is determined by interpolation between μ_1 and μ_2.

If $\beta > 15^\circ$: μ_s is determined from an additional load amounting to 50 percent of the maximum total load on the adjacent slope of the upper roof and is distributed linearly as shown in the figure. The load on the upper roof is calculated according to **6.4.2.1** or **6.4.2.2.**

If $\beta \leq 15^\circ$: $\mu_s = 0$

6.4.2.5 *Complex multilevel roofs*



$l_2 = 2h_1$; $l_3 = 2h_2$; $\mu_1 = 0.80$;

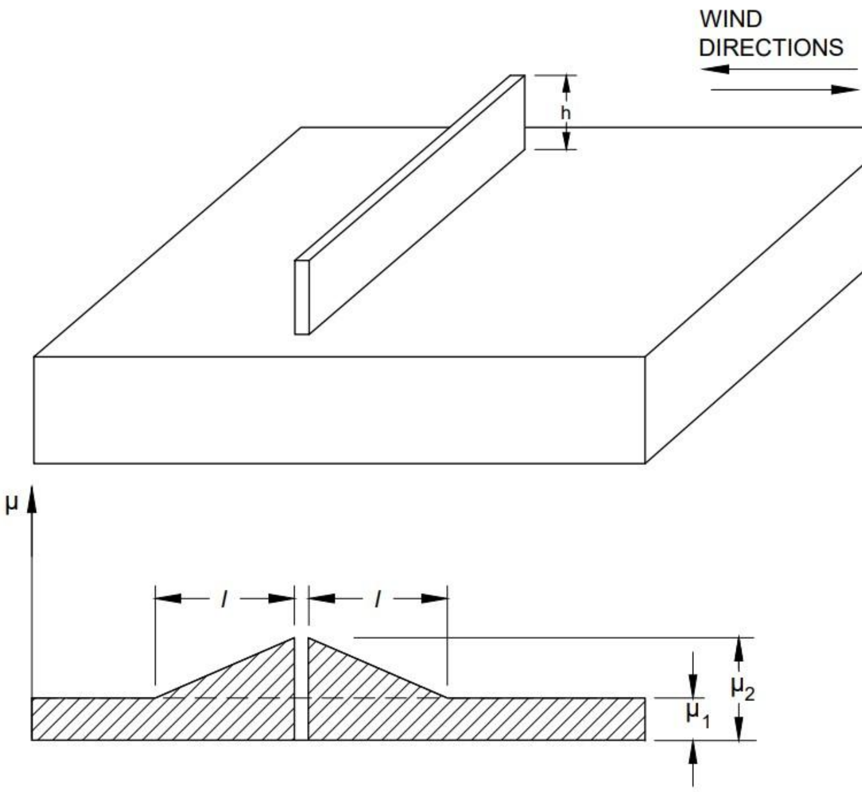
Restrictions:

$5\ m \leq l_2 \leq 15\ m$

$5\ m \leq l_3 \leq 15\ m$

μ_2 and $\mu_3 = \mu_s + \mu_w$, are calculated according to **6.4.2.1, 6.4.2.2** and **6.4.2.4**

6.4.2.6 *Roofs with local projections and obstructions*



$$\mu_2 = \frac{kh}{s_0}$$

$$\mu_1 = 0.80$$

where

h is in meters

s_0 is in kPa $(\mathrm{kN/m^2})$

$k = 2\,\mathrm{kN/m^2}$

$l = 2\,h$

Restriction:

$0.80 \leq \mu_2 \leq 2.0$

$5\,m \leq l \leq 15\,m$

6.4.3 *Shape Coefficients in Areas Exposed to Wind*

The shape coefficients given in **6.4.2** and Annex N may be reduced by 15 percent, provided the designer has demonstrated that the following conditions are fulfilled:

a) The building is located in an exposed location such as open level terrain with only scattered buildings, trees or other obstructions so that the roof is exposed to the winds on all sides and is not likely to become shielded in the future by obstructions higher than the roof within a distance from the building equal to ten times the height of the obstruction above the roof level; and

b) The roof does not have any significant projections such as parapet walls which may prevent snow from being blown off the roof.

NOTE – In some areas, winter climate may not be of such a nature as to produce a significant reduction of roof loads from the snow load on the ground. These areas are:

1 Winter calm valleys in the mountains where sometimes layer after layer of snow accumulates on roofs without any appreciable removal of snow by wind; and
2 Areas (that is, high temperature) where the maximum snow load may be the result of single snowstorm, occasionally without appreciable wind removal.

In such areas, the determination of the shape coefficients shall be based on local experience with due regard to the likelihood of wind drifting and sliding.

7 SPECIAL LOADS

7.1 This clause gives guidance on loads and load effects due to temperature changes, soil and hydrostatic pressures, internally generating stresses (due to creep, shrinkage, differential settlement, etc), accidental loads, etc, to be considered in the design of buildings as appropriate. This clause also includes guidance on load combinations. The nature of loads to be considered for a particular situation is to be based on engineering judgment (*see also* **3.6**)

7.2 Temperature Effects

7.2.1 Expansion and contraction due to changes in temperature of the materials of a structure shall be considered in design. Provision shall be made either to relieve the stress by the provision of expansion/contraction joints in accordance with good practice [6-1(21)] or design the structure to carry additional stresses due to temperature effects as appropriate to the problem.

7.2.1.1 The temperature range varies for different regions and under different diurnal and seasonal conditions. The absolute maximum and minimum temperature which may be expected in different localities in the country are indicated in Annex B of good practice [6-1(22)]. These figures may be used for guidance in assessing the maximum variations of temperature.

7.2.1.2 The temperatures indicated are the air temperatures in the shade. The range of variation in temperature of the building materials may be appreciably greater or less than the variation of air temperature and is influenced by the condition of exposure and the rate at which the materials composing the structure absorb or radiate heat. This difference in temperature variations of the material and air should be given due consideration.

7.2.1.3 The structural analysis must take account of changes of the mean (through the section) temperature in relation to the initial (st) and the temperature gradient through the section.

a) It should be borne in mind that the changes of mean temperature in relation to the initial are liable to differ as between one structural element and another in buildings or structures, as for example, between the external walls and the internal elements of a building. The distribution of temperature through section of single-leaf structural elements may be assumed linear for the purpose of analysis.

b) The effect of mean temperature changes t_1 and t_2, and the temperature gradients v_1 and v_2 in the hot and cold seasons for single-leaf structural elements shall be evaluated on the basis of analytical principles.

NOTES

1 For portions of the structure below ground level, the variation of temperature is generally insignificant. However, during the period of construction, when the portions of the structure are exposed to weather elements, adequate provision should be made to encounter adverse effects, if any.

2 If it can be shown by engineering principles, or if it is known from experience, that neglect of some or all the effects of temperature do not affect the structural safety and serviceability, they need not be considered in design.

7.3 Hydrostatic and Soil Pressure

7.3.1 In the design of structures or parts of structures below ground level such as retaining walls and other walls in basement floors, the pressure exerted by the soil or water or both shall be duly accounted for on the basis of established theories. Due allowance shall be made for possible surcharge from stationary or moving loads. When a portion or whole of the soil is below the free water surface, the lateral earth pressure shall be evaluated for weight of soil diminished by buoyancy and the full hydrostatic pressure.

7.3.1.1 All foundation slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift equal to the full hydrostatic pressure. Checking of overturning of foundation under submerged condition shall be done considering buoyant weight of foundation.

7.3.2 While determining the lateral soil pressure on column like structural members such as pillars which rest in sloping soils, the width of the member shall be taken as follows (*see* Fig. 11).

Actual Width of Member	*Ratio of Effective Width to Actual Width*
Less than 0.5 m	3.0
Beyond 0.5 m and up to 1 m	3.0 to 2.0
Beyond 1 m	2.0

The relieving pressure of soil in front of the structural member concerned may generally not be taken into account.

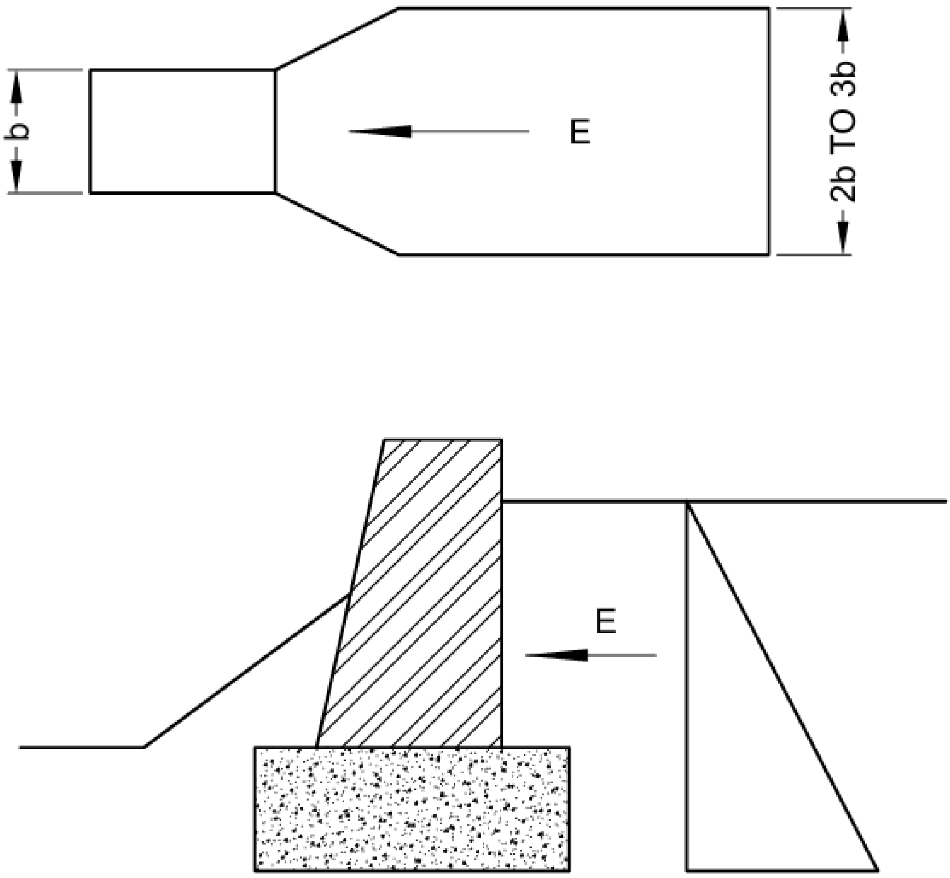


FIG. 11 SKETCH SHOWING EFFECTIVE WIDTH OF PILLAR FOR CALCULATING SOIL PRESSURE

7.3.3 Safe-guarding of structures and structural members against overturning and horizontal sliding shall be verified. Imposed loads having favourable effect shall be disregarded for the purpose. Due consideration shall be given to the possibility of soil being permanently or temporarily removed.

7.4 Fatigue

7.4.1 *General*

Fatigue cracks are usually initiated at points of high stress concentration. These stress concentrations may be caused by or associated with holes (such as bolt or rivet holes in steel structures), welds including stray or fusions in steel structures, defects in materials, and local and general changes in geometry of members. The cracks usually propagate if the loading is cyclic and repetitive.

Where there is such cyclic and repetitive loading, sudden changes of shape of a member or part of a member, especially in regions of tensile stress and/or local secondary bending, shall be avoided. Suitable steps shall be taken to avoid critical vibrations due to wind and other causes.

7.4.2 Where necessary, permissible stresses shall be reduced to allow for the effects of fatigue. Allowance for fatigue shall be made for combinations of stresses due to dead load and imposed load. Stresses due to wind and earthquakes may be ignored, when fatigue is being considered, unless otherwise specified in relevant Codes of practice.

Each element of the structure shall be designed for the number of stress cycles of each magnitude to which it is estimated that the element is liable to be subjected during the expected life of the structure. The number of cycles of each magnitude shall be estimated in the light of available date regarding the probable frequency of occurrence of each type of loading.

> NOTE – Apart from the general observations made herein, the section is unable to provide any precise guidance in estimating the probabilistic behaviour and response of structures of various types arising out of repetitive loading approaching fatigue conditions in structural members, joints, materials, etc.

7.5 Structural Safety During Construction

7.5.1 All loads required to be carried by the structures or any part of it due to storage or positioning of construction materials and erection equipment including all loads due to operation of such equipment, shall be considered as erection loads. Proper provision shall be made, including temporary bracings, to take care of all stresses due to erection loads. The conjunction with the temporary bracings shall be capable of sustaining these erection loads, without exceeding the permissible stresses specified in respective Codes of practice. Dead load, wind load and such parts of imposed load, as would be imposed on the structure during the period of erection, shall be taken as acting together with erection loads.

7.6 Accidental Loads

The occurrence of which, with a significant value, is unlikely on a given structure over the period of time under consideration, and also in most cases, is of short duration. The occurrence of an accidental load could, in many cases, be expected to cause severe consequences, unless special measures are taken.

The accidental loads arising out of human action include the following:

a) Impacts and collisions,
b) Explosions, and
c) Fire.

Characteristic of the above stated loads are that they are not a consequence of normal use and that they are undesired, and that extensive effects are made to avoid them. As a result, the probability of occurrence of an accidental load is small whereas the consequences may be severe.

The causes of accidental loads may be:

1) inadequate safety of equipment (due to poor design or poor maintenance); and

2) wrong operation (due to insufficient teaching or training, indisposition, negligence or unfavourable external circumstances).

In most cases, accidental loads only develop under a combination of several unfavourable occurrences. In practical applications, it may be necessary to neglect the most unlikely loads. The probability of occurrence of accidental loads, which are neglected, may differ for different consequences of a possible failure. A data base for a detailed calculation of the probability will seldom be available.

7.6.1 *Impact and Collisions*

7.6.1.1 *General*

During an impact, the kinetic impact energy has to be absorbed by the vehicle hitting the structure and by the structure itself. In an accurate analysis, the probability of occurrence of an impact with a certain energy object hitting the structure and the structure itself at the actual place must be considered. Impact energies for dropped object should be based on the actual loading capacity and lifting height.

Common sources of impact are:

a) Vehicles;
b) Dropped objects from cranes, fork lifts, etc;
c) Cranes out of control, crane failures; and
d) Flying fragments.

The codal requirements regarding impact from vehicles and cranes are given in **7.6.1.2** and **7.6.1.3**.

7.6.1.2 The requirements for loads as a result of collision between vehicle and structural or non-structural elements shall be as per **7.6.1.2.1** and **7.6.1.2.2.**

7.6.1.2.1 *Collisions between vehicles and structural elements*

In road traffic, the requirement that a structure shall be able to resist collision may be assumed to be fulfilled, if it is demonstrated that the structural element is able to stop a fictitious vehicle, as described below. It is assumed that the vehicle strikes the structural element at a height of 1.2 m in any possible direction and at a speed of 10 m/s (36 km/h).

The fictitious vehicle shall be considered to consist of two masses m_1 and m_2 which during compression of the vehicle, produce an impact force increasing uniformly from zero, corresponding to the rigidities C_1 and C_2. It is assumed that the mass m_1 is broken completely before the breaking of mass m_2 begins.

The following numerical values should be used:

m_1 = 400 kg, C_1 = 10 000 kN/m, the vehicle is compressed.

m_2 = 12 000 kg, C_2 = 300 kN/m, the vehicle is compressed.

NOTE – The described fictitious collision corresponds in the case of a non-elastic structural element to a maximum static force of 630 kN for the mass m_1 and 600 kN for the mass m_2 irrespective of the elasticity, it will therefore be on the safe side to assume the static force to be 630 kN.

In addition, breaking of the mass m_1 will result in an impact wave, the effect of which will depend, to a great extent, on the kind of structural element concerned. Consequently, it will not always be sufficient to design for the static force.

7.6.1.2.1.1 *For structural elements in stilts and car parking levels*

For structural elements in stilts and car parking levels, where driveways and car parking are provided in the buildings which have limited vehicle speeds, the design shall take care of the following considerations:

a) Speed of the vehicle to be 20 km/h maximum (since within a building and inside the car parks, speed of the vehicle may not generally be more than 20 km/h).
b) Impact to be considered at a height of 450 mm to 600 mm.

Structural elements such as columns, RC walls or vertical load resisting elements shall be designed for loads due to the above unless protected by separate resisting system.

7.6.1.2.2 *Collision between vehicles and non-structural elements:*

Non-structural elements, such as vehicle barriers for car parks shall be designed to resist the impact and the momentum as outlined below:

a) The horizontal force *F* (in kN), normal to and uniformly distributed over any length of 1.5 m of a barrier for a car park, required to withstand the impact of a vehicle shall be as given below:

$$F = \frac{0.5mv^2}{\delta_c + \delta_b}$$

where

m = gross mass of the vehicle, in kg;
v = velocity of the vehicle normal to the barrier, in m/s;
δ_c = deformation of the vehicle, in mm;
δ_b = deflection of the barrier, in mm.

b) Where the car park has been designed on the basis that the gross mass of the vehicles using it will not exceed 2 500 kg, the following values shall be used to determine the force *F*:

m = 1 500 kg;
v = 4.5 m/s;
δ_c = 100 mm;

For a rigid barrier, for which δ_b may be taken as zero, the force, *F*, appropriate to vehicles up to 2 500 kg gross mass, shall be taken as 150 kN.

c) Where the car park has been designed for vehicles whose gross mass exceeds 2 500 kg, the following values shall be used to determine the force *F*:

m = actual mass of the vehicle for which the car park is designed, in kg;
v = 4.5 m/s;
δ_c = 100 mm;

d) The force determined as in (b) or (c) may be considered to act at bumper height. In the case of car parks intended for motor cars whose gross mass does not exceed 2 500 kg, this height may be taken as 375 mm above the floor level.

e) Barriers to access ramps of car parks have to withstand one-half of the force determined in (b) or (c) acting at a height of 610 mm above the ramp.

Opposite to the ends of straight ramps intended for downward travel, which exceed 20 m in length, the barrier shall be designed to withstand twice the force determined in (b) or (c) acting at a height of 610 mm above the ramp.

NOTE – The mass of 1 500 kg is taken as being more representative of the vehicle population than the extreme value of 2 500 kg.

7.6.1.3 *Safety railings*

With regard to safety, railings put up to protect structures against collision due to road traffic, it should be shown that the railings are able to resist the impact as described in **7.6.1.2**.

NOTE – When a vehicle collides with safety railings, the kinetic energy of the vehicle will be absorbed partly by the deformation of the railings and partly by the deformation of the vehicle. The part of the kinetic energy which the railings should be able to absorb without breaking down may be determined on the basis of the assumed rigidity of the vehicle during compression.

7.6.1.4 *Crane impact load on buffer stop*

The basic horizontal load P_y (tonne), acting along the crane track produced by impact of the crane on the buffer stop, is calculated by the following formula:

$$P_y = MV^2/F$$

where

V = speed at which the crane is traveling at the moment of impact (assumed equal to half the nominal value) (m/s).

F = maximum shortening of the buffer, assumed equal to 0.1 m for light duty, medium-duty and heavy-duty cranes with flexible load suspension and loading capacity not exceeding 50 t, and 0.2 m in every other cranes.

M = reduced crane mass, (t.s^2/m); and is obtained by the formula:

$$M = \frac{1}{g}\left[\frac{P_{\mathrm{h}}}{2} + \left(P_{\mathrm{t}} + kQ\right)\frac{L_{\mathrm{k}} - l}{L_{\mathrm{k}}}\right]$$

where

g = acceleration due to gravity (9.81 m/s^2);
P_{h} = crane bridge weight (*t*);
P_{t} = crab bridge weight (*t*);
Q = crane loading capacity (*t*);

k = coefficient, assumed equal to zero for cranes with flexible load suspension and to one for cranes with rigid suspension;
L_{k} = crane span (m); and
ℓ = nearness of crab (m).

7.6.2 *Explosions*

7.6.2.1 *General*

Explosions may cause impulsive loading on a structure. The following types of explosions are particularly relevant:

a) Internal gas explosions which may be caused by leakage of gas piping (including piping outside the room), evaporation from volatile liquids or unintentional evaporation from surface material (for example, fire);

b) Internal dust explosions;

c) Boiler failure;

d) External gas cloud explosions; and

e) External explosions of high explosives (TNT, dynamite).

The codal requirement regarding internal gas explosions is given in **7.6.2.2**.

7.6.2.2 *Explosion effect in closed rooms*

Gas explosion may be caused, for example by leaks in gas pipes (inclusive of pipes outside for room), evaporation from volatile liquids or unintentional evaporation of gas from wall sheathings (for example, caused by fire).

NOTES

1 The effect of explosions depends on the exploding medium, the concentration of the explosion, the shape of the room, possibilities of ventilation of the explosion, and the ductility and dynamic properties of the structure. In rooms with little possibility for relief of the pressure from the explosion, very large pressures may occur.

Internal over pressure from an internal gas explosion in rooms of sizes comparable to residential rooms and with ventilation areas consisting of window glass breaking at a pressure of 4 kN/m^2 (3 - 4 mm machine made glass) may be calculated from the following method:

a) The over pressure is assumed to depend on a factor *A*/*V*, where A is the total windows area, in m^2, and *V* is the volume, in m^3, of the room considered;
b) The internal pressure is assumed to act simultaneously upon all walls and floors in one closed room; and
c) The action q_0 may be taken as static action.

2 If account is taken of the time curve of the action, the schematic correspondence between pressure and time is assumed (*see* Fig. 12), where t_1 is the time from the start of combustion until maximum pressure is reached and t_2 is the time from maximum pressure to the end of combustion. For t_1 and t_2, the most unfavourable values should be chosen in relation to the dynamic properties of the structures. However, the values should be chosen within the intervals as given in Fig. 13.

Figure 19 is based on tests with gas explosions in room corresponding to ordinary residential flats and should, therefore, not be applied to considerably different conditions. The figure corresponds to an explosion caused by town gas and it might, therefore, be somewhat on the safe side in rooms where there is only the possibility of gases with a lower rate of combustion.

The pressure may be applied solely in one room or in more rooms at the same time. In the latter case, all rooms are incorporated in the volume *V*. Only windows or other similarly weak and light weight structural elements may be taken to be ventilation areas even though certain limited structural parts break at pressures less than q_0.

Figure 19 is given purely as guide and probability of occurrence of an explosion should be checked in each case using appropriate values.

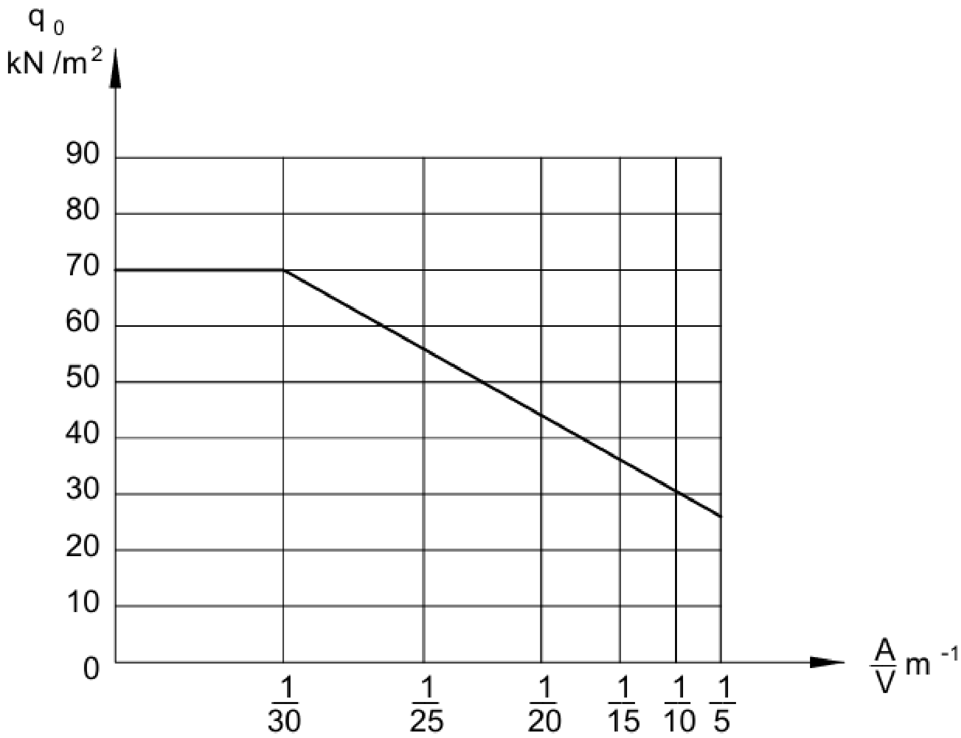


FIG. 12 SKETCH SHOWING RELATION BETWEEN PRESSURE AND TIME

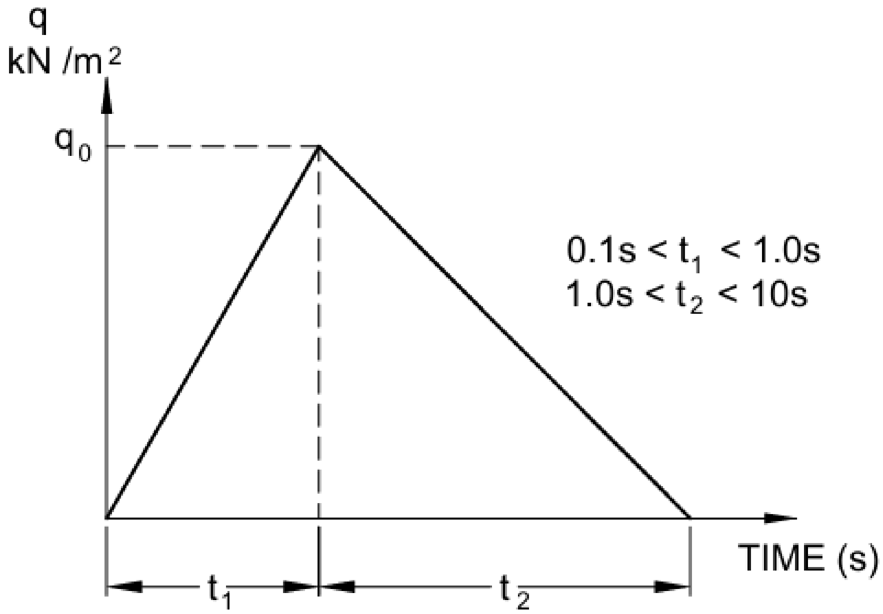


FIG. 13 SKETCH SHOWING TIME INTERVAL AND PRESSURE

7.6.3 *Vertical Load on Air Raid Shelters*

7.6.3.1 *Characteristic values*

As regards buildings in which the individual floors are acted upon by a total characteristic imposed action of up to 5.0 kN/m^2, vertical actions on air raid shelters generally located below ground level, for example, basement, etc, should be considered to have the following characteristic values:

a)	Buildings with up to 2 storeys	:	28 kN/m^2
b)	Buildings with 3-4 storeys	:	34 kN/m^2
c)	Buildings with more than 4 storeys	:	41 kN/m^2
d)	Buildings of particularly stable construction irrespective of the number of storeys	:	28 kN/m^2

In the case of buildings with floors that are acted upon by a characteristic imposed action larger than 5.0 kN/m^2, the above values should be increased by the difference between the average imposed action on all storeys above the one concerned and 5.0 kN/m^2.

NOTES

1 By storeys it is understood, every utilizable storey above the shelter.

2 By buildings of a particular stable construction, it is understood, buildings in which the load-bearing structures are made from reinforced *in-situ* concrete.

7.6.4 *Fire*

7.6.4.1 *General*

Possible extraordinary loads during a fire may be considered as accidental actions. Examples are loads from people along escape routes and loads on another structure from structure failing because of a fire.

7.6.4.2 *Thermal effects during fire*

The thermal effect during fire may be determined from one of the following methods:

a) Time-temperature curve and the required fire resistance (in min), and
b) An energy balance method.

If the thermal effect during fire is determined from an energy balance method, the fire load is taken to be:

$$q = 12\, t_b$$

where

q = fire action (KJ/m^2 floor), and
t_b = required fire resistance (in min) {*see* [6-1(23)]}.

NOTE – The fire action is defined as the total quantity of heat produced by complete combustion of all combustible material in the fire compartment, inclusive of stored goods and equipment together with building structures and building materials.

7.7 Vibrations

For general details on loads due to vibrations, reference may be made to Annex P.

7.8 Blast Loads

For provisions related to blast loads, *see* Annex Q.

7.9 Other Loads

Other loads not included in this Section, such as special loads due to technical process, moisture and shrinkage effects, etc, should be taken into account where stipulated by building design codes or established in accordance with the performance requirement of the structure.

7.10 For additional information regarding loads, forces and effects about cyclone resistant buildings and landslide control aspects, reference may be made to good practices [6-1(6)] and [6-1(24)] to [6-1(28)], respectively.

8 LOAD COMBINATIONS

8.1 General

A judicious combination of the loads keeping in view the probability of,

a) their acting together; and

b) their disposition in relation to other loads and severity of stresses or deformations caused by the combinations of the various loads is necessary to ensure the required safety and economy in the design of a structure.

8.2 Load Combinations

Keeping the aspect specified in **8.1**, the various loads should, therefore, be combined in accordance with the stipulation in the relevant design Codes. In the absence of such recommendations, the following loading combinations, whichever combination produces the most unfavourable effect in the building, foundation or structural member concerned may be adopted (as a general guidance). It should also be recognized in load combinations that the simultaneous occurrence of maximum values of wind, earthquake, imposed and snow loads is not likely.

a) *DL*
b) *DL* and *IL*
c) *DL* and W*L*
d) *DL* and E*L*
e) *DL* and T*L*
f) *DL, IL* and *WL*
g) *DL, IL* and *EL*
h) *DL, IL* and *TL*
j) *DL,* W*L* and *TL*
k) *DL, EL* and *TL*
m) *DL, IL, WL* and *TL*
n) *DL, IL, EL* and *TL*

(*DL* = dead load, *IL* = imposed load, *WL* = wind load, *EL* = earthquake load and *TL* = temperature load).

NOTES

1 When snow load is present on roofs, replace imposed load by snow load for the purpose of above load combinations.

2 The relevant design codes shall be followed for permissible stresses when the structure is designed by working stress method and for partial safety factors when the structure is designed by limit state design method for each of the above load combinations.

3 Whenever imposed load (*IL*) is combined with earthquake load (*EL*), the appropriate part of imposed load as specified in **5** should be used, both for evaluating earthquake effect and also for combined load effects used in such combination.

4 For the purpose of stability of the structure as a whole against overturning, the restoring moment shall be not less than 1.2 times the maximum overturning moment due to dead load plus 1.4 times the maximum overturning moment due to imposed loads. In cases where dead load provides the restoring moment, only 0.9 times the dead load shall be considered. The restoring moments due to imposed loads shall be ignored. In case of high water table, the effects of buoyancy have to be suitably taken into consideration.

5 In case of high water able, the factor of safety of 1.2 against uplift alone shall be provided.

6 The structure shall have a factor against sliding of not less than 1.4 under the most adverse combination of the applied loads/forces. In this case, only 0.9 times the dead load shall be taken into account.

7 Where the bearing pressure on soil due to wind alone is less than 25 percent of that due to dead load and imposed load, it may be neglected in design where this exceeds 25 percent, foundation may be so proportioned that the pressure due to combined effect of dead load, imposed load and wind load does not exceed the allowable bearing pressure by more than 25 percent. When earthquake effect is included, the permissible increase in allowable bearing pressure in the soil shall be in accordance with **5.**

Reduced imposed load specified in **3** for the design of supporting structures should not be applied in combination with earthquake forces.

8 Other loads and accidental load combination not included should be dealt with appropriately.

9 Crane load combinations are covered in **3.6.4**.

9 MULTI-HAZARD RISK IN VARIOUS DISTRICTS OF INDIA

9.1 Multi-hazard Risk Concept

The commonly encountered hazards are:

a) Earthquake,
b) Cyclone,
c) Wind storm,
d) Floods,
e) Landslides,
f) Liquefaction of soils,
g) Extreme winds,
h) Cloud bursts, and
j) Failure of slopes.

A study of the earthquake, wind/cyclone, and flood hazard maps of India indicate that there are several areas in the country which run the risk of being affected by more than one of these hazards.

Further there may be instances where one hazard may cause occurrence or accentuation of another hazard such as landslides may be triggered/accelerated by earthquakes and wind storms and floods by the cyclones.

It is important to study and examine the possibility of occurrence of multiple hazards, as applicable to an area. However, it is not economically viable to design all the structures for multiple hazards. The special structures such as nuclear power plants, and life line structures such as hospitals and emergency rescue shelters may be designed for multiple hazards. For such special structures, site specific data have to be collected and the design be carried out based on the accepted levels of risk. The factors that have to be considered in determining this risk are:

i) The severity of the hazard characterized by M.M. (or M.S.K.) intensity in the case of earthquake; the duration and velocity of wind in the storms; and unprotected or protected situation of flood prone areas; and

ii) The frequency of occurrence of the severe hazards.

Till such time that risk evaluation procedures are formalized, the special structures may be designed for multiple hazards using the historical data, that can be obtained for a given site and the available Code for loads already covered. The designer may have to consider the loads due to any one of the hazards individually or in combination as appropriate.

9.2 Multi-hazard Prone Areas

The criteria adopted for identifying multi hazard prone areas may be as follows:

a) *Earthquake and flood risk prone* – Districts which have seismic zone of Intensity 7 or more and also flood prone unprotected or protected area. Earthquake and flood can occur separately or simultaneously.
b) *Cyclone and flood risk prone* – Districts which have cyclone and flood prone areas. Here floods can occur separately from cyclones, but simultaneous also along with possibility of storm surge too.
c) *Earthquake, cyclone and flood risk prone* – Districts which have earthquake zone of intensity 7 or more, cyclone prone as well as flood prone (protected or unprotected) areas. Here the three hazards can occur separately and also simultaneously as in (a) and (b) above but earthquake and cyclone will be assumed to occur separately only.
d) *Earthquake and cyclone risk prone* – Districts which have earthquake zone of intensity 7 or more and prone to cyclone hazard too. The two will be assumed to occur separately.

Based on the approach given above, the districts with multi-hazard risk are given in Annex R.

9.3 Use of the List of the District with Multi-hazard Risk

The list provides some ready information for use of the authorities involved in the task of disaster mitigation, preparedness and preventive action. This information gives the districts which are prone to high risk for more than one hazard. This information will be useful in establishing the need for developing housing design to resist such multi-hazard situation.

ANNEX A

(*Foreword and Clause* 2.1)

DEAD LOADS – UNIT WEIGHTS OF CONSTRUCTION MATERIALS AND STORED MATERIALS

A-1 BUILDING MATERIALS

The unit weight of materials used in construction of buildings and structures are specified in Table 50. Also mentioned alongside are the corresponding Indian Standards, if applicable

Table 50 Unit Weight of Building Materials
(*Clause* A-1)

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
1)	**Acoustical Material** (*as per IS 2526*)			
	Eelgrass	10	5.7 to 7.65x10^{-3}	m^2
	Glass fibre	10	3.8x10^{-3}	m^2
	Hair	10	19.1x10^{-3}	m^2
	Mineral wool	10	13.45x10^{-3}	m^2
	Slag wool		2.65	m^3
	Cork		2.35	m^3
	Wood fibre board		5.9 to 6.15	m^3
	Wood particle Board			
	As per IS 3129		3.92	m^3
	As per IS 12823		4.9 to 8.83	m^3
	As per IS 3478		8.9 to 11.8	m^3
	Compressed wood wool	–	3.95 to 4.45	m^3
	Mineral/glass wool quilts and mats	–	0.157 to 0.314	m^3
	Mineral compressed glass wool tiles	–	1.87 to 2.06	m^3
	Polyester board	–	1.87 to 2.06	m^3
2)	**Aggregate, Course**			
	Broken stone ballast			
	Dry, well-shaken	–	15.70 to 18.35	m^3
	Perfectly wet	–	18.85 to 21.95	m^3
	Shingles, 3 to 38 mm	–	14.35	m^3
	Broken bricks:			
	Fine	–	14.20	m^3
	Coarse	–	9.90	m^3
	Foam slag (foundry pumice)	–	6.85	m^3
	Cinder	–	7.85	m^3
	Slag			

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Iron slag		32.4	m^3
	Steel Slag		31-36	m^3
	Copper slag		31-36	m^3
	River pebbles		15	m^3
3)	***Aggregate, Fine***			
	Sand			
	Dry, clean	–	15.10 to 15.70	m^3
	River	–	18.05	m^3
	Wet	–	17.25 to 19.60	m^3
	Brick dust (*SURKHI*)	–	9.90	m^3
	Crushed stone sand (as per IS 383)		19 to 20	m^3
	Quarry dust		14 to 20	m^3
4)	***Aggregate, Organic***			
	Saw dust, loose	–	1.55	m^3
	Peat			
	Dry	–	5.50 to 6.30	m^3
	Sandy, compact	–	7.85	m^3
	Wet, compact	–	13.35	m^3
5)	***Asbestos***			
	Felt	10	0.145	m^2
	Fibres:			
	Pressed	–	9.40	m^3
	Sprayed	10	0.02	m^2
	Natural	–	29.80	m^2
	Raw	–	5.90 to 8.85	m^2
6)	***Asbestos Cement Building Pipes*** *(see under 44 'pipes' in this table)*			
7)	***Asbestos Cement Gutters*** [as per IS1626 (Part 2)]			
	Boundary wall gutters 400×150×250 mm 450×150×300 mm 300×150×225 mm 275×125×175 mm	 12.5 12.5 12.5 10.0	 0.157 0.157 0.128 0.084	 m m m m
	Valley gutters 900×200×225 mm 600×150×225 mm 450×125×150 mm 400×125×250 mm	 12.5 12.5 12.5 12.5	 0.245 0.160 0.145 0.130	 m m m m
	Half round gutters 150 mm 250 mm 300 mm	 9.5 9.5 9.5	 0.043 0.079 0.087	 m m m
8)	***Asbestos Cement Pressure Pipes*** *(see under 44 'Pipes' in this table)*			

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
9)	***Asbestos Cement Sheeting*** (*As per* IS 459)			
	Corrugated (pitch – 146mm)	6	0.118 to 0.130	m^2
	Semi-corrugated (pitch …340mm)	6	0.118 to 0.127	m^2
	Plain	5	0.09	m^2
10)	**Bitumen**	-	0.8-1.0	m^3
11)	**Binders**			
	Fly Ash		5.05 to 7.05	m^3
	Fly Ash, Compacted		16	
	Ground Granulated blast furnace slag		14.9	m^3
	Silica fume		3.5 to 7	m^3
	Rice husk ash		1.0 to 3.5	m^3
	Metakaolin		8 to 10	m^3
12)	**Blocks**			
	Lime-based solid blocks (as per IS 3115)	–	9.8 to 12.55	m^3
	Hollow (open and closed cavity concrete blocks) [as per IS 2185 (Part 1)]			
	Grade A (load bearing)	–	14.71	m^3
	Grade B (load bearing)	–	10.80 to 14.71	m^3
	Solid concrete blocks Grade C (Load bearing)	–	17.65	m^3
	Autoclaved cellular concrete blocks (as per IS 2185 part 3)		4.42 to 9.81	m^3
13)	**Boards**			
	Cork boards (as per IS 4253)			
	Compressed		2.16 to 3.73	m^3
	Ordinary		2	m^3
	Fibre building boards (as per IS 1658)			
	Medium hardboard	6	0.028 to 0.047	m^2
		8	0.038 to 0.063	m^2
		10	0.047 to 0.078	m^2
		12	0.056 to 0.095	m^2
	Standard hardboard	3	0.024 to 0.035	m^2
		4	0.031 to 0.047	m^2
		5	0.039 to 0.059	m^2
		6	0.048 to 0.070	m^2
		8	0.062 to 0.094	m^2
	Tempered hardboard	6	0.047 to 0.071	m^2
		9	0.071 to 0.106	m^2
		3	0.0235 to 0.053	m^2

Sl No.	Materials	Nominal Size or Thickness mm	Weight kN	per
(1)	(2)	(3)	(4)	(5)
		4	0.031 to 0.047	m^2
		5	0.0391x 0.059	m^2
		7	0.054 to 0.082	m^2
	Fibre cement boards (as per IS 14862)		13 to 16	m^3
	Fibre insulation board, ordinary or flame-retardant type bitumen-bounded fibre insulation board (as per IS 3348*)*	9	0.035	m^2
		12	0.047	m^2
		18	0.071	m^2
		25	0.098	m^2
	Plain gypsum plaster boards [as per IS 2095) (Part 1)]	9.5	0.069 to 0.098	m^2
		12.5	0.093 to 0.147	m^2
		15	0.110 to 0.154	m^2
	Coated/laminated gypsum plaster boards [IS 2095 (Part 2)]	10	0.113	m^2
	Reinforced gypsum plaster boards and ceiling tiles [IS 2095 (Part 3)]	12	0.098	m^3
	Glass reinforced Gypsum		24.52	m^3
	Insulating board (*fibre*) IS 3348		43.93	m^2
	Laminated board (*fibre*) Medium density (IS 14587)		45.93	m^2
	Wood particle boards (as per IS 3087)			
	FPS-1	–	4.90 to 8.85	m^3
	FPT-2	–	4.90 to 8.85	m^3
	XPS	–	4.90 to 8.85	m^3
	XPT	–	4.90 to 8.85	m^3
	Low density wood particles boards for insulation purposes (as per IS 3129)	–	3.90	m^3
	High density wood particle boards (as per IS 3478)			
	Type 1 & 2, Grade A	–	11.77	m^3
	Type 1 & 2, Grade B	–	8.83	m^3
13)	***Bricks***			
	Common burnt clay bricks (*As per* IS 1077)	–	15.70 to 18.85	m^3
	Engineering bricks	–	21.20	m^3
	Heavy duty bricks (as per IS 2180)	–	24.50	m^3
	Pressed bricks	–	17.25 to 18.05	m^3
	Refractory bricks	–	17.25 to 19.60	m^3
	Sand cement bricks	–	18.05	m^3
	Sand lime bricks	–	20.40	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Pulverized Fuel-Ash Lime bricks (as per IS 12894)		12 to 15	m^3
	Pulverized Fuel-Ash Cement bricks (as per IS 16720)		10.8 to 19.6	m^3
	Acid resistance bricks (as per IS 4860)		22.5-24.83	m^3
	Soil based bricks (as per IS 1725)		17.5	m^3
	Perforated building bricks (as per IS 2222)		Depends on degree of perforation	m^3
	Calcium silicate bricks (as per IS 4139)		25-29	m3
14)	**Brick Chips and Broken Bricks** (*see* under 2 'Broken Bricks' in this table)			
15)	**Brick dust (SURKHI)**	–	9.00	m^3
16)	**Cast Iron, Manhole Covers** (as per IS 1726)			
	Double triangular (HD)	500	1.16	Cover
		560	1.37	Cover
	Circular (HD)	500	1.16	Cover
		560	1.37	Cover
	Circular (MD)	500	0.57	Cover
		560	0.63	Cover
	Rectangular (MD)	–	0.78	Cover
	Rectangular (LD) :			
	Single seal (Pattern 1)	–	0.23	Cover
	(Pattern 2)	–	0.15	Cover
	Double seal	–	0.28	Cover
	Square (LD):			
	Single seal	455	0.13	Cover
		610	0.25	Cover
	Double seal	455	0.23	Cover
		610	0.36	Cover
17)	**Cast Iron, Manhole Frames** (as per IS 1726)			
	Double triangular (HD)	500	1.09	Frame
		560	1.13	Frame
	Circular (HD)	500	0.83	Frame
		560	1.06	Frame
	Circular (MD)	500	0.57	Frame
		560	0.63	Frame
	Rectangular (MD)	–	0.63	Frame
	Rectangular (LD):			
	Single seal (Pattern 1)	–	0.15	Frame
	(Pattern 2)	–	0.10	Frame
	Double seal	–	0.23	Frame
	Square (LD):			
	Single seal	455	0.07	Frame

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		610	0.13	Frame
	Double seal	455	0.15	Frame
		610	0.18	Frame
18.	**Cast Iron Pipes** (*see* under 44 'Pipes' in this table)			
19.	**Cement** (as per IS 269)			
	Ordinary and aluminous	–	14.10	m^3
	Rapid-hardening	–	12.55	m^3
	Portland Pozzolana			
20.	**Ceilings**			
	Plaster on tile or concrete	13 mm thick	0.25	m^2
	Plaster on wood lath	25 mm thick	0.39	m^2
	Suspended metal lath and cement plaster	25 mm thick	0.74	m^2
	Suspended metal lath and gypsum plaster	25 mm thick	0.49	m^2
22.	**Cement Concrete, Plain**			
	Aerated		7.45	m^3
	No-fines, with heavy aggregate	–	15.70 to 18.80	m^3
	No-fines, with light aggregate	–	8.65 to 12.55	m^3
	With burnt clay aggregate	–	17.25 to 21.20	m^3
	With expanded clay aggregate	–	9.40 to 16.50	m^3
	With clinker aggregate	–	12.55 to 17.25	m^3
	With pumice aggregate	–	5.50 to 11.00	m^3
	With sand and gravel or crushed natural stone aggregate	–	22.00 to 23.50	m^3
	With saw dust	–	6.30 to 16.50	m^3
	With foamed slag aggregate	–	9.40 to 18.05	m^3
	With light weight cinder aggregate		6.4	m^3
	Light weight concrete new		8 to 20	m^3
	Cellular concrete (Foam concrete) (as per IS 6598)		3 to 5	m^3
	Transparent concrete		15	m^3
	Glass fiber reinforced concrete		18 to 22	m^3
	Geopolymer concrete		24 to 25	m^3
	Shotcrete (as per IS 9012)		22 to 25	m^3
	Ferro cement		22 to 25	m^3
	Roller compacted concrete (as per IRC SP068)		22 to 25	m^3
	Controlled low strength mix		22 to 25	m^3
	Plum concrete (as per IS 457)		22 to 25	m^3
	Autoclaved Aerated Concrete		4.42 to 9.81	m^3

Sl No.	Materials		Nominal Size or Thickness mm	Weight kN	per
(1)	(2)		(3)	(4)	(5)
	Vermiculite concrete			6 to 9	m^3
23)	**Cement Concrete, Prestressed** (conforming to IS 1343)		–	23.50	m^3
24)	**Cement Concrete, Reinforced:**				
	With 1 percent steel		–	22.75 to 24.20	m^3
	With 2 percent steel		–	23.25 to 24.80	m^3
	With 5 percent steel		–	24.80 to 26.50	m^3
25.	**Cork**		–	2.35	m^3
26.	**Damp-proofing** (*see* 28 'Felt bituminous for waterproofing and damp proofing' in Table 50)				
	Earth filling (*see* 49 'Soils and gravels in Table 50)				
27.	**Expanded Metal** (conforming to IS 412)				
	Reference No.	Size of Mesh, Nominal			
		SWM mm	LWM mm		
	1	100	250	0.030	m^2
	2	100	250	0.024	m^2
	3	100	250	0.016	m^2
	4	75	200	0.042	m^2
	5	75	200	0.032	m^2
	6	75	200	0.021	m^2
	7	40	115	0.080	m^2
	8	40	115	0.060	m^2
	9	40	75	0.060	m^2
	10	40	75	0.028	m^2
	11	40	115	0.039	m^2
	12	40	75	0.039	m^2
	13	40	115	0.020	m^2
	14	40	75	0.020	m^2
	15	25	75	0.054	m^2
	16	25	75	0.038	m^2
	17	25	75	0.028	m^2
	18	25	75	0.021	m^2
	19	20	60	0.070	m^2
	20	20	50	0.070	m^2
	21	20	60	0.050	m^2
	22	20	50	0.050	m^2
	23	20	60	0.036	m^2
	24	20	50	0.036	m^2
	25	20	50	0.021	m^2
	26	20	60	0.021	m^2

Sl No.	Materials		Nominal Size or Thickness mm	Weight kN	per
(1)	(2)		(3)	(4)	(5)
	27	12.5	50	0.050	m^2
	28	12.5	50	0.050	m^2
	29	12.5	40	0.040	m^2
	30	12.5	50	0.030	m^2
	31	12.5	40	0.030	m^2
	32	12.5	50	0.025	m^2
	33	12.5	40	0.025	m^2
	34	10	40	0.050	m^2
	35	10	40	0.035	m^2
	36	10	40	0.028	m^2
	37	95	285	0.050	m^2
	38	95	285	0.028	m^2
	39	95	285	0.020	m^2
	40	6	25	0.074	m^2
	41	6	25	0.048	m^2
	42	6	25	0.038	m^2
	43	5	20	0.050	m^2
	44	3	15	0.041	m^2
28)	**Felt, Bituminous for Waterproof, and Damp–proofing** (as per IS 1322)				
	Fibre base				
	Type 1 (under lay)		–	7.5×10^{-3}	m^2
	Type 2 (self-finished felt)		–	2.16×10^{-2}	m^2
	Hessian base				
	Type 3 (self-finished felt)				
	Grade 1		–	2.26×10^{-2}	m^2
	Grade 2		–	3.64×10^{-2}	m^2
29)	**Finishing** (*see* also 'Floor finishes' given under 31 'Flooring' and 46 'Roofing' in table 1)				
	Aluminium foil		–	Negligible	
	Plaster:				
	Acoustic		10	0.08	m^2
	Anhydrite		10	0.21	m^2
	Barium Sulphate		10	0.28	m^2
	Fibrous		10	0.09	m^2
	Gypsum or lime		10	0.19	m^2
	Hydraulic lime or cement		10	0.23	m^2
	Plaster ceiling on wire netting		10	0.26	m^2
	NOTE – When wood or metal lathing is used, add		–	0.06	
30)	**Float Glass** (as per IS 2835) **and Safety Glass** (as per IS 2553)				
	Sheet		2	0.049	m^2
			2.5	0.062	m^2
			3	0.074	m^2
			4	0.098	m^2

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		5	0.123	m^2
		5.5	0.134	m^2
		6.5	0.167	m^2
		8	0.200	m^2
		10	0.250	m^2
		12	0.300	m^2
		15	0.375	m^2
		19	0.475	m^2
31)	**Flooring**			
	Asphalt flooring	10	0.22	m^2
	NOTE – For macadam finish, add	10	0.26	m^2
	Compressed cork	10	0.04	m^2
	Floors, structural:			
	Hollow clay blocks including reinforcement and mortar jointing between blocks, but excluding any concrete topping	100	1.47	m^2
		125	1.67	m^2
		150	1.86	m^2
		175	2.16	m^2
		200	2.55	m^2
	NOTE – Add extra for concrete topping			
	Hollow clay blocks including reinforcement and concrete ribs between blocks, but excluding any concrete topping	100	1.18	m^2
		115	1.27	m^2
		125	1.37	m^2
		140	1.47	m^2
		150	1.57	m^2
		175	1.76	m^2
		200	1.96	m^2
	NOTE – Add extra for concrete topping.			
	Hollow concrete units including any concrete topping necessary for constructional purposes	100	1.67	m^2
		125	1.96	m^2
		150	2.16	m^2
		175	2.35	m^2
		200	2.65	m^2
		230	3.14	m^2
	Floors, wood:			
	Hard wood	22	0.16	m^2
		28	0.20	m^2
	Soft wood	22	0.11	m^2
		28	0.13	m^2
	Weight of mastic used in laying wood block flooring	–	0.015	m^2
	NOTE – All thicknesses are 'finished thicknesses'.			
	Floor finishes:			
	Clay floor tiles (as per IS 1478)	12.5 to 25.4	0.10 to 0.2	m^2

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	NOTE – This weight is 'as laid' but excludes screeding.			
	Magnesium oxychloride:			
	Normal type (saw dust filler)	10	0.142	m^2
	Heavy duty type (mineral filler)	10	0.216	m^2
	Parquet flooring	–	0.08 to 0.12	m^2
	Rubber (*As per* IS : 809	3.2	0.048 to 0.062	m^2
		4.8	0.070 to 0.09	m^2
		6.4	0.093 to 0.130	m^2
	Terra cotta, filled 'as laid'	–	5.54 to 7.06	m^2
	Terrazzo paving 'as laid'	10	0.23	m^2
32)	**Foam Slag, Foundry Pumice**	–	6.85	m^3
33)	**Gutters, Asbestos Cement** (*see* under *7* '**Asbestos cement gutter**' in this table)			
34)	***Gypsum***			
	Gypsum mortar (As per IS 2547)	–	18.6	m^3
	Gypsum powder (As per IS 12679)	–	13.89 to 17.25	m^3
35)	***Iron***			
	Pig	–	70.60	m^3
	Gray, cast	–	68.95 to 69.90	m^3
	White, cast	–	74.30 to 75.70	m^3
	Wrought	–	75.70	m^3
36)	***Lime***			
	Lime concrete with burn clay Aggregate (as per IS 2541)	–	18.80	m^3
	Lime mortar	–	15.70 to 18.05	m^3
	Lime plaster (as per IS 2394)	–	17.25	m^3
	Lime stored in lumps, uncalcined	–	12.55 to 14.10	m^3
	Lime, unslaked, freshly burnt in pieces	–	8.60 to 10.20	m^3
	Lime slaked, fresh	–	5.70 to 6.30	m^3
	Lime slaked, after 10 days	–	7.85	m^3
	Lime, unslaked (KANKAR)	–	11.55	m^3
	Lime, slaked (KANKAR)	–	10.00	m^3
37)	***Linoleum*** (as per IS 653)			
	Sheets and tiles	4.5	0.0569	m^2
		3.2	0.0402	m^2
		2.0	0.0265	m^2
		1.65	0.0215	m^2
38)	***Masonry***			
	Brick Masonry (excluding plaster)			
	With common burnt clay bricks (as per IS1077)	–	19.25	m^3
	Engineering bricks	–	21.1	m^3
	Glazed bricks	–	20.40	m^3
	Pressed bricks	–	18.65	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Block Masonry			
	With lime based solid blocks		13.55	m^3
	Hollow concrete blocks			
	Grade A		15.4	m^3
	Grade B		12.00	m^3
	Solid Concrete Blocks		18.15	m^3
	Autoclaved cellular concrete blocks		9.1 to 11.1	m^3
39.	***Masonry, Stone***			
	Cast	–	21.75	m^3
	Dry rubble	–	20.40	m^3
	Granite ashlar	–	25.00	m^3
	Granite rubble	–	23.55	m^3
	Lime stone ashlar	–	25.10	m^3
	Marble dressed	–	26.50	m^3
	Sand stone	–	22.00	m^3
40)	***Mastic asphalt***	10	0.216	m^2
41)	***Metal sheeting, protected galvanized steel sheets, plain and corrugated*** (as per IS 277)			
	Class A (corrugated)	1.60	0.131	m^2
		1.00	0.104	m^2
		0.9	0.084	m^2
		0.80	0.069	m^2
		0.63	0.056	m^2
	Class B (corrugated)	1.60	0.129	m^2
		1.00	0.102	m^2
		0.9	0.083	m^2
		0.80	0.067	m^2
		0.63	0.054	m^2
	Class C (corrugated)	1.60	0.128	m^2
		1.00	0.101	m^2
		0.9	0.081	m^2
		0.80	0.066	m^2
		0.63	0.053	m^2
	Class D (corrugated)	1.60	0.127	m^2
		1.00	0.100	m^2
		0.9	0.081	m^2
		0.80	0.065	m^2
		0.63	0.052	m^2
42)	***Mortar***			
	Cement	–	20.40	m^3
	Gypsum	–	11.75	m^3
	Lime	–	15.70 to18.05	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight kN	per
(1)	(2)	(3)	(4)	(5)
44)	***Pipes***			
	Asbestos cement pipes as per IS 1626 (Part 1)	50	0.032 to 0.034	m
		60	0.032 to 0.043	m
		80	0.051 to 0.054	m
		90	0.052 to 0.060	m
		100	0.058 to 0.065	m
		125	0.072 to 0.086	m
		150	0.086 to 0.108	m
	Asbestos cement pressure pipes as per IS 1592	50	0.056	m
		80	0.067	m
		100	0.090	m
		125	0.139	m
		150	0.175	m
		200	0.264	m
		250	0.380	m
		300	0.539	m
	Cast iron pipes (as per IS 1230)			m^3
	Standard overall length 1.8 m with socket	50	0.073	pipe
		75	0.108	pipe
		100	0.137	pipe
		125	0.196	pipe
		150	0.255	pipe
	Standard overall length 1.5 m with socket	50	0.064	pipe
		75	0.093	pipe
		100	0.123	pipe
		125	0.172	pipe
		150	0.230	pipe
	Pressure pipes for water, gas and sewage:			
	a) Centrifugally cast (as per IS 1536)			
	Socket and spigot pipes			
	Barrel: Class LA	80	0.145	m
		100	0.182	m
		125	0.237	m
		15O	0.297	m
		200	0.432	m
		250	0.582	m
		300	*0.750*	m
		350	0.944	m
		400	1.146	m
		450	1.383	m
		500	1.620	m
		600	2.156	m

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		700	2.718	m
		750	3.111	m
	Class A	80	0.151	m
		100	0.201	m
		125	0.259	m
		150	0.326	m
		200	0.472	m
		250	0.637	m
		300	0.824	m
		350	1.030	m
		400	1.262	m
		450	1.530	m
		500	1.775	m
		600	2.367	m
		700	3.056	m
		750	3.422	m
	Class B	80	0.172	m
		100	0.216	m
		125	0.281	m
		150	0.352	m
		200	0.511	m
		250	0.692	m
		300	0.896	m
		350	1.112	m
		400	1.368	m
		450	1.657	m
		500	1.929	m
		600	2.578	m
		700	3.317	m
		750	3.733	m
	Sockets for Class LA, Class A and Class B barrels	80	0.054	Socket
		100	0.069	Socket
		125	0.090	Socket
		150	0.113	Socket
		200	0.165	Socket
		250	0.225	Socket
		300	0.292	Socket
		350	0.368	Socket
		400	0.454	Socket
		450	0.549	Socket
		500	0.647	Socket
		600	0.876	Socket

Sl No.	**Materials**	**Nominal Size or Thickness** mm	**Weight**	
			kN	per
(1)	(2)	(3)	(4)	(5)
		700	1.145	Socket
		750	1.292	Socket
	Flanged pipe with screwed flanges:			
	Barrel:			
	Class A	80 to 300	Same as for centrifugally cast socket and spigot piles, Class A	
	Class B	80 to 300	Same as for centrifugally cast socket and spigot piles, Class B	
	Flanges for Class A and Class B barrels	80	0.042	Flange
		100	0.049	Flange
		125	0.065	Flange
		150	0.080	Flange
		200	0.112	Flange
		250	0.144	Flange
		300	0.182	Flange
	a) Vertically cast socket and spigot pipes (as per IS 1537)			
	Barrel:	80	Same as for centrifugally cast socket and spigot pipes, Class A	
		to		
		750		
	Class A 7.15kg/dm^3	800	3.82	m
		900	4.65	m
		1000	5.59	m
		1100	6.59	m
		1200	7.67	m
		1500	11.98	m
	Class B	80	–	–
		to	–	–
		750	–	–
		800	–	m
		900	5.07	m
		1000	6.07	m
		1100	7.23	m
		1200	8.35	m
		1500	13.07	m
	Socket of Class A and Class B barrels	80	–	–
		to	–	–
		750	–	–
		800	–	Socket
		900	179	Socket

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		1000	218	Socket
		1100	260	Socket
		1200	360	Socket
		1500	491	Socket
	b) Sand cast (flanked pipes)			
	Barrel:			
	Class A	80 to 750 800 to 1500	Same as for centrifugally cast socket and spigot pipes, Class A –	
	Class B	80 to 750	Same as for vertically cast socket and spigot pipes, Class B	
		800 to 1500		
	Flanges for Class A and Class B barrels	80	0.036	Flange
		100	0.041	Flange
		125	0.052	Flange
		150	0.066	Flange
		200	0.091	Flange
		250	0.117	Flange
		300	0.145	Flange
		350	0.186	Flange
		400	0.229	Flange
		450	0.250	Flange
		500	0.315	Flange
		600	0.431	Flange
		750	0.587	Flange
		700	0.685	Flange
		800	0.792	Flange
		900	0.928	Flange
		1000	1.18	Flange
		1100	1.38	Flange
		1200	1.70	Flange
		1500	2.71	Flange
	Concrete pipes (as per IS 458)			
	Class NP 1 (unreinforced non- pressure pipes)	80	0.19	m
		100	0.22	m
		150	0.30	m
		250	0.40	m
		300	0.69	m
		350	0.84	m
		400	0.95	m

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		450	1.17	m
	Class NP 2 (reinforced concrete, light duty, non- pressure pipes)	80	0.196	m
		100	0.215	m
		150	0.324	m
		250	0.510	m
		300	0.736	m
		350	0.902	m
		400	1.02	m
		450	1.26	m
		500	1.38	m
		600	1.89	m
		700	2.19	m
		800	2.81	m
		900	3.51	m
	Class NP2 (reinforced concrete, light duty, non-pressure pipes)	1000	4.30	m
		1100	5.15	m
		1200	6.09	m
		1400	8.18	m
		1600	9.93	m
		1800	12.58	m
	Class NP3 (reinforced concrete, heavy duty, non-pressure pipes)	350	2.35	m
		400	2.63	m
		450	2.91	m
		500	3.19	m
		600	4.02	m
		700	4.61	m
		800	5.92	m
		900	7.39	m
		1000	8.13	m
		1100	10.34	m
		1200	11.18	m
	Class P1 (reinforced concrete pressure pipes safe for 20 MPa pressure tests	80	0.196	m
		100	0.235	m
		150	0.324	m
		250	0.510	m
		300	0.736	m
		350	0.902	m
		400	1.02	m
		450	1.26	m
		500	1.38	m
		600	1.89	m
		700	2.19	m

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		800	2.81	m
		900	3.51	m
		1000	4.30	m
		1100	5.15	m
		1200	6.09	m
	Class P2 (reinforced concrete pressure pipes safe for 40 MPa pressure tests)	80	0.196	m
		100	0.235	m
		150	0.324	m
		250	0.608	m
		300	1.01	m
		350	1.31	m
		400	1.67	m
		450	1.84	m
		500	1.56	m
		600	3.20	m
	Class P3 (reinforced concrete pressure pipes safe for 60 MPa pressure tests)	80	0.196	m
		100	0.235	m
		150	0.324	m
		250	0.736	m
		300	1.15	m
		350	1.65	m
		400	2.04	m
	Lead pipes [*As per* IS 404 (Part 1)] (service and distribution pipes 10 be laid underground) :			
	For working 40 MPa	10	0.018	m
		13	0.031	m
		20	0.042	m
		25	0.060	m
		30	0.074	m
		40	0.091	m
		50	0.142	m
	For working 70 MPa	10	0.022	m
		15	0.038	m
		20	0.050	m
		25	0.069	m
		32	0.126	m
		40	0.175	m
	For working 100 MPa	10	0.029	m
		15	0.048	m
		20	0.067	m
		25	0.105	m

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Service pipes to be fixed or laid above ground:			
	For working pressure 40 MPa	10	0.014	m
		15	0.021	m
		20	0.027	m
		25	0.036	m
		32	0.059	m
		40	0.091	m
		50	0.142	m
	For working pressure 70 MPa	10	0.018	m
		15	0.024	m
		20	0.030	m
		25	0.069	m
		32	0.126	m
		40	0.175	m
	For working pressure 100 MPa	10	0.029	m
		15	0.048	m
		20	0.067	m
		25	0.105	m
	Cold water distribution pipes to be fixed or laid above ground:			
	For working pressure 25 MPa	10	0.014	m
		15	0.021	m
		20	0.027	m
		25	0.036	m
		32	0.048	m
		40	0.067	m
		50	0.084	m
	For working pressure 40 MPa	10	0.014	m
		15	0.021	m
		20	0.027	m
		25	0.036	m
		32	0.059	m
		40	0.091	m
		50	0.142	m
	Hot water distribution pipes to be fixed or laid above ground:			
	For working pressure 20 MPa	10	0.015	m
		15	0.023	m
		20	0.031	m
		25	0.041	m
		32	0.062	m
		40	0.082	m

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
		50	0.142	m
	For working pressure 35 MPa	10	0.015	m
		15	0.027	m
		20	0.045	m
		25	0.085	m
		32	0.132	m
	Soil, waste, and soil and waste ventilation pipes	50	0.050	m
		75	0.073	m
		100	0.097	m
		150	0.160	m
	Flushing and warning pipes	20	0.020	m
		25	0.025	m
		32	0.032	m
		40	0.039	m
		50	0.049	m
	Gas pipes:			
	Heavy weight gas pipes	10	0.008	m
		15	0.017	m
		20	0.025	m
		25	0.034	m
		32	0.045	m
		40	0.061	m
		50	0.071	m
	Light weight gas pipes	10	0.008	m
		15	0.012	m
		20	0.020	m
		25	0.029	m
		32	0.037	m
		40	0.047	m
		50	0.058	m
	Stoneware, salt-glazed pipes (as per IS 651)	100	0.137	m
		150	0.216	m
		200	0.324	m
		230	0.412	m
		250	0.510	m
		300	0.775	m
		350	0.980	m
		400	1.26	m
		450	1.44	m
		500	1.77	m
		600	2.35	m

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
45.	***Plaster***			
	Cement	10	0.204	m^2
	Lime 17.25x 10/1000	10	0.172	m^2
	Acoustic	10	0.078	m^2
	Anhydrite	10	0.206	m^2
	Barium sulphate	10	0.284	m^2
	Fibrous	10	0.088	m^2
	Gypsum	10	0.186	m^2
46.	***Roofing***			
	Asbestos cement sheeting			
	(*see* 'Asbestos cement sheeting' in Table 1)			
	Allahabad tiles (single) including battens (*see* Note below)	–	0.83	m^2
	Allahabad tiles (double) including battens (*see* Note below)	–	1.67	m^2
	Country tiles (single) including battens (*see* Note below)	–	0.69	m^2
	Country tiles (double) including battens (*see* Note below)	–	1.18	m^2
	Mangalore tiles battens (*see* Note below)	–	0.64	m^2
	Mangalore tiles bedded in mortar over flat tiles (*see* Note below)	–	1.08	m^2
	Mangalore tiles with flat tiles (*see* Note below)	–	0.78	m^2
	Copper sheet roofing including laps and rolls {	0.56	0.08	m^2
	Flat roofs:	0.72	0.10	m^2
	Clay tiles hollow (*see* 31 'Flooring' in this Table)			
	Concrete hollow precast (*see* 31 'Flooring' in this table)			
	Galvanized iron sheeting (*see* 'Metal sheeting, protected' in Table 50)			
	Glazed Roofing:			
	Glazing with aluminium alloy bars for spans up to 3 m	6.4	0.19	m^2
	Glazing with lead-covering steel bars at 0.6 m centres	6.4	0.25 to 0.28	m^2
	States on battens	–	0.34 to 0.49	m^2
	Thatch with battens	–	0.34 to 0.49	m^2
	NOTE – Weights acting vertically on horizontally projection to be multiplied by cosine of roof angle to obtain weights normal to the roof surface.			
	Specification for clay flooring, tiles (*first revision*).			

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Specification for rubber flooring materials for general purposes (*first revision*).			
	Roof finished			
	Bitumen macadam	10	0.22	m^2
	Felt roofing (*see* 28 'Felt bituminous for water-proofing and damp-proofing' in Table 1)	10	0.008	m^2
	Glass silk quilted	0.5	0.05	m^2
	Lead sheet	0.8	0.07	m^2
	Mortar screeding	10	0.21	m^2
47.	***Sheeting***			
	Asbestos (*see* under 9 'asbestos cement sheeting' in this table)			
	Galvanized iron (*see* under 39 'metal sheeting, protected' in this table)			
	Glass (*see* under 30 'Glass' in this table)			
	Plywood as per IS 303	1	0.007	m^3
48.	***Slugwool***	–	2.65	m^3
49.	***Soils and Gravels***			
	Alluvial ground, undisturbed	–	15.69	m^3
	Broken stone ballast:			
	Dry, well-shaken	–	15.70 to 18.35	m^3
	Perfectly wet	–	18.85 to 21.95	m^3
	Chalk		15.70 to 18.85	m^3
	Clay:			
	China, compact	–	21.95	m^3
	Clay fills:			
	Dry, lumps	–	10.20	m^3
	Dry, compact	–	14.10	m^3
	Damp, compact	–	17.25	m^3
	Wet , compact	–	20.40	m^3
	Undisturbed	–	18.85	m^3
	Undisturbed, gravelly	–	20.40	m^3
	Crush stone sand (Msand)	–		
	Earth:			
	Dry	–	13.85 to 18.05	m^3
	Moist	–	15.70 to 19.60	m^3
	Gravel:			
	Loose	–	1570	m^3
	Rammed	–	18.85 to 21.20	m^3
	Kaolin, compact	–	25.50	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Loam:			
	Dry, loose	–	11.75	m^3
	Dry, compact	–	15.70	m^3
	Wet, compact	–	18.85	m^3
	Loess, dry	–	14.10	m^3
	Marl, compact	–	17.25 to 18.85	m^3
	Mud, river, wet	–	17.25 to 18.85	m^3
	Peat:			
	Dry	–	5.50 to 6.30	m^3
	Sandy, compact	–	7.85	m^3
	Wet, compact	–	13.35	m^3
	Rip-rap	–	12.55 to 14.10	m^3
	Sand:			
	Dry, clean	–	15.10 to 15.70	m^3
	River	–	18.05	m^3
	Wet	–	17.25 to 19.60	m^2
	Shingles:			
	Aggregate 3 to 38 mm	–	13.75	m^2
	Fine sand:			
	Dry	–	15.70	m^2
	Saturated	–	20.40	m^2
	Silt, wet	–	17.25 to 18.85	m^2
50	***Steel sections***			
	Density of steel sections shall be taken as 78.50 kg/m^3			
	Weight of hot rolled sections (ISJB, ISLB, ISMB, ISWB, ISNPB, ISWPB, ISSC, ISHB, ISJC, ISLC, ISMC, ISMPC, ISA, ISPBP) shall be referred from IS 808.			
	Weight of hollow steel sections shall be referred from IS 4923.			
	Weight of cold formed light gauge structural steel sections shall be referred from IS 811.			
	Weight of rolled and slit T bars shall be referred from IS 1173.			
	Weight of steel sheet piling sections shall be referred from IS 2314.			
51.	***Stone***			
	Agate	–	25.50	m^3
	Aggregate	–	15.70 to 18.85	m^3
	Basalt	–	27.95 to 29.05	m^3
	Cast	–	21.95	m^3
	Chalk	–	21.50	m^3
	Dolomite	–	28.25	m^3
	Emery	–	39.25	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Flint	–	25.40	m^3
	Gneiss	–	23.55 to 26.40	m^3
	Granite	–	25.90 to 27.45	m^3
	Loose	–	15.70	m^3
	Moderately rammed, dry	–	18.85	m^3
	Green stone	–	28.25	m^3
	Gypsum	–	21.95 to 23.55	m^3
	Kota			
	Laterite	–	20.40 to 23.55	m^3
	Lime stone	–	23.55 to 25.90	m^3
	Marble	–	26.70	m^3
	Pumice	–	7.85 to 11.00	m^3
	Quartz rock	–	25.90	m^3
	Sand stone	–	21.95 to 23.54	m^3
	Slate	–	27.45	m^3
	Soap stone	–	26.45	m^3
52.	***Tar, Coal***			
	Crude (*As per* IS 212)	–	9.90	m^3
	Naphtha, light (*As per* IS 213)	–	9.90	m^3
	Naphtha, heavy	–	9.90	m^3
	Road tar (*As per* IS 215)	–	9.90	m^3
	Pitch (*As per* IS 216)	–	9.90	m^3
53.	***Thermal Insulation***			
	Unbonded glass wool	–	12.75 to 23.55	m^3
	Unbonded rock and slag wool	–	11.30 to 19.60	m^3
	Cellular concrete			
	Grade A	–	Up to 29.40	m^3
	Grade B	–	29.50 to 39.20	m^3
	Grade C	–	39.30 to 49.00	m^3
	Performed calcium silicate Insulation (for temperature up to 650 ºC)	–	19.60 to 34.30	m^3
54.	***Tiles and stone cladding***			
	Terra cotta	–	18.35 to 23.25	m^2
	Terrazzo			

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Paving	10	0.24	m^2
	Cast partitions	40	0.93	m^2
	Tiles			
	Mangalore pattern (*As per* IS 654)	–	0.02 to 0.03	Tile
	Polystyrene wall tiles (*As per* IS 3463)	99 x 99	0.013	m^2
		148.5 x 148.5	0.013	m^2
	Pressed ceramic tiles, glazed and unglazed (as per IS 15622)		22	m^3
	Concrete paving blocks (as per IS 15658)		24	m^3
	Note -The unit of timbers correspond to average unit of typical Indian timbers at 12 percent moisture content			
55.	***Timber***			
	Typical Indian timbers (as per IS 399)			
	Aglaia	–	8.34	m^3
	Aini	–	5.83	m^3
	Alder	–	3.63	m^3
	Amari	–	6.13	m^3
	Amla	–	7.85	m^3
	Amra	–	4.41	m^3
	Anjan	–	8.33	m^3
	Arjun	–	7.99	m^3
	Ash	–	7.06	m^3
	Axlewood	–	8.82	m^3
	Babul	–	7.70	m^3
	Baen	–	7.70	m^3
	Bahera	–	7.99	m^3
	Bakota	–	4.21	m^3
	Balasu	–	7.55	m^3
	Ballagi	–	11.13	m^3
	Bamboo			
	Banati	–	4.41	m^3
	Benteak	–	6.62	m^3
	Ber	–	6.91	m^3
	Bhendi	–	7.55	m^3
	Bijasal	–	7.85	m^3
	Black chuglam	–	6.13	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Birch	–	7.85	m^3
	Black locust	–	8.34	m^3
	Blue gum	–	8.34	m^3
	Blue pine	–	5.05	m^3
	Bola	–	6.42	m^3
	Bonsum	–	5.20	m^3
	Bullet wood	–	8.78	m^3
	Casuarina	–	8.34	m^3
	Cettis	–	6.42	m^3
	Champ	–	4.85	m^3
	Chaplash	–	5.05	m^3
	Chatian	–	4.07	m^3
	Chikrassy	–	6.62	m^3
	Chilauni	–	6.42	m^3
	Chilla	–	7.85	m^3
	Chir	–	5.64	m^3
	Chuglam:			
	Black	–	7.85	m^3
	White (silver grey-wood)	–	6.91	m^3
	Cinnamon	–	6.42	m^3
	Cypress	–	5.05	m^3
	Debdaru	–	6.28	m^3
	Deodar	–	5.35	m^3
	Devdam	–	7.06	m^3
	Dhaman:			
	Grewia tiliofolia	–	7.70	m^3
	Grewia vestita	–	7.40	m^3
	Dhup	–	6.42	m^3
	Dilenia	–	6.13	m^3
	Dudhi	–	5.49	m^3
	Ebony	–	8.19	m^3
	Elim	–	5.20	m^3
	Eucalyptus	–	8.33	m^3
	Figs	–	4.56	m^3
	Fir	–	4.14	m^3
	Frash	–	6.62	m^3
	Gamari	–	5.05	m^3
	Gardenia	–	7.40	m^3
	Garuga	–	5.98	m^3
	Geon	–	4.07	m^3
	Gluta	–	7.06	m^3
	Gokul	–	4.07	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Grewia sp.	–	7.55	m^3
	Gurjan	–	7.70	m^3
	Gutel	–	4.41	m^3
	Haldu	–	6.62	m^3
	Hathipaila	–	5.84	m^3
	Hiwar	–	7.70	m^3
	Hollock	–	5.98	m^3
	Hollong	–	7.21	m^3
	Hoom	–	7.21	m^3
	Horse chestnut	–	5.05	m^3
	Imli	–	8.97	m^3
	Indian Chestnut	–	6.28	m^3
	Indian Hemlock	–	3.92	m^3
	Indian Oak	–	8.48	m^3
	Indian Olive	–	10.35	m^3
	Irul	–	8.33	m^3
	Jack	–	5.83	m^3
	Jaman	–	7.70	m^3
	Jarul	–	6.13	m^3
	Jathtkai	–	5.05	m^3
	Jhingan	–	5.63	m^3
	Jutili	–	7.85	m^3
	Kadam	–	4.85	m^3
	Kail	–	5.05	m^3
	Kaim	–	6.42	m^3
	Kambli	–	4.07	m^3
	Kanchan	–	6.62	m^3
	Kanjuj	–	5.84	m^3
	Karada	–	8.34	m^3
	Karal	–	7.99	m^3
	Karani	–	6.28	m^3
	Karar	–	5.34	m^3
	Kardahi	–	9.27	m^3
	Karimgotta	–	3.92	m^3
	Kasi	–	5.83	m^3
	Kasum	–	10.84	m^3
	Kathal	–	5.85	m^3
	Keora	–	6.13	m^3
	Khair	–	9.00	m^3
	Khasipine	–	5.05	m^3
	Kindal	–	7.55	m^3
	Kokko	–	6.28	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Kongoo	–	9.76	m^3
	Kuchla	–	8.63	m^3
	Kumbi	–	7.70	m^3
	Kurchi	–	5.20	m^3
	Kurung	–	9.76	m^3
	Kusum	–	11.28	m^3
	Kuthan	–	4.71	m^3
	Lakooch	–	6.28	m^3
	Lambapatti	–	5.34	m^3
	Lampati	–	5.05	m^3
	Laurel	–	8.33	m^3
	Lendi	–	7.40	m^3
	Machilus			
	Gamblei	–	5.05	m^3
	Macrantha	–	5.20	m^3
	Maharukh	–	4.07	m^3
	Mahogany	–	6.62	m^3
	Mahua	–	8.97	m^3
	Maina	–	5.64	m^3
	Makai	–	3.14	m^3
	Malabar neem	–	4.41	m^3
	Mango	–	6.77	m^3
	Maniawga	–	7.40	m^3
	Maple	–	5.64	m^3
	Mesua	–	9.76	m^3
	Milla	–	9.12	m^3
	Mokha	–	7.99	m^3
	Mulberry	–	6.62	m^3
	Mullilam	–	7.21	m^3
	Mundani	–	6.77	m^3
	Murtenga	–	7.70	m^3
	Myrabolan	–	9.27	m^3
	Narikel	–	5.49	m^3
	Nedunar	–	5.05	m^3
	Oak	–	8.48	m^3
	Padauk	–	7.06	m^3
	Padri	–	7.06	m^3
	Palang	–	5.98	m^3
	Pali	–	6.28	m^3
	Papita	–	3.28	m^3
	Parrotia	–	8.48	m^3
	Persian lilac	–	5.84	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	Piney	–	6.13	m^3
	Ping	–	8.97	m^3
	Pinus insignis	–	6.13	m^3
	Pipli	–	5.83	m^3
	Pitraj	–	6.77	m^3
	Poon	–	6.42	m^3
	Poplar	–	4.41	m^3
	Pula	–	3.78	m^3
	Pyinma	–	3.98	m^3
	Rajbrikh	–	8.48	m^3
	Red sanders	–	10.84	m^3
	Rohini	–	11.33	m^3
	Rosewood (black wood)	–	8.19	m^3
	Rudrak	–	4.71	m^3
	Sal	–	8.48	m^3
	Salai	–	5.64	m^3
	Sandal wood	–	8.97	m^3
	Sadan	–	8.34	m^3
	Satin wood	–	9.41	m^3
	Saykaranji	–	7.40	m^3
	Seleng	–	4.85	m^3
	Semul	–	3.78	m^3
	Silver oak	–	6.28	m^3
	Siris	–	3.92	m^3
	Kala-siris	–	7.21	m^3
	Safed- siris	–	6.28	m^3
	Siaso	–	7.70	m^3
	Spruce	–	4.71	m^3
	Suji	–	2.65	m^3
	Sundri	–	9.41	m^3
	Talauma	–	5.64	m^3
	Tanaku	–	2.09	m^3
	Teak	–	6.28	m^3
	Toon	–	5.05	m^3
	Udal	–	2.50	m^3
	Upas	–	3.14	m^3
	Uriam	–	7.40	m^3
	Vakai	–	9.41	m^3
	Vellapine	–	5.83	m^3
	Walnut	–	5.64	m^3
	White bombwe	–	5.98	m^3
	White cedar	–	7.06	m^3

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
	White chuglam (silver grey-wood)	–	6.91	m^3
	White dhup	–	4.22	m^3
	Yon	–	8.33	m^3
	NOTE – The unit of timbers correspond to average unit of typical Indian timbers at 12 percent moisture content.			
56.	***Water***			
	Fresh	–	10.0	m^3
	Salt	–	10.05	m^3
57.	***Wood – Wool Building Slabs***	10mm	0.059	m^2
58.	***Walling*** (IS 6072)			
	Autoclaved reinforced cellular concrete wall slabs			
	Class A	–	8.35 to 9.80	m^2
	Class B	–	7.35 to 8.35	m^2
	Class C	–	6.35 to 7.35	m^2
	Class D	–	5.40 to 6.35	m^2
	Class E	–	4.40 to 5.40	m^2
	Brick masonry (*see* 38 ‘Masonry Brick’ in Table 1)			
	blocks masonry (*see* 11 ’Block’ in Table 1)			
	Stone masonry (*see* 37 ‘Masonry Stone’ in Table 1)			
	Partitions 39			
	Brick wall	100	1.91	m^2
	Cinder concrete	75	1.13	m^2
	Galvanized on sheet	–	0.15	m^2
	Hollow glass block (bricks)	100	0.88	m^2
	Hollow blocks per 200 mm of thickness			
	Ballast or stone concrete	200	0.201	m^2
	Clay	200	0.201	m^2
	Clinker concrete	200	0.220	m^2
	Coke breeze concrete	200	9.176	m^2
	Diatomaceous earth	200	0.093	m^2
	Gypsum	200	0.137	m^2
	Pumice concrete	200	0.177	m^2
	Slag concrete, air-cooled	200	0.196	m^2
	Slag concrete foamed	200	0.186	m^2
	Lath and plaster	–	0.392	m^2
	Solid blocks per 20 mm of thickness			
	Ballast or stone	20	0.451	m^2
	Clinker concrete	20	0.300	m^2
	Coke breeze concrete	20	0.221	m^2
	Pumice concrete	20	0.221	m^2
	Slag concrete, foamed	20	0.250	m^2
	Terrazzo cast partitions	40	0.932	m^2
	Timber studding plastered	–	9.981	m^2

Sl No.	**Materials**	**Nominal Size or Thickness** mm	**Weight**	
			kN	per
(1)	(2)	(3)	(4)	(5)
NOTE – For unit weight of fixtures and fittings required to buildings including builder's hardware, reference may be made to appropriate Indian standards Specification for autoclaved reinforced cellular concrete wall slabs.				

A-2 STORED AND MISCELLANEOUS MATERIALS

A-2.1 Units weights of stored and miscellaneous materials intended for dead load calculation and other general purposes are given in Table 51.

Table 51 Unit Weights of Stored and Miscellaneous Materials
[Clauses 1.1.1 *(Note) and* 3.1*]*

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
1)	***Agriculture and Food Products***		
	Butter	8.45	–
	Coffee in bags	5.50	–
	Drinks in bottle, in boxes	7.35	–
	Eggs, packed	2.95	–
	Fats, oil	5.80	–
	Fish meal	4.90	45
	Flour in sacks up to 1 m height	2.20 to 5.90	–
	Forage (bales)	1.25	–
	Fruits	3.45	–
	Grains:		
	Barley	6.75	27
	Corn, shelled	7.55	27
	Flax seed	7.35	30
	Oats	5.30	30
	Rice	6.55	33
	Soyabeans	7.35	30
	Wheat	8.15	28
	Wheat flour	6.85	30
	Grain sheaves up to 4 m stack height	0.98	30
	Grain sheaves over 4 m stack height	1.45	30
	Grass and clover	3.45	–
	Hay:		–
	Compressed	1.65	–
	Loose up to about 3 m stack height	0.69	–
	Honey	14.10	–
	Hops:		
	In stacks	1.65	–

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	In cylinder hop bins	4.60	–
	Sewn up or compressed in cylindrical shape in hop cloth	2.85	–
	Malt:		
	Crushed	3.90	20
	Germinated	1.85	–
	Meat and meat products	7.05	–
	Milk	10.05	–
	Molasses	4.40	–
	Onion in bags	5.40	0
	Oil cakes, crushed	5.80	0
	Potatoes	7.05	30
	Preserves (tins in cases)	4.90 to7.85	–
	Bags	7.05	–
	Bulk	9.40	–
	Seeds:		
	Heaps	4.90 to 7.85	25
	Sacks	3.90 to 6.85	–
	Straw and chaff:		
	Loose up to about 3 m stack height	0.45	–
	Compressed	1.65	–
	Sugar:		
	Crystal	7.35	30
	Cube sugar in boxes	7.85	–
	Sugar beet, pressed out	7.85	–
	Tobacco bundles	3.45	–
	Vinegar	10.40	–
2)	***Chemicals and Allied Materials***		
	Acid, hydrochloric	11.75	–
	Acid, nitric 91%	14.80	–
	Acid, sulphuric 87%	17.55	–
	Alcohol	7.65	–
	Alum, pearl in barrel	5.20	–
	Ammonia, liquid	8.85	–
	Ammonium, chloride, crystalline	8.15	30-40
	Ammonium nitrate	7.05 to 9.80	25
	Ammonium sulphate	7.05 to 9.00	32-45
	Beeswax	9.40	–
	Benzole	8.90	–
	Benzene hexachloride	8.75	–
	Bicarbonate of soda	6.40	–
	Bone	18.65	–
	Borax	17.15	–
	Calcite	26.50	–
	Camphor	9.70	–

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Carbon disulphide	12.75	–
	Casein	13.25	–
	Caustic soda	13.85	–
	Creosole	10.50	–
	Dicalcium phosphate	6.65	–
	Disodium phosphate	3.90 to 4.80	30-45
	Iodine	48.55	–
	Oils in bottle or barrels	5.70 to 8.90	–
	Oil, linsee:		
	In barrels	5.70	–
	In drums	7.05	–
	Oil, turpentine	8.50	–
	Paints	9.40	–
	Paraffin wax	7.85 to 9.40	–
	Petroleum	9.90	–
	Phosphorus	17.85	–
	Plastics and polymers:		
	Cellulose acetate	12.25 to 13.35	–
	Cellulose nitrate	13.25 to 15.70	–
	Methyl methacrylate (as per IS 14753)	11.60	–
	Phenol formaldehyde	12.55	–
	Expanded Polystyrene (as per IS 4671)	0.15 to 0.35	–
	Extruded polystyrene	0.21	
	Polyvinyl chloride (Perspex)	11.75 to 13.25	–
	Resin bonded sheet	12.85 to 13.55	–
	Urea formaldehyde	13.25 to 13.55	–
	Extruded polystyrene	0.21	
	Polyurethane	0.59 to 0.66	
	Polyethylene (as per IS 2508)	8.8 to 9.6	
	Polyvinyl Butaryl	11	
	Polycarbonate (as per IS 14443)	12	
	Ethylene tetrafluoroethylene	17	
	Polytetrafluoroethylene (as per IS 14635)	22	
	Potash	14.40	–
	Potassium	8.65	–
	Potassium nitrate	9.90	–
	Red lead, dry	20.70	–
	Red lead, paste	87.30	–
	Rosin in barrels	6.75	–
	Rubber:		
	Raw	8.90 to 9.40	–
	Vulcanized	8.90 to 9.10	–
	Saltpeter	9.91	–

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Sodium silicate in barrels	8.35	–
	Sulphur	20.10	–
	Talc	27.45	–
	Varnishes	9.40	–
	Vitriol, blue, in barrels	7.05	–
3)	***Fuels***		
	Brown coal	6.83	–
	Brown coal briquettes heaped	7.85	35
	Brown coal briquettes stacked	12.75	–
	Charcoal	2.95	–
	Coal:		
	Untreated, mine-moist	9.80	35
	In washeries	11.75	0
	Dust	6.85	25
	All other sorts	8.35	35
	Coke:		
	Furnace or gas	4.90	35
	Brown coal, low-temperature	9.80	35
	Hard, raw coal	8.35	35
	Hard, raw coal, mine-damp	9.80	35
	Diesel oil	9.40	0
	Firewood, chopped	3.90	45
	Petrol	6.75	0
	Wood in chips	1.95	45
	Wood shaving, loose	1.45	35
	Wood shaving, shaken down	2.45	35
4)	***Manures***		
	Animal manures:		
	Loosely heaped	11.75	45
	Stacked dung up to 25 m stack height	17.65	45
	Artificial manures	11.75	24-30
5)	***Metals and Alloys***		
	Aluminium	25.30 to 26.60	
	Cast	25.30 to 26.60	–
	Wrought	25.90 to 27.45	–
	Sheet per mm of thickness per m^2	0.028	–
	Antimony, pure:		
	Amorphous	60.90	–
	Solid	65.70	–
	Bismuth:		
	Liquid	98.07	–
	Solid	95.02 to 97.9	–
	Cadmium:		
	Cast	83.75 to 84.05	–
	Wrought	85.03	–

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Calcium	15.60	–
	Chromium	63.95 to 66.00	–
	Cobalt:		
	Cast	83.25 to 85.10	–
	Wrought	88.45	–
	Copper:		
	Cast	86.20 to 87.65	–
	Wrought	86.70 to 87.65	–
	Sheet per mm of thickness	0.09	–
	Gold:		
	Cast	188.75 to 189.55	–
	Wrought	189.55	–
	Iron:		
	Pig	70.60	–
	Grey, cast	68.95 to 69.90	–
	White, cast	74.35 to 75.70	–
	Wrought	75.50	–
	Lead:		
	Cast	111.20	–
	Liquid	105.00	–
	Wrought	111.40	–
	Sheet per mm of thickness	0.11	–
	Magnesium	16.45 to 17.15	–
	Manganese	72.55	–
	Mercury	133.35	–
	Nickel	81.20 to 87.20	–
	Platinum	210.25	–
	Silver:		
	Cast	102.0 to 102.85	–
	Liquid	93.15	–
	Wrough	103.35 to 103.55	–
	Sodium:		
	Liquid	9.10	–
	Solid	9.30	–
	Tungsten	188.30	–
	Uranium	180.45	–
	Zinc		
	Cast	68.95 to 70.20	–
	Wrought	70.50	–
	Sheet per mm of thickness	0.07	–
	Alloys:		
	Aluminium and copper		
	Aluminium 10% copper 90%	75.40	–
	Aluminium 5% copper 95%	82.00	–
	Aluminium 3% copper 97%	85.10	–

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Aluminium 91% zinc 9%	27.45	–
	Babbit metal tin 90%	71.70	–
	Lead 5% copper 5%		
	Wood's metal bismuth 50%	95.00	–
	Lead 25% cadmium 12.5%		
	Tin 12.5%		
	Brasses:		
	Muntz metal (copper 60%, zinc 40%)	80.60	–
	Red (copper 90%, zinc 10%)	84.25	–
	White (copper 50%, zinc 50%)	80.30	–
	Yellow (copper 70%, zinc 30%)		
	Cast	82.75	–
	Drawn	85.10	–
	Rolled	83.85	–
	Bronzes:		
	Bell metal (copper 80%, tin 20%)	85.60	–
	Gun metal (copper 90%, tin 10%)	86.10	–
	Cadmium and tin	75.40	–
	German Silver:		
	Copper 52%, zinc 26%, nickel 22%	82.75	–
	Copper 59%, zinc 30%, nickel 11%	81.70	–
	Copper 63%, zinc 30%, nickel 7%	81.40	–
	Gold and Copper:		
	Gold 98%, copper 2%	184.75	–
	Gold 90%, copper 10%	168.20	–
	Lead and Tin:		
	Lead 87.5%, tin 12.5%	103.85	–
	Lead 30.5%, tin 69.5%	81.10	–
	Monel metal cast (nickel 70%, copper 30%)	87.00	–
	Steel:		
	Cast	77.00	–
	Wrought	76.30	–
	Black plate per mm of thickness	0.08	–
	Steel sections (*see* 46 'steel sections' in Table 1)		
6)	***Miscellaneous Materials***		

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Aggregate, coarse	10.80 to 15.70	30
	Ashes, coal, dry, 12 mm and under	5.50 to 6.30	40
	Ashes, coal, dry, 75 mm and under	5.50 to 6.30	38
	Ashes, coal, wet 12 mm and under	7.05 to 7.85	52
	Ashes, coal, wet 75 mm and under	7.05 to 7.85	50
	Asphalt, crushed 12 mm and under	7.05	30-45
	Ammonium nitrate, polls	3.55 to 8.35	27
	Bone	18.65	–
	Books and files, stacked	8.35	–
	Calcium ammonium nitrate	9.80	28
	Copper sulphate, ground	11.75	30
	Chalk	21.95	–
	Chinaware, earthenware, stacked (including cavities)	10.80	–
	Clinker, furnace, clean	7.85	30
	Diammonium phosphate	7.85 to 8.50	29
	Double salt (ammonium sulphate nitrate)	7.05 to 9.30	34
	Filling cabinet and cupboards with contents in records offices, libraries, archives	5.90	–
	Flue dust, boiler house, dry	5.50 to 7.05	30
	Fly ash, pulverised	5.50 to 7.05	–
	Glass, solid	23.50 to 26.70	–
	Wool	0.16 to 1.18	–
	In sheets	25.50	–
	Glue	12.55	–
	Gypsum, calcined, 12mm and under	8.60 to 9.40	40
	Gypsum, calcined, powdered	9.40 to 12.55	45
	Gypsum, raw, 25 mm and under	14.10 to 15.70	30-45
	Hides:		
	Dry, Salted, Ice } Only green	8.65	–
		8.90	–
	Leather put in rows	7.85	–
	Lime, ground 3 mm and under	9.40	>45
	Lime, hydrated 3 mm and under	6.30	30-45
	Lime, hydrated, pulverized	5.00 to 6.30	30-45
	Lime pebble	8.25 to 8.75	
	Limestone, agricultural 3 mm and under	10.60	
	Limestone, crushed	13.30 to 14.10	30-45
	Limestone dust	8.65 to 14.90	30-45

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Magnesite, caustic, in powder form	7.85	–
	Magnesite sinter and Magnesite granular	19.60	–
	Phosphate, rock, pulverized	9.40	40-52
	Phosphate rock	11.75 to 13.35	30-45
	Phosphate sand	14.10 to 15.70	30-45
	Potassium carbonate	7.95	30-45
	Potassium chloride, pellets	18.85 to 20.40	30-45
	Potassium nitrate	4.85	>30
	Potassium sulphate	6.55 to 7.45	45
	Pyrites, pellets	18.85 to 20.40	30-45
	Pumice	5.80 to 9.90	–
	Rubbish:		
	Building	13.80	–
	General	6.30	–
	Salt, common, dry, coarse	6.30 to 10.00	30-45
	Salt, common, dry, fine	11.00 to 12.55	30-45
	Salt cake, dry, coarse	13.35	30
	Salt cake, dry, pulverized	11.20 to 13.35	35
	Sand, bank. damp	17.25 to 20.40	45
	Sand, bank, dry	14.10 to 17.25	30
	Sand, silica, dry	14.10 to 15.70	30-45
	Saw dust	1.57	30
	Silica gel	4.40	30-45
	Soda ash, heavy	8.67 to 10.20	35
	Soda ash, light	4.70 to 6.00	37
	Sodium nitrate, granular	11.00 to 12.55	24
	Sulphur, crushed, 12 mm and under	7.85 to 8.25	35-45
	Sulphur, 76 mm and under	8.65 to 13.35	32
	Sulphur, powdered	7.85 to 9.40	30-45
	Single superphosphate (S.S.P), granulated	7.65 to 8.25	37
	Slag, furnace, crushed	14.90	35
	Steel goods:		
	Cylinders, usually stored for Carbonic acid, etc	13.80	
	Sheets, railway rails, etc, usually stored	44.00	
	Trisodium phosphate	9.40	30-45
	Triple superphosphate	7.85 to 8.65	30-45
	Turf	2.85 to 5.70	–
	Urea, prills	6.40	23-26
7)	***Ores***		
	Antimony	29.80	–

Sl. No.	**Materials**	**Weight** kN/m^3	**Angle of Friction,** Degress
(1)	(2)	(3)	(4)
	Ferrous sulphide	26.50	–
	Ferrous sulphide ore waste after roasting	13.85	–
	Iron ore, compact storing	29.80	–
	Magnesium ore	19.60	–
8)	***Textiles, Paper and Allied Materials***		
	Cellulose in bundles	7.35	–
	Cotton, compressed	12.75	–
	Flax, piles and compressed in bales	2.95	–
	Furs	8.80	–
	Jute in bundles	6.85	–
	Paper:		
	In bundles and rolls	6.85	–
	Newspaper in bundles	3.90	–
	Put in rows	10.80	–
	Thread in bundles	4.90	–
	Wood, compressed	12.75	–

ANNEX B

[*Clause* 3.3.2.1(b)]

ILLUSTRATIVE EXAMPLE SHOWING REDUCTION OF UNIFORMLY DISTRIBUTED IMPOSED FLOOR LOADS IN MULTI-STOREYED BUILDINGS FOR DESIGN OF COLUMNS

B-1 The total imposed loads from different floor levels (including the roof) coming on the central column of a multi-storeyed building (with mixed occupancy) is shown in Fig. 14. Calculate the reduced imposed load for the design of column members at different floor levels using **3.3.2.1**. Floor loads do not exceed 5.0 kN/m^2.

B-1.1 Applying reduction coefficients in accordance with **3.3.2.1**, total reduced floor loads on the column at different levels is indicated along with Fig. 14.

FLOOR NO. FROM TOP INCLUDING ROOF	IMPOSED FLOOR LOADS ON COLUMNS AT DIFFERENT FLOORS, kN	DISCOUNTED IMPOSED LOADING ON COLUMNS, kN
1	30	
2	40	30
3	50	(30 + 40) (1 - 0.1) = 63
4	50	(30 + 40 + 50) (1 - 0.2) = 96
5	40	(30 + 40 + 50 + 50) (1 - 0.3) = 119
6	45	(30 + 40 + 50 + 50 + 40) (1 - 0.4) = 126
7	50	(30 + 40 + 50 + 50 + 40 + 45) (1 - 0.4) = 153
8	50	(30 + 40 + 50 + 50 + 40 + 45 + 50) (1 - 0.4) = 183
9	40	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50) (1 - 0.4) = 213
10	40	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40) (1 - 0.4) = 237
11	40	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40) (1 - 0.4) = 261
12	55	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40) (1 - 0.5) = 237.5 < 261 adopt 261 for design
13	55	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55) (1 - 0.5) = 265
14	70	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55 + 55) (1 - 0.5) = 292.5
15	80	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55 + 55 + 70) (1 - 0.5) = 327.5
		(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55 + 55 + 70 + 80) (1 - 0.5) = 367.5

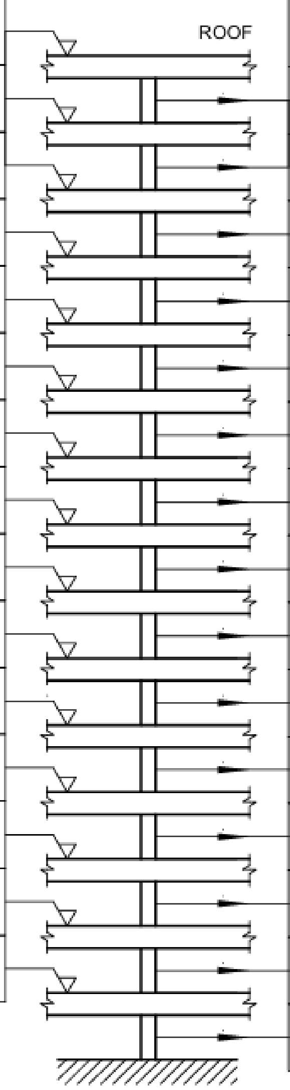


FIG. 14 LOADING DETAILS

ANNEX C
(*Clause* 4.2)

NOTATIONS

A	=	Surface area of a structure or part of a structure
A_e	=	Effective frontal area
A_z	=	Effective frontal area of the building at height z
b	=	Breadth of a structure or structural member normal to the wind stream in the horizontal plane
B_s	=	Background factor
C_d	=	Drag coefficient
C_f	=	Force coefficient
C_{fn}	=	Normal force coefficient
C_{ft}	=	Transverse force coefficient
C_f'	=	Frictional drag coefficient
C_p	=	Pressure coefficient
C_{pe}	=	External pressure coefficient
C_{pi}	=	Internal pressure coefficient
C_{fs}	=	Cross-wind force spectrum coefficient
$C_{f,z}$	=	Drag force coefficient of the building corresponding to the area A_z
C	=	Coefficient, which depends on θ_s, used in the evaluation of k_3 factor
d	=	Depth of a structure or structural member parallel to wind stream in the horizontal plane
d_w	=	Wake width
D	=	Diameter of cylinder or sphere
E	=	Wind energy factor

F_z	=	Along wind load on the building/structure at any height *z*
F	=	Force normal to the surface
f_a	=	First mode natural frequency of the building/structure in along wind direction, in Hz
f_c	=	First mode natural frequency of the building/structure in across wind direction, in Hz
f_s	=	Vortex shedding frequency
F_n	=	Normal force
F_t	=	Transverse force
F'	=	Frictional force
G	=	Gust factor
g_R	=	Peak factor for resonant response
g_v	=	Peak factor for upwind velocity fluctuations
h	=	Height of structure above mean ground level
h_x	=	Height of development of a velocity profile at a distance *x* down wind from a change in terrain category
H_s	=	Height factor for resonant response
H	=	Height above mean ground level on the topography feature
I	=	Turbulence intensity
$I_{h,i}$	=	Turbulence intensity at height *h* in terrain category *i*
$I_{z,i}$	=	Turbulence intensity at height *z* in terrain category *i*
IF	=	Interference factor
k	=	Mode shape power exponent
k_1, k_2, k_3, k_4	=	Wind speed modification factors
$\overline{k}_{2,i}$	=	Hourly mean wind speed factor

K	=	Force coefficient multiplication factor for individual members of finite length
K_a	=	Area averaging factor
K_c	=	Combination factor
K_d	=	Wind directionality factor
l	=	Length of the member or larger horizontal dimension of a building
L	=	Actual length of upwind slope
L_e	=	Effective length of upwind slope
L_h	=	Integral turbulence length scale at the height, h
m_0	=	Average mass per unit height of the structure
M_a	=	Design peak along wind base bending moment
M_c	=	Design peak across wind base bending moment
N	=	Effective reduced frequency
p_d	=	Design wind pressure
p_z	=	Wind pressure at height z
$\overline{p}_d$	=	Design hourly mean wind pressure corresponding to $\overline{V}_{z,d}$
p_e	=	External pressure
p_i	=	Internal pressure
r	=	Roughness factor which is twice the longitudinal turbulence intensity at height h, $I_{h,i}$
R_e	=	Reynolds number
s	=	Level on a building/structure for the evaluation of along wind load effects
s_0	=	Factor, which depends on H and X, used for the evaluation of k_3 factor

S_t	=	Strouhal number
S	=	Size reduction factor
V_b	=	Regional basic wind speed
V_z	=	Design wind speed at height z
$\overline{V}_d$	=	Design hourly mean wind speed
$\overline{V}_{z,d}$	=	Design hourly mean wind speed at height z
$\overline{V}_{z,H}$	=	Hourly mean wind speed at height z
w	=	Lesser horizontal dimension of a building, or a structural member
w'	=	Bay width in multi-bay building
$\hat{\ddot{x}}$	=	Peak acceleration at the top of the building/structure in along wind direction, in m/s^2;
x	=	Distance down wind from a change in terrain category
X	=	Distance from the summit or crest of topography feature relative to the effective length, L_e
$\hat{\ddot{y}}$	=	Peak acceleration at the top of the building/structure in across wind direction
z	=	Height or distance above the ground
$z_{0,i}$	=	Aerodynamic roughness height for i^{th} terrain
Z	=	Effective height of the topography feature
α	=	Inclination of the roof to the horizontal
β	=	Damping coefficient of the building/structure
η	=	Shielding factor
ϕ	=	Factor to account for the second order turbulence intensity
Φ	=	Solidity ratio

Φ_e	=	Effective solidity ratio
ε	=	Average height of the surface roughness
θ_s	=	Upwind slope of the topography feature in the wind direction
θ	=	Wind angle from a given axis

ANNEX D
(*Clause* 4.4.2)

BASIC WIND SPEED AT 10 m HEIGHT FOR SOME IMPORTANT CITIES/TOWNS

City/Town	Basic Wind Speed m/s	City/Town	Basic Wind Speed m/s
Agra	47	Kanpur	47
Ahmedabad	39	Kohima	44
Ajmer	47	Kolkata	50
Almora	39	Kozhikode	39
Amritsar	50	Kurnool	39
Asansol	47	Lakshadweep	39
Aurangabad	39	Lucknow	50
Bahraich	50	Ludhiana	50
Bengaluru	33	Madurai	39
Barauni	47	Mandi	39
Bareilly	50	Mangaluru	39
Bhatinda	47	Moradabad	50
Bhilai	44	Mumbai	44
Bhopal	47	Mysuru	33
Bhubaneshwar	50	Nagpur	44
Bhuj	50	Nainital	47
Bikaner	47	Nasik	39
Bokaro	39	Nellore	50
Chandigarh	50	Panaji	39
Chennai	50	Patiala	50
Coimbatore	39	Patna	47
Cuttack	50	Puducherry	50
Darbhanga	47	Port Blair	44
Darjeeling	47	Pune	39
Dehra Dun	39	Raipur	44
Delhi	50	Rajkot	39
Durgapur	47	Ranchi	39
Gangtok	47	Roorkee	50
Guwahati	50	Rourkela	39
Gaya	39	Simla	39
Gorakhpur	47	Srinagar	39
Hyderabad	44	Surat	44
Imphal	44	Tiruchirrappalli	47
Jabalpur	39	Thiruvananthapuram	39
Jaipur	47	Udaipur	47
Jamshedpur	47	Vadodara	39
Jhansi	47	Varanasi	47
Jodhpur	47	Vijaywada	50
		Visakhapatnam	50

ANNEX E

[*Clause* 4.4.3.2.4 (2) (ii)]

CHANGES IN TERRAIN CATEGORIES

E-1 LOW TO HIGH TERRAIN CATEGORY NUMBER

In cases of transition from a low terrain category number (corresponding to a low terrain roughness) to a higher terrain category number (corresponding to a rougher terrain), the velocity profile over the rougher terrain shall be determined as follows:

a) Below height h_x, the velocities shall be determined in relation to the rougher terrain; and

b) Above height h_x, the velocities shall be determined in relation to the less rough (more distant) terrain.

E-2 HIGH TO LOW TERRAIN CATEGORY NUMBER

In cases of transition from a more rough to a less rough terrain, the velocity profile shall be determined as follows:

a) Above height h_x, the velocities shall be determined in accordance with the rougher (more distant) terrain; and

b) Below height h_x, the velocity shall be taken as the lesser of the following:

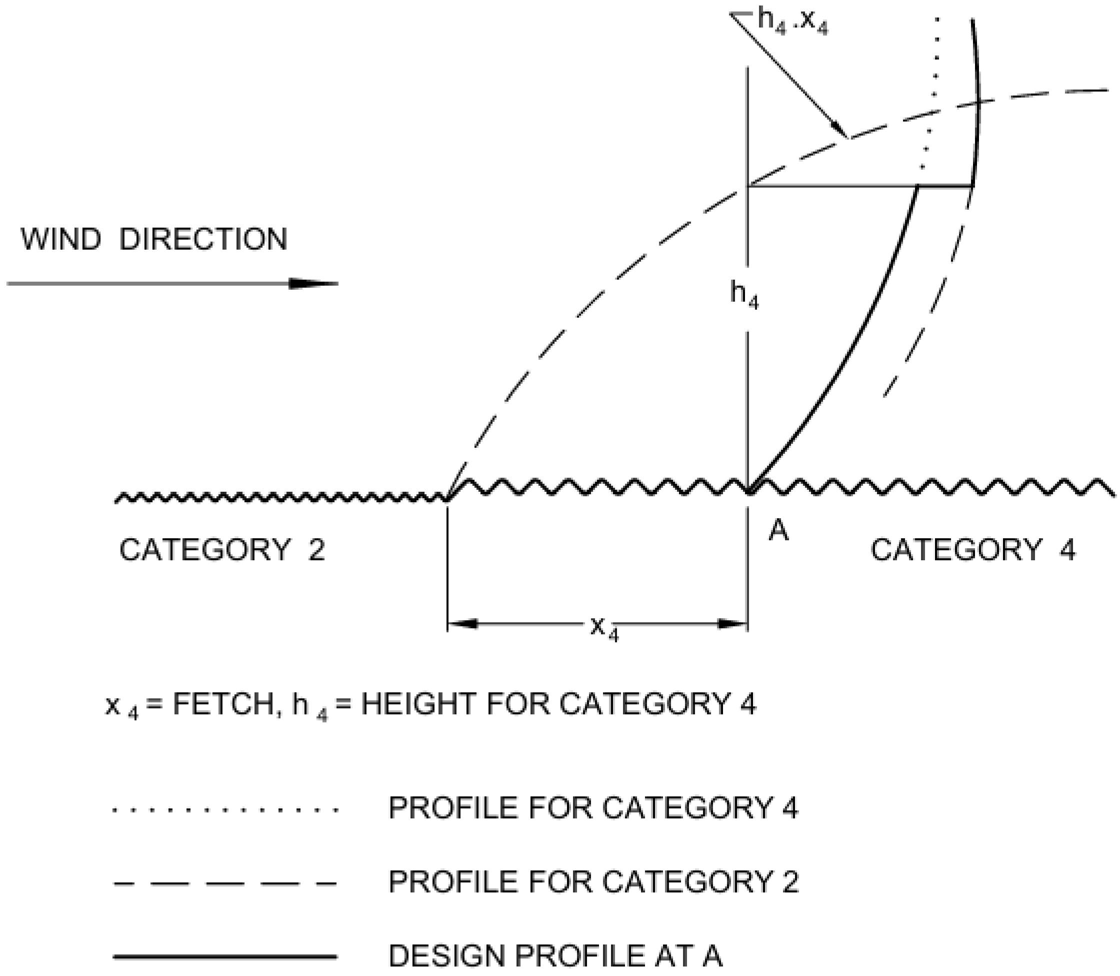
1) That determined in accordance with the less rough terrain, and
2) The velocity at height h_x as determined in relation to the rougher terrain

NOTE ─ Examples of determination of velocity profiles in the vicinity of a change in terrain category are shown in Fig. 15A and Fig. 15B.

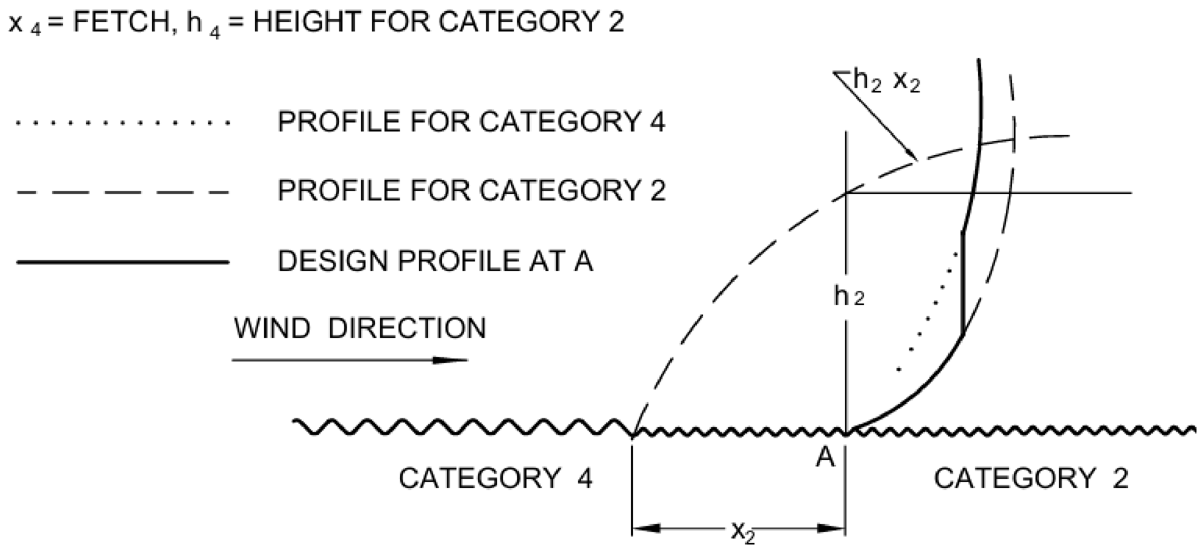
E-3 MORE THAN ONE CATEGORY

Terrain changes involving more than one category shall be treated in similar way to that described in **E-1** and **E-2**.

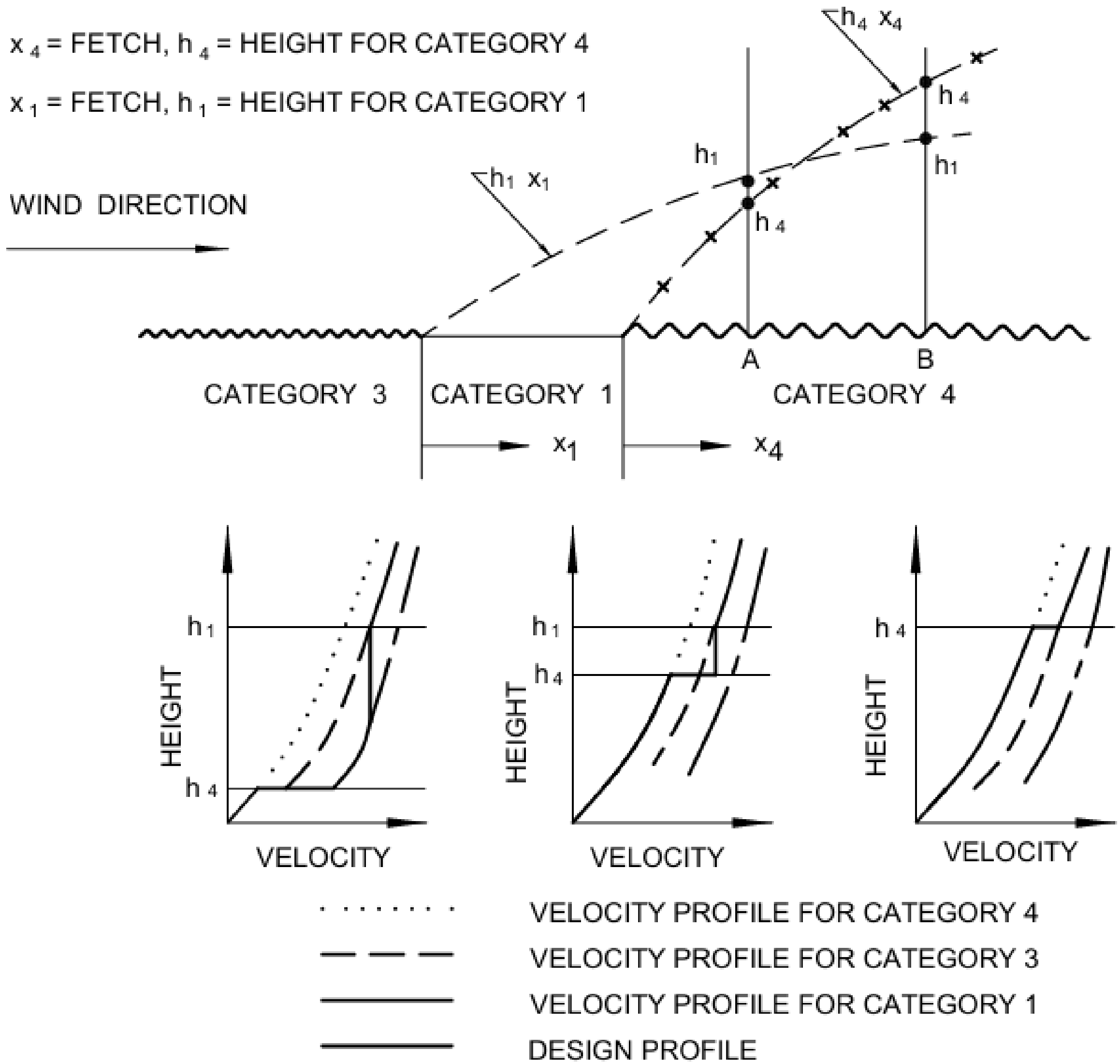
NOTE ─ Examples involving three terrain categories are shown in Fig. 15B.



15A DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGORY (Less rough to more rough)



15B DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGORY (More rough to less rough)



15C DETERMINATION OF DESIGN PROFILE INVOLVING MORE THAN ONE CHANGE IN TERRAIN CATEGORY

FIG. 15 VELOCITY PROFILES IN THE VICINITY OF A CHANGE IN TERRAIN CATEGORY

ANNEX F
(*Clause* 4.4.3.3)

EFFECT OF A CLIFF OR ESCARPMENT ON EQUIVALENT HEIGHT ABOVE GROUND (k_3 FACTOR)

F-1 The influence of the topographic feature is considered to extend 1.5 L_e upwind and 2.5 L_e downwind of the summit of crest of the feature where L_e is the effective horizontal length of the hill depending on slope as indicated below (*see* Fig. 16):

Slope	L_e
$3° < \theta_s \leq 17°$	L
$\theta_s > 17°$	$Z / 0.3$

where

L = actual length of the upwind slope in the wind direction,

Z = effective height of the topography feature, and

θ_s = upwind slope in the wind direction.

In case, the zone in downwind side of the crest of the feature is relatively flat ($\theta_s < 3°$) for a distance exceeding L_e, then the feature should be treated as an escarpment. Otherwise, the feature should be treated as a hill or ridge. Examples of typical features are given in Fig. 16.

NOTES

1 No difference is made, in evaluating k_3 between a three dimensional hill and two dimensional ridge.

2 In undulating terrain, it is often not possible to decide whether the local topography to the site is significant in terms of wind flow. In such cases, the average value of the terrain upwind of the site for a distance of 5 km should be taken as the base level from wind to assess the height, Z, and the upwind slope θ, of the feature.

F-2 TOPOGRAPHY FACTOR, k_3

The topography factor k_3 is given by the following:

$$k_3 = 1 + C\, s_0$$

where C has the following values:

Slope	*C*
$3° < \theta_s \leq 17°$	1.2 (Z/L)

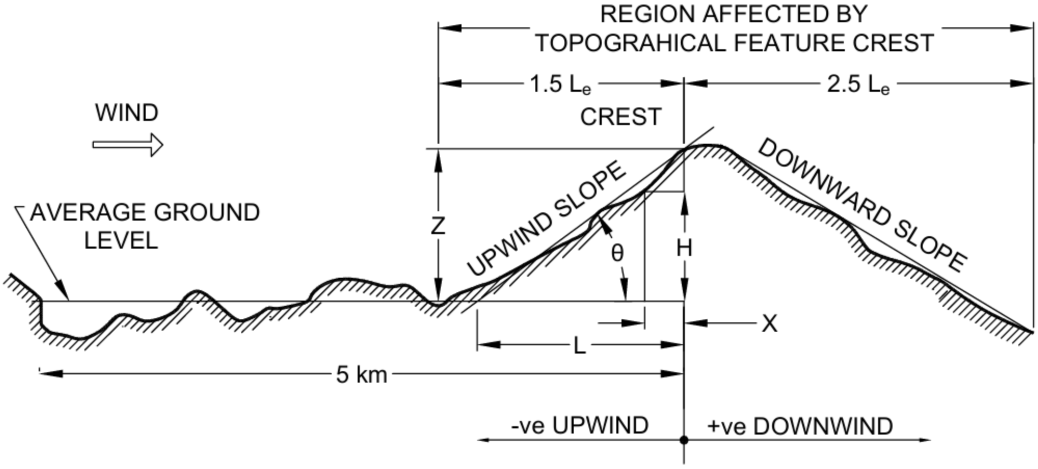
$\theta_s > 17°$ 0.36

and s_0 is a factor derived in accordance with **F-2.1** appropriate to the height, *H* above mean ground level and the distance, *X*, from the summit or crest relative to the effective length, L_e.

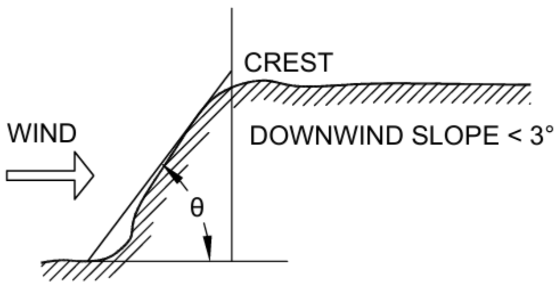
F-2.1 The factor, s_0 should be determined from:

a) Figure 17 for cliffs and escarpments, and
b) Figure 18 for ridges and hills.

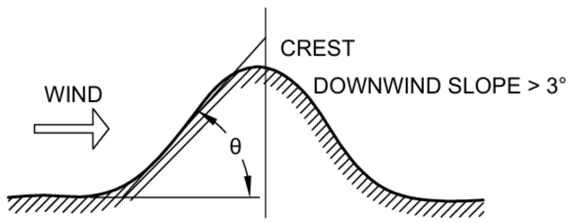
NOTE – Where the downwind slope of a hill or ridge is more than 3°, there will be large regions of reduced accelerations or even shelter and it is not possible to give general design rules to cater for these circumstances. Values of s_0 from Fig. 18 may be used as upper bound values.



16A General Notations



16B CLIFF AND ESCARPMENT



16C HILL AND RIDGE

Fig. 16 TOPOGRAPHICAL DIMENSIONS

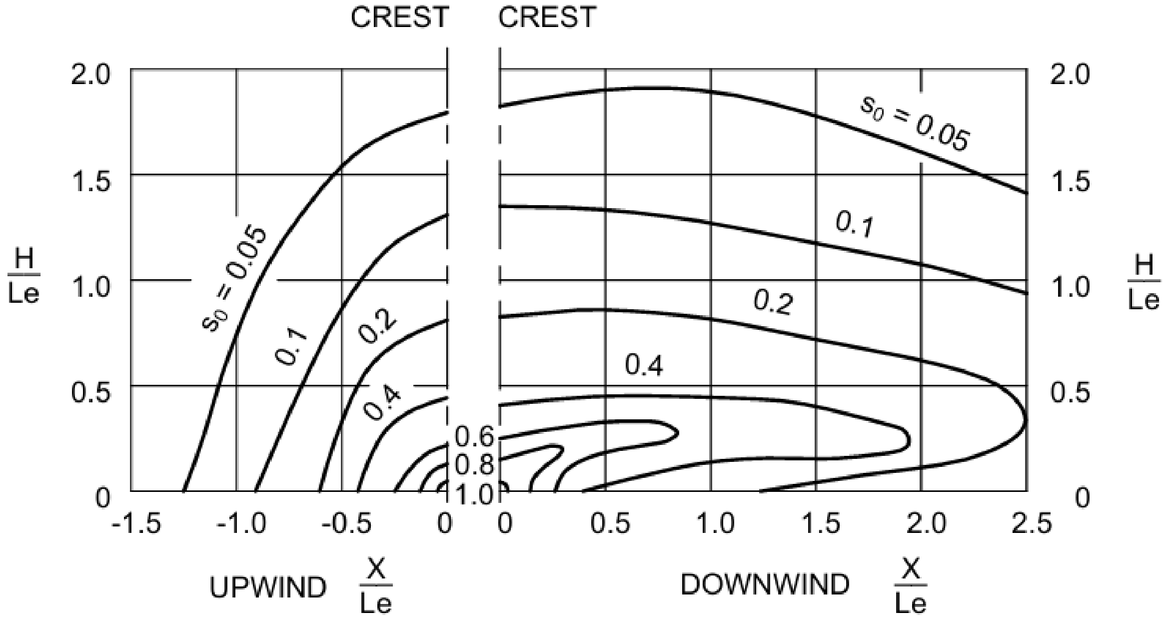


FIG. 17 FACTOR s_0 FOR CLIFF AND ESCARPMENT

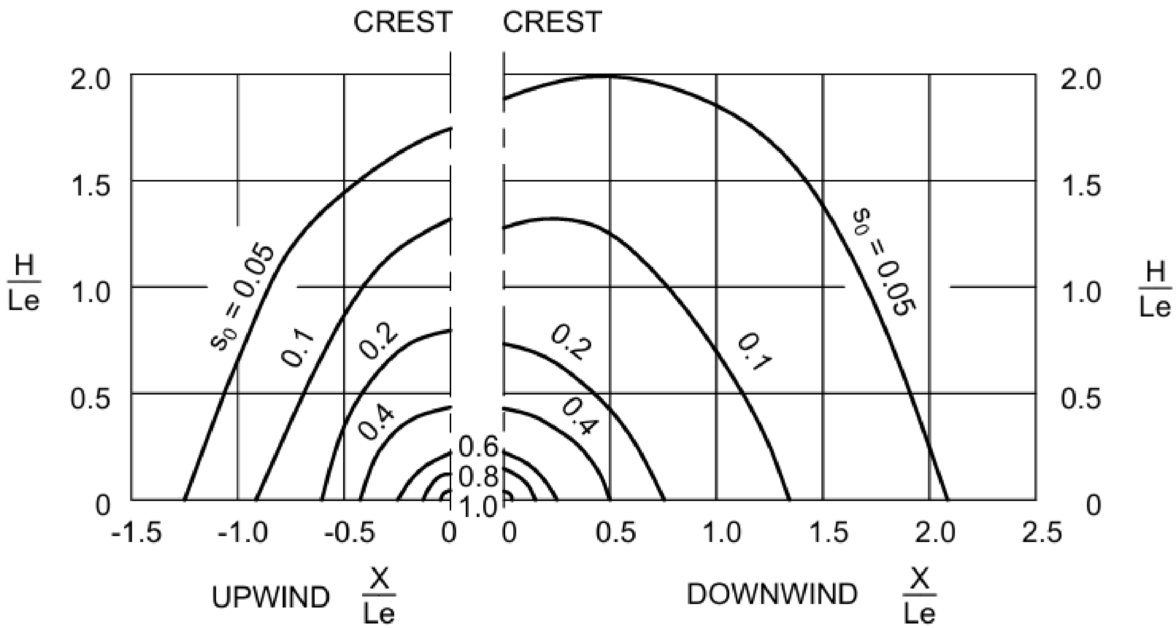


FIG. 18 FACTOR s_0 FOR RIDGE AND HILL

ANNEX G
[*Clauses* 4.5.4.2.2, 4.5.4.3.2 (c), and 4.5.4.3.3]

WIND FORCE ON CIRCULAR SECTIONS

G-1 The wind force on any object is given by:

$$F = C_f A_e p_d$$

where

C_f = force coefficient;
A_e = effective area of the object normal to the wind direction; and
p_d = design pressure of the wind.

For most shapes, the force coefficient remains approximately constant over the whole range of wind speeds likely to be encountered. However, for objects of circular cross section, it varies considerably.

For a circular section, the force coefficient depends on the way in which the wind flows around it and is dependent upon the velocity and kinematic viscosity of the wind and diameter of the section. The force coefficient is usually quoted against a non-dimensional parameter, called the Reynolds number, which takes into account of the velocity and viscosity of the flowing medium (in this case the wind), and the member diameter.

Reynolds number, $R_e = \frac{D\bar{V}_d}{\nu}$

where

D = diameter of the member,
$\bar{V}_d$ = design hourly mean wind speed, and
ν = kinematic viscosity of the air which is 1.46×10^{-5} m^2/s at 15°C and standard atmospheric pressure.

Since in most natural environments likely to be found in India, the kinematic viscosity of the air is fairly constant, it is convenient to use $D\bar{V}_d$ as the parameter instead of Reynolds number and this has been done in this Section.

The dependence of a circular section's force coefficient on Reynolds number is due to the change in the wake developed behind the body.

At a low Reynolds number, the wake is as shown in Fig. 19 and the force coefficient is typically 1.2. As Reynolds number is increased, the wake gradually changes to that shown in Fig. 20, that is, the wake width d_w decreases and the separation point, denoted as s_p, moves from front to the back of the body.

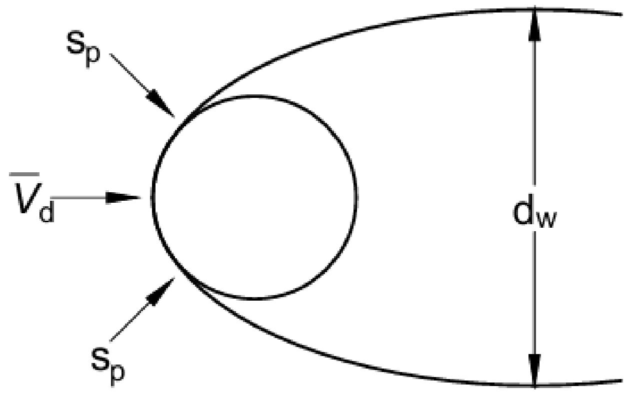


FIG. 19 WAKE IN SUB-CRITICAL FLOW

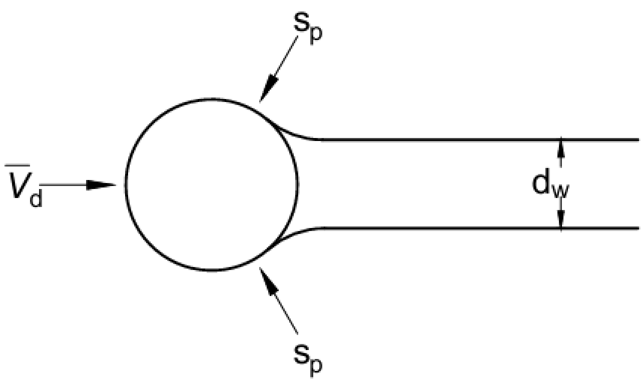


FIG. 20 WAKE IN SUPER-CRITICAL FLOW

As a result, the force coefficient shows a rapid drop at a critical value of Reynolds number followed by a gradual rise as Reynolds number is increased still further.

The variation of C_f with parameter $D\overline{V}_d$ is shown in Fig. 5 for infinitely long circular cylinders having various values of relative surface roughness (ε/D) when subjected to wind having an intensity and scale of turbulence typical of built-up urban areas. The curve for a smooth cylinder (ε/D) = 1 x 10^{-5} in a steady air stream, as found in a low-turbulence wind tunnel, is also shown for comparison.

It can be seen that the main effect of free-stream turbulence is to decrease the critical value of the parameter $D\overline{V}_d$. For subcritical flows, turbulence can produce a considerable reduction in C_f below the steady air-stream values. For supercritical flows, this effect becomes significantly smaller.

If the surface of the cylinder is deliberately roughened, such as by incorporating flutes, riveted construction, etc, then the data given in Fig. 5 for appropriate value of $\varepsilon/D > 0$ shall be used.

NOTE – In case of uncertainty regarding the value of ε to be used for small roughness, ε/D shall be taken as 0.001.

ANNEX H

SYMBOLS

The provision of this clause will be added suitably after the finalization/publication of Part 1 and 2 of Design Earthquake Hazard and Criteria for Earthquake Resistant Design of Structures - Part 1 : General Provisions and Part 5 Buildings [(Second Revision of IS 1893 (Part 1)].

ANNEX J

MSK 1964 INTENSITY SCALE

The provision of this clause will be added suitably after the finalization/publication of Part 1 and 2 of Design Earthquake Hazard and Criteria for Earthquake Resistant Design of Structures - Part 1 : General Provisions and Part 5 Buildings [(Second Revision of IS 1893 (Part 1)].

ANNEX K

EARTHQUAKE HAZARD ASSESSMENT FOR MACRO-ZONING AND SITE-SPECIFIC STUDIES

The provision of this clause will be added suitably after the finalization/publication of Part 1 and 2 of Design Earthquake Hazard and Criteria for Earthquake Resistant Design of Structures - Part 1 : General Provisions and Part 2 Buildings [(Second Revision of IS 1893 (Part 1)].

ANNEX L

LIST OF SOME IMPORTANT TOWNS AND THEIR SEISMIC ZONE FACTOR (Z)

The earthquake zone factors mentioned below are those corresponding to the return period of 2 475 years.

Sl No.	Town	Zone	Z
(1)	(2)	(3)	(4)
i)	Agartala	VI	0.75
ii)	Agra	III	0.25
iii)	Ahilyanagar	II	0.15
iv)	Ahmedabad	IV	0.35
v)	Aizawl	VI	0.75
vi)	Ajmer	III	0.25
vii)	Akola	II	0.15
viii)	Aligarh	IV	0.35
ix)	Almora	VI	0.75
x)	Alwar	IV	0.35
xi)	Ambala	V	0.50
xii)	Ambassa	VI	0.75
xiii)	Ambattur	III	0.25
xiv)	Ambernath	III	0.25
xv)	Amravati	III	0.25
xvi)	Amritsar	V	0.50
xvii)	Anantapur	II	0.15
xviii)	Anantnag	VI	0.75
xix)	Arrah	III	0.25
xx)	Asansol	IV	0.35
xxi)	Avadi	III	0.25
xxii)	Bahraich	V	0.50
xxiii)	Bally	IV	0.35
xxiv)	Bengaluru	II	0.15
xxv)	Baranagar	IV	0.35
xxvi)	Barasat	IV	0.35
xxvii)	Barauni	IV	0.35
xxviii)	Barddhaman	IV	0.35
xxix)	Bareilly	V	0.50
xxx)	Begusarai	IV	0.35
xxxi)	Belagavi	II	0.15
xxxii)	Ballari	II	0.15
xxxiii)	Brahmapur	II	0.15
xxxiv)	Bhagalpur	V	0.50
xxxv)	Bharatpur	III	0.25
xxxvi)	Bhatinda	III	0.25
xxxvii)	Bhatpara	IV	0.35
xxxviii)	Bhavnagar	IV	0.35
xxxix)	Bhilai	II	0.15
xl)	Bhilwara	II	0.15
xli)	Bhiwadi	IV	0.35
xlii)	Bhiwandi	III	0.25
xliii)	Bhopal	II	0.15
xliv)	Bhubaneshwar	III	0.25
xlv)	Bhuj	VI	0.75
xlvi)	Bidar	II	0.15
xlvii)	Bidhan Nagar	IV	0.35
xlviii)	Biharsharif	III	0.25
xlix)	Bikaner	II	0.15
l)	Bilaspur	II	0.15
li)	Bokaro	III	0.25
lii)	Bulandshahr	IV	0.35
liii)	Burdwan	IV	0.35
liv)	Burhanpur	III	0.25
lv)	Kozhikode	III	0.25
lvi)	Champhai	VI	0.75
lvii)	Chandigarh	VI	0.75
lviii)	Chandrapur	II	0.15
lix)	Chhapra	IV	0.35
lx)	Chhatrapati Sambhajinagar	II	0.15

Sl No.	Town	Zone	Z
(1)	(2)	(3)	(4)
lxi)	Chennai	III	0.25
lxii)	Chitradurga	II	0.15
lxiii)	Coimbatore	III	0.25
lxiv)	Cuddalore	III	0.25
lxv)	Cuttack	III	0.25
lxvi)	Daman	III	0.25
lxvii)	Darbhanga	V	0.50
lxviii)	Darjeeling	VI	0.75
lxix)	Davanagere	II	0.15
lxx)	Dehradun	VI	0.75
lxxi)	Delhi	IV	0.35
lxxii)	Deoghar	III	0.25
lxxiii)	Dewas	II	0.15
lxxiv)	Dhanbad	III	0.25
lxxv)	Dharashiv	III	0.25
lxxvi)	Dharwad	II	0.15
lxxvii)	Dhule	II	0.15
lxxviii)	Dibrugarh	VI	0.75
lxxix)	Dimapur	VI	0.75
lxxx)	Dindigul	III	0.25
lxxxi)	Dispur	VI	0.75
lxxxii)	Diu	III	0.25
lxxxiii)	Durg	II	0.15
lxxxiv)	Durgapur	IV	0.35
lxxxv)	Eluru	III	0.25
lxxxvi)	English Bazar	V	0.50
lxxxvii)	Etawah	II	0.15
lxxxviii)	Faridabad	IV	0.35
lxxxix)	Farrukhabad	III	0.25
xc)	Firozabad	III	0.25
xci)	Gandhidham	VI	0.75
xcii)	Gandhinagar	IV	0.35
xciii)	Ganganagar	II	0.15
xciv)	Gangtok	VI	0.75
xcv)	Gaya	III	0.25
xcvi)	Ghaziabad	IV	0.35

Sl No.	Town	Zone	Z
(1)	(2)	(3)	(4)
xcvii)	Gorakhpur	V	0.50
xcviii)	Guntur	III	0.25
xcix)	Gurugram	IV	0.35
c)	Guwahati	VI	0.75
ci)	Gwalior	II	0.15
cii)	Haldia	IV	0.35
ciii)	Haldwani	VI	0.75
civ)	Howrah	IV	0.35
cv)	Hapur	IV	0.35
cvi)	Haridwar	VI	0.75
cvii)	Hisar	IV	0.35
cviii)	Hosapete	II	0.15
cix)	Hubbali	II	0.15
cx)	Hyderabad	II	0.15
cxi)	Ichalkaranji	III	0.25
cxii)	Imphal	VI	0.75
cxiii)	Indore	III	0.25
cxiv)	Itanagar	VI	0.75
cxv)	Jabalpur	III	0.25
cxvi)	Jaipur	IV	0.35
cxvii)	Jalandhar	V	0.50
cxviii)	Jalgaon	III	0.25
cxix)	Jalna	II	0.15
cxx)	Jammu	VI	0.75
cxxi)	Jamnagar	IV	0.35
cxxii)	Jamshedpur	III	0.25
cxxiii)	Jhansi	II	0.15
cxxiv)	Jodhpur	III	0.25
cxxv)	Jorhat	VI	0.75
cxxvi)	Junagadh	III	0.25
cxxvii)	Kadapa	II	0.15
cxxviii)	Kakching	VI	0.75
cxxix)	Kakinada	III	0.25
cxxx)	Kakrapara	IV	0.35
cxxxi)	Kalpakkam	III	0.25
cxxxii)	Kalyan	III	0.25

Sl No.	Town	Zone	Z
(1)	(2)	(3)	(4)
cxxxiii)	Kamarhati	IV	0.35
cxxxiv)	Kanchipuram	III	0.25
cxxxv)	Kanpur	III	0.25
cxxxvi)	Karawal	IV	0.35
cxxxvii)	Karimnagar	II	0.15
cxxxviii)	Karnal	V	0.50
cxxxix)	Karwar	II	0.15
cxl)	Katihar	V	0.50
cxli)	Kavaratti	II	0.15
cxlii)	Khandwa	III	0.25
cxliii)	Kharagpur	IV	0.35
cxliv)	Kochi	III	0.25
cxlv)	Kohima	VI	0.75
cxlvi)	Kolhapur	IV	0.35
cxlvii)	Kolkata	IV	0.35
cxlviii)	Kollam	II	0.15
cxlix)	Korba	II	0.15
cl)	Kota	II	0.15
cli)	Kulti	IV	0.35
clii)	Kurnool	II	0.15
cliii)	Latur	III	0.25
cliv)	Leh	VI	0.75
clv)	Loni	IV	0.35
clvi)	Lucknow	III	0.25
clvii)	Ludhiana	V	0.50
clviii)	Madurai	III	0.25
clix)	Maheshtala	IV	0.35
clx)	Malegaon	II	0.15
clxi)	Mandi	VI	0.75
clxii)	Mangaluru	II	0.15
clxiii)	Mathura	III	0.25
clxiv)	Meerut	V	0.50
clxv)	Mirzapur	III	0.25
clxvi)	Moradabad	V	0.50
clxvii)	Morena	II	0.15
clxviii)	Mumbai	III	0.25

Sl No.	Town	Zone	Z
(1)	(2)	(3)	(4)
clxix)	Mungher	IV	0.35
clxx)	Murwara	III	0.25
clxxi)	Muzaffarnagar	V	0.50
clxxii)	Muzaffarpur	V	0.50
clxxiii)	Mysuru	II	0.15
clxxiv)	Nadiad	IV	0.35
clxxv)	Nagarjuna Sagar	II	0.15
clxxvi)	Nagercoil	II	0.15
clxxvii)	Nagpur	II	0.15
clxxviii)	Naharlagun	VI	0.75
clxxix)	Naihati	IV	0.35
clxxx)	Nainital	VI	0.75
clxxxi)	Namchi	VI	0.75
clxxxii)	Nanded	II	0.15
clxxxiii)	Nandyal	II	0.15
clxxxiv)	Nangloi	IV	0.35
clxxxv)	Nashik	III	0.25
clxxxvi)	Navi Mumbai	III	0.25
clxxxvii)	Nellore	III	0.25
clxxxviii)	Nizamabad	III	0.25
clxxxix)	Noida	IV	0.35
cxc)	Ongole	III	0.25
cxci)	Kalaburagi	II	0.15
cxcii)	Ozhukarai	III	0.25
cxciii)	Pali	III	0.25
cxciv)	Pallavaram	III	0.25
cxcv)	Panaji	II	0.15
cxcvi)	Panchkula	VI	0.75
cxcvii)	Panihati	IV	0.35
cxcviii)	Panipat	V	0.50
cxcix)	Parbhani	II	0.15
cc)	Patiala	V	0.50
cci)	Patna	IV	0.35
ccii)	Pilibhit	V	0.50
cciii)	Pimpri-Chinchwad	III	0.25
cciv)	Puducherry	III	0.25

Sl No. (1)	Town (2)	Zone (3)	Z (4)
ccv)	Port Blair	VI	0.75
ccvi)	Prayagraj	II	0.15
ccvii)	Pune	III	0.25
ccviii)	Puri Town	III	0.25
ccix)	Purnia	V	0.50
ccx)	Raichur	II	0.15
ccxi)	Raipur	II	0.15
ccxii)	Rajahmahendravaram	III	0.25
ccxiii)	Rajarhat Gopalpur	IV	0.35
ccxiv)	Rajkot	IV	0.35
ccxv)	Rajpur Sonarpur	IV	0.35
ccxvi)	Ramagundam	II	0.15
ccxvii)	Rampur	V	0.50
ccxviii)	Ranchi	III	0.25
ccxix)	Ratlam	II	0.15
ccxx)	Rourkela	III	0.25
ccxxi)	Rewa	II	0.15
ccxxii)	Rohtak	IV	0.35
ccxxiii)	Roorkee	VI	0.75
ccxxiv)	Sagar	II	0.15
ccxxv)	Saharanpur	V	0.50
ccxxvi)	Salem	III	0.25
ccxxvii)	Sambhal	IV	0.35
ccxxviii)	Sangli Miraj	III	0.25
ccxxix)	Satna	II	0.15
ccxxx)	Secunderabad	II	0.15
ccxxxi)	Shahjahanpur	IV	0.35
ccxxxii)	Shillong	VI	0.75
ccxxxiii)	Shimla	VI	0.75
ccxxxiv)	Shivamogga	II	0.15
ccxxxv)	Sikar	III	0.25
ccxxxvi)	Siliguri	VI	0.75
ccxxxvii)	Singrauli	IV	0.35
ccxxxviii)	Sironj	II	0.15
ccxxxix)	Solapur	III	0.25
ccxl)	Sonipat	IV	0.35
ccxli)	Srinagar	VI	0.75
ccxlii)	Surat	IV	0.35
ccxliii)	Tarapur	III	0.25
ccxliv)	Tezpur	VI	0.75
ccxlv)	Thane	III	0.25
ccxlvi)	Thanjavur	III	0.25
ccxlvii)	Thiruvananthapuram	II	0.15
ccxlviii)	Thiruvannamalai	III	0.25
ccxlix)	Thoothukkudi	II	0.15
ccl)	Thrissur	III	0.25
ccli)	Tiruchirappalli	III	0.25
cclii)	Tirunelveli	II	0.15
ccliii)	Tirupati	III	0.25
ccliv)	Tiruppur	III	0.25
cclv)	Tiruvottiyur	III	0.25
cclvi)	Tumakuru	II	0.15
cclvii)	Udaipur	III	0.25
cclviii)	Ujjain	II	0.15
cclix)	Ulhasnagar	III	0.25
cclx)	Uluberia	IV	0.35
cclxi)	Vadodara	IV	0.35
cclxii)	Varanasi	III	0.25
cclxiii)	Vasai-Virar	III	0.25
cclxiv)	Vellore	III	0.25
cclxv)	Vijayapura	II	0.15
cclxvi)	Vijayawada	III	0.25
cclxvii)	Vishakhapatnam	II	0.15
cclxviii)	Vizianagaram	II	0.15
cclxix)	Warangal	II	0.15
cclxx)	Yamunanagar	V	0.50

ANNEX M
(*Foreword* and *Clause* 4.2)

CHARACTERISTIC GROUND SNOW LOAD

M-1 This Annex presents the characteristic ground snow load computation procedure for the union territory of Jammu and Kashmir, union territory of Ladakh, Himachal Pradesh, Uttarakhand and Sikkim which is the result of scientific work carried out by Defence Geoinformatics Research Establishment (DGRE), Chandigarh under Defence Research and Development Organization (DRDO).

M-2 The characteristic value of the ground snow load at any site is defined with a 3.4 percent annual exceedance probability or, equivalently, with a mean recurrence interval, MRI=30 years. The site altitude is the altitude at which the structure is or will be located, measured from mean sea level. The procedure shall be used for the computation of characteristic ground snow load on sites with an altitude higher than 1 000 m and lower than 4 000 m from mean sea level.

M-3 The characteristic ground snow load, s_0 (kPa) to be used for any site in the union territory of Jammu and Kashmir, union territory of Ladakh, Himachal Pradesh, Uttarakhand should be obtained from the zone map shown in Fig. 1 and equation given below:

$$s_0 = 1.624 * [Z - 0.5] * \left[1 + \left(\frac{A}{2677}\right)^2\right] \text{ kPa}$$

where

s_0	= characteristic ground snow load (kPa);
Z	= zone number (for example 1, 2, 3, 4 or 5) obtained from the zone map in Fig. 21; and
A	= site altitude (m).

M-4 The characteristic ground snow load s_0 (kPa) to be used for any site in Sikkim should be directly obtained from the characteristic ground snow load map shown in Fig. 22.

M-5 The ground snow loads for any mean recurrence interval of *n* years, different to that for the characteristic snow load, s_0, (which by definition is based on annual probability of exceedance of 0.034 that is MRI = 30 years) shall be calculated by the equation given below:

$$s_n = s_0 \left\{ \frac{1 - C_{\mathrm{v}}\frac{\sqrt{6}}{\pi}\left[ln\left(-ln(1-P_{\mathrm{n}})\right)+0.57722\right]}{(1+2.1887C_{\mathrm{v}})} \right\}$$

Where,

s_0 is the characteristic snow load on the ground with MRI = 30 years

s_n is the characteristic snow load on the ground with MRI = *n* years

P_n is the annual probability of exceedance (equivalent to $1/n$, where n is the corresponding recurrence interval in years)

C_v is the coefficient of variation of annual maximum snow load.

M-5.1 The coefficient of variation of maximum snow load at different snow-meteorological observatories of DGRE located in union territory of J&K, union territory of Ladakh, HP and UK ranges between 0.35 to 0.65. For application of above equation, a constant value of $C_v = 0.5$ is recommended for all the zones to calculate snow loads on the ground for other probability of exceedance (for example, MRI of 50 years for structures where a higher risk of exceedance is deemed acceptable). The above equation is shown graphically in Fig. 23.

NOTES

1 The characteristic ground snow load *versus* altitude relation as given above in equation and Fig. 21 is determined based on the snow depth observations at more than 70 field observatories of DGRE located in different parts of union territory of Jammu and Kashmir, union territory of Ladakh and states of Himachal Pradesh and Uttarakhand as well as High Asia Refined analysis (HAR10) dataset (HAR, Maussion et al., 2014).

2 Since adequate snow depth measurements are not available for Sikkim region, hence the characteristic ground snow load map for Sikkim given in Fig. 22 is generated based solely on high Asia refined analysis (HAR10) dataset (HAR, Maussion et al., 2014).

3 Annex A does not apply for sites at altitudes above 4 000 m. For altitudes greater than 4 000 m specialist advice should be sought from DGRE on the characteristic ground snow loads likely to occur at the site.

4 Unusual local effects have not been accounted for in the analysis undertaken to produce the characteristic ground snow load map given in Annex A. These include local shelter from the wind, which can result in increased local snow loads and local configurations in mountainous areas, which may funnel the snow and give increased local loading. Annex A is also not applicable for sites with permanent snow/ice cover throughout the year. If the designer suspects that there are unusual local conditions that need to be taken into account, DGRE may be consulted.

5 In special cases where more refined data is needed, the characteristic value of snow load on the ground (s_0) may be refined using an appropriate statistical analysis of long term records taken in a well sheltered area near the site.

6 Ground characteristics snow load at any place depends on the critical combination of maximum depth of undisturbed aggregate cumulative snowfall and its density. In due course the characteristics snow load for states of Arunachal Pradesh will be included based on studies. Till such time the users of this standard are advised to consult DGRE for the specific information relating to the state of Arunachal Pradesh.

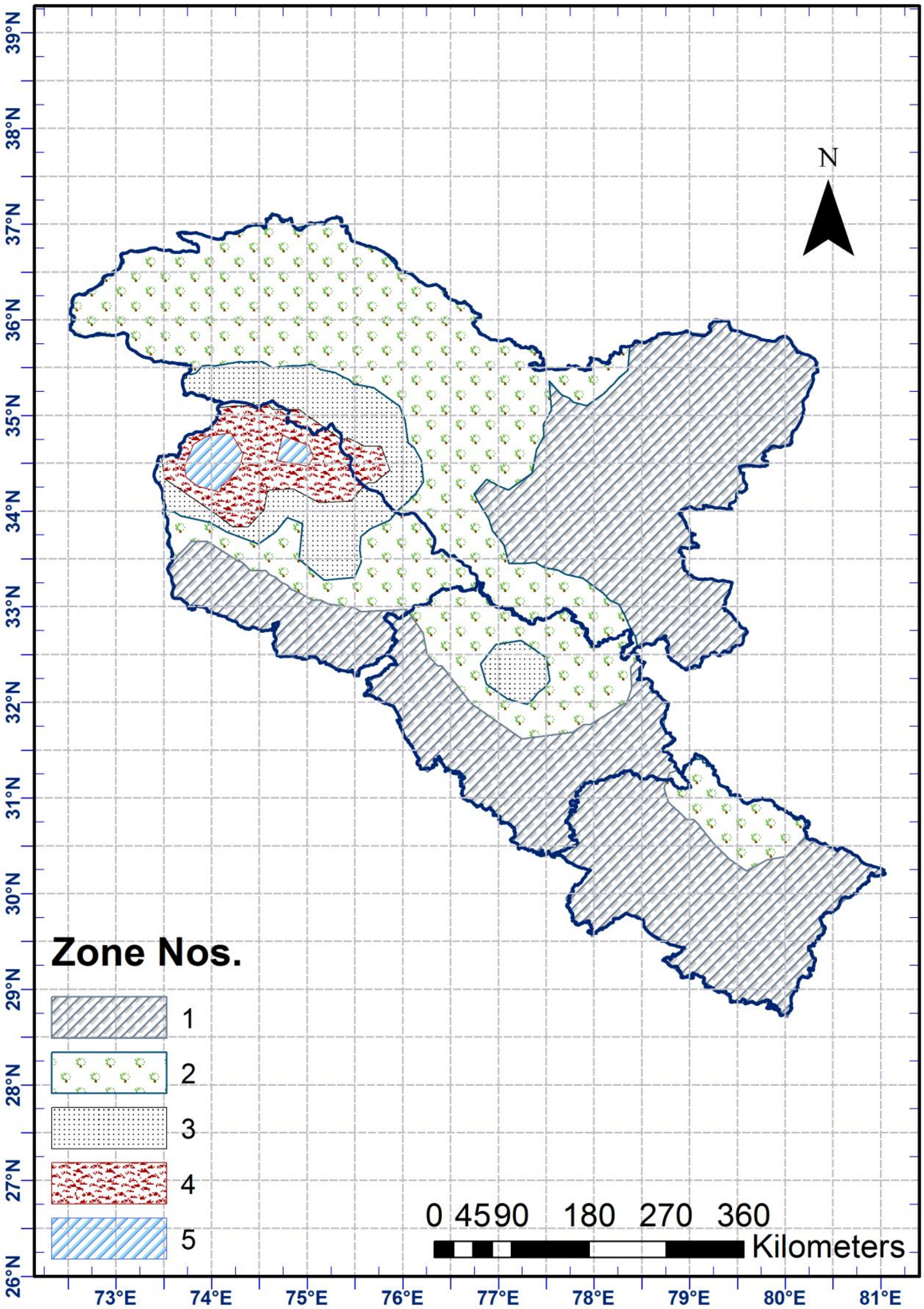


FIG. 21 ZONE MAP FOR J&K, LADAKH, HP AND UK AT 10 KM RESOLUTION

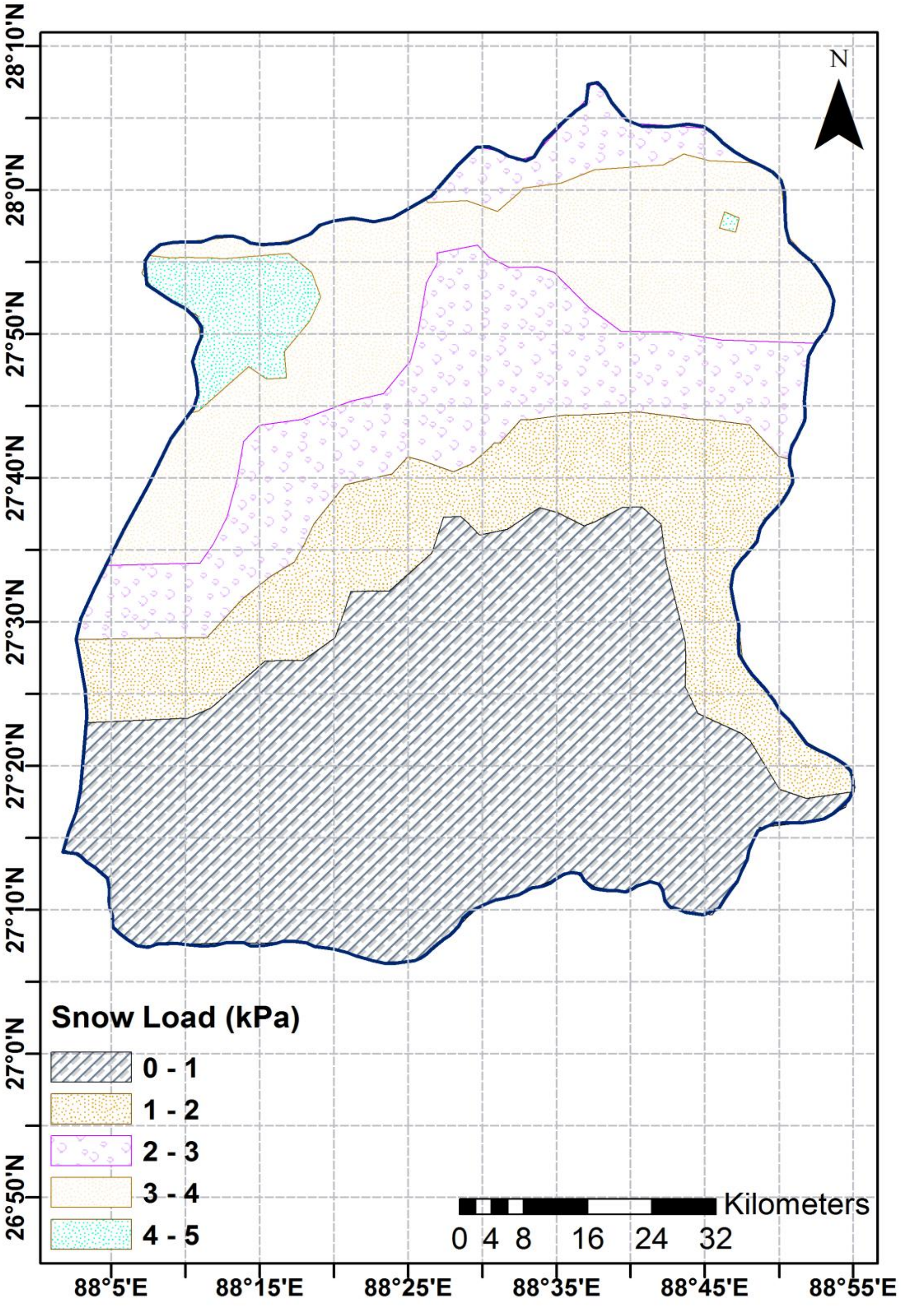


FIG. 22 CHARACTERISTIC GROUND SNOW LOAD MAP FOR SIKKIM AT 10 KM RESOLUTION

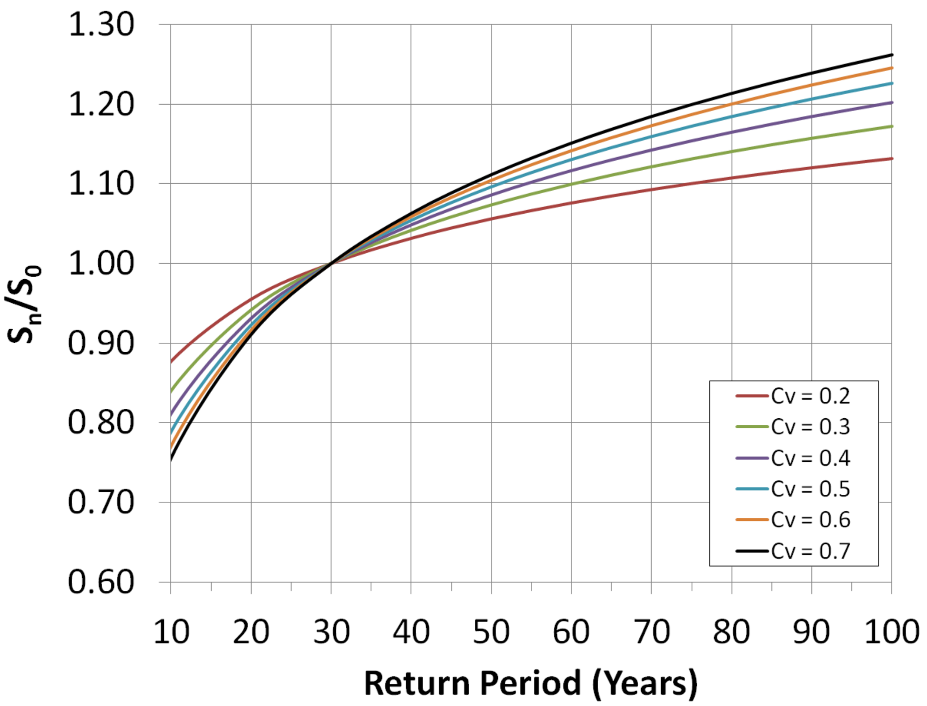
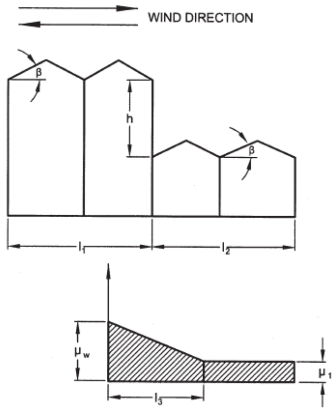


FIG. 23 ADJUSTMENT OF THE GROUND SNOW LOAD ACCORDING TO THE RETURN PERIOD

ANNEX N
(*Clause* 6.4.3)

SHAPE COEFFICIENTS FOR MULTILEVEL ROOFS

N-1 A more comprehensive formula for the shape coefficient for multilevel roofs than that given in **6.4.2** is as follows:



$$\mu_w = 1 + \frac{1}{h}(m_1 l_1 + m_2 l_2)(l_2 - 2h)$$

$$\mu_1 = 0.8$$

$$l_3 = 2h$$

(h and l in m*)*

Restriction:

$$\mu_w \le \frac{kh}{S_o}$$

where

S_o is in kilopascals (kilonewtons per square metre)

k is in newtons per cubic metre

$l_3 \le 15\,m$

Values of $m_1(m_2)$ for the higher (lower) root depend on its profile and are taken as equal to:

0.5 for plane roofs with slopes $\beta \le 20°$ and vaulted roofs with $\frac{f}{l} \le \frac{1}{18}$

0.3 for plane roofs with slopes $\beta \le 20°$ and vaulted roofs with $\frac{f}{l} \le \frac{1}{18}$

The coefficient m_1 and m_2 may be adjusted to take into account conditions for transfer of snow on the roof surface (that is wind, temperature, etc).

NOTE — The other condition of loading shall also be tried.

ANNEX P
(*Clause* 7.7)

VIBRATIONS IN BUILDINGS

P-1 GENERAL

In order to design the buildings safe against vibrations, it is necessary to identify the source and nature of vibration. Vibrations may be included in the buildings due to various actions, such as,

a) human induced vibrations, for example, the walking or running or a single person or a number of persons or dancing or motions in stadia or concert halls;
b) machine induced vibrations;
c) wind induced vibrations;
d) blast induced vibrations;
e) traffic load, for example, due to rail, fork-lift, trucks, cars, or heavy vehicles;
f) airborne vibrations;
g) crane operations; and
h) other dynamic actions such as wave loads or earthquake actions.

The dynamic response of buildings for the above mentioned causes of vibration of buildings may have to be evaluated by adopting standard mathematical models and procedures.

The severity or otherwise of these actions have to be assessed in terms of the limits set for dynamic response (frequencies and amplitude of motion) of the buildings related to (a) human comfort, (b) serviceability requirements such as deflections and drifts and separation distances to avoid damage due to pounding, and (c) limits set on the frequencies and amplitude of motion for machines and other installations.

In order to verify that the set limits are not exceeded, the actions may be modelled in terms of force-time histories for which the structural responses may be determined as time histories of displacements or accelerations by using appropriate analytical/numerical methods.

P-2 SERVICEABILITY LIMIT STATE VERIFICATION OF STRUCTURE SUSCEPTIBLE TO VIBRATIONS

P-2.1 While giving guidance for serviceability limit state verification of structure susceptible to vibrations, here it is proposed to deal with the treatment of the action side, the determination of the structural response and the limits to be considered for the structural response to ensure that vibrations are not harmful or do not lead to discomfort.

P-2.2 Source of Vibrations

Vibrations may be included by the following sources:

a) By the movement of persons as in pedestrian bridges, floors where people walk, floors meant for sport or dancing activities, and floors with fixed seating and spectator galleries;
b) By working of machines as in machine foundations and supports, vibrations transmitted through the ground, and pile driving operations;
c) By wind blowing on buildings, towers, chimneys and masts, guyed masts, pylons, bridges, cantilevered roofs, airborne vibrations;
d) Induced by traffic on rail or road bridges and car park structures and exhibition halls; and
e) By earthquakes.

P-2.3 Modelling of Actions and Structures

For serviceability limit states, the modelling of these actions and of the structure depends on how the serviceability limits are formulated. The serviceability limit states may refer to,

(a) human comfort;
(b) limits for the proper functioning of machines and other installations; and
(c) maximum deformation limits to avoid damage or pounding.

In order to verify that these limits are not exceeded, the actions may be modeled in terms of force-time histories, for which the structural responses may be determined as time histories of displacements or accelerations by using appropriate analytical/numerical methods. Where the structural response may significantly influence the force-time histories to be applied, these interactions have to be considered either in modeling a combined load-structure vibration system or by appropriate modifications of the force time histories. In addition to the levels of vibration for which presently limits have been specified, the possible deformations of structural members and systems using different clauses in the relevant codes have to be evaluated by adopting standard mathematical models and procedures.

P-2.4 Force-Time Histories

The force-time histories used in the dynamic analysis should adequately represent the relevant loading situations for which the serviceability limits are to be verified. The force-time histories may model,

a) human induced vibrations, for example the walking or running of a single person or a number of persons or dancing or motions in stadia or concert halls;
b) machine induced vibrations, for example by force vectors due to mass eccentricities and frequencies, that may be variable with time;
c) wind induced vibrations;
d) blast induced vibrations;

e) traffic load, for example rail, fork-lift, trucks, cars, or heavy vehicles;
f) airborne vibrations;
g) crane operations; and
h) other dynamic actions such as wave loads or earthquake actions.

ANNEX Q
(*Clause* 7.8)

BLAST LOAD

Q-1 BLAST LOAD

Q-1.1 This annex prescribes the minimum criteria for analysis and design of new structures for blast effects of above-ground explosions of high-explosives under three different detonation scenarios: surface burst, free-air burst, and air burst. The primary objective of this standard is to minimize human casualties and safeguard assets due to above ground explosions. The performance expectation of a structure designed as per the provisions of this standard would depend on the type of protection category selected for that structure and the blast loading determined through threat assessment.

Q-1.2 This Annex shall be used in conjunction with other applicable standards for analysis and design of structures. This annex does not supersede or replace other design standards. This annex shall be applicable if design against malevolent or accidental blast loading is determined to be a credible threat to a structure.

Q-1.3 The owner of a structure is the competent authority who can determine the necessity of blast-resistant design. In the scenario of accidental and malevolent blast event which may pose public health and safety issue, competent authorities other than the owner of building that govern or regulate the construction, operation, and maintenance of the structural facility, can specify the requirement of blast-resistant design.

Q-1.4 The provisions of this Annex do not address the following:

a) Estimation of structural loads due to nuclear detonations, pressure vessel explosions, dust, and vapour cloud explosions;
b) Effects of cased explosives and shaped charges;
c) Threats emanating from fragmentation, induced fire or thermal effects, biological or chemical radiations, electromagnetic pulse (EMP) and deflagration events; and
d) Detonations in contact with structural elements.

Q-2 TERMINOLOGY

For the purpose of this standard, the following definitions shall apply.

Q-2.1 Arrival Time — Time taken by the shock after the explosion to reach the location of interest.

Q-2.2 Charge Mass — Mass of the explosive material responsible for generating blast effects.

Q-2.3 Clearance Time — Time in which the reflected pressure decays down to the sum of the side on overpressure and the dynamic pressure.

Q-2.4 Close-in Detonation — Detonation at a scaled distance (Z) below which the resulting blast pressure distribution is non-uniform over the reflecting surface of the element being considered. It corresponds to a value of Z less than 1.2 m/kg1/3.

Q-2.5 Decay Parameter — The coefficient of the negative power of exponent e governing the fall of pressure with time in the pressure-time curves.

Q-2.6 Detonation — Fast and sudden release of huge amount of energy through a stable chemical reaction in which reaction front propagates into unreacted substance at supersonic speed resulting in formation of shock waves.

Q-2.7 Drag Force — Force on a structure or structural element due to the dynamic pressure of the blast wave. On any structural element, the drag force equals dynamic pressure multiplied by the drag coefficient and the area of the element.

Q-2.8 Ductility Ratio — Ratio of the maximum deflection to the deflection corresponding to the elastic limit.

Q-2.9 Dynamic Increase Factor (DIF) — A factor applied to material strength to account for high strain rate loading effects.

Q-2.10 Dynamic Pressure — The pressure due to air mass movement behind the shock front.

Q-2.11 Exponential Blast Pressure History — The variation of blast pressure with time at a point represented through an exponential decaying function, which considers positive and negative phase of blast overpressures.

Q-2.12 Far-field Detonation — Detonation at a scaled distance (*Z*) above which the resulting blast pressure distribution is uniform over the reflecting surface of element being considered. It corresponds to a value of *Z* greater than 1.2 $m/kg^{1/3}$.

Q-2.13 High-Explosives — Chemical explosive that undergoes detonation to produce shock waves with high pressure and short duration.

Q-2.14 Impulse — The integral of the pressure-time curve per unit projected area due to explosion (*see* specific impulse).

Q-2.15 Mach Number — The ratio of the speed of the shock front propagation to the speed of sound in standard atmosphere at sea level.

Q-2.16 Negative Phase — The duration of reflected overpressure when it is below the ambient atmospheric pressure.

Q-2.17 Overpressure — The rise in pressure above atmospheric pressure due to the shock wave from an air blast.

Q-2.18 Positive Phase —The duration of overpressure when it is above the ambient atmospheric pressure.

Q-2.19 Protection Category — Category assigned to different structures that correlates its importance to the performance goals through qualitative estimate of damage.

Q-2.20 Reflected Overpressure —The overpressure resulting due to reflection of a shock wave front striking any surface. If the shock front is parallel to the surface, the reflection is normal.

Q-2.21 Regular Shaped Structure — A structure that does not have any spatial geometric irregularity.

Q-2.22 Scaled Distance — The distance between the centre of explosion to the point of interest divided by the cube root of the charge mass expressed in equivalent TNT. The unit for scaled distance is $m/kg^{1/3}$.

Q-2.23 Shape Function — The displaced shape of the component as a function of its length normalized to the maximum deflection.

Q-2.24 Shock Wave Front — The discontinuity between the blast wave and the surrounding atmosphere. It propagates away from the point of explosion in all directions at a speed greater than the speed of sound in the undisturbed atmosphere.

Q-2.25 Side-on (Incident) Overpressure — Overpressure if it is not reflected by any surface.

Q-2.26 Simplified Overpressure History — A triangular representation of the overpressure history obtained by equating the impulse and the peak overpressure of the exponential representation of the overpressure history.

Q-2.27 Specific Impulse — The area under the pressure-time curve due to explosion. This is often referred to as impulse (*see* **Q-2.13**).

Q-2.28 Standoff Distance — The shortest distance between the centre of detonation and the point of interest at which the effects of explosion need to be considered.

Q-2.29 Support Rotation — The angle formed by a flexural member in its deflected shape with respect to the chord adjoining the two support ends (*see* Fig. 45).

Q-2.30 Transit Time — Time required for the shock front to travel across the structure or its element under consideration.

Q-3 SYMBOLS

For the purpose of this standard, the following notations shall apply.

b	Decay parameter/coefficient
C_d	Drag coefficient

C_r	Velocity of sound in air
$C_{r\alpha}$	Reflection coefficient
E	Modulus of elasticity of material
H	Height of the structure
I	Moment of inertia of member
i_e	Equivalent specific impulse of a uniformly distributed load
i_{ro}	Peak reflected specific impulse
j	Number of concentrated load points
K_L	Load factor
K_{LM}	Load mass factor
K_M	Mass factor
k_E	Effective stiffness of equivalent single spring-mass system
L	Length of structure in the direction of motion of blast wave (*see* Fig. 39)
M	Charge mass/weight
M_p	Plastic moment of section
M_{pm}	M_p at mid-span
M_{ps}	M_p at support
M_{pfa}	Total positive ultimate moment capacity along mid-span section parallel to short edge
M_{pfb}	Total positive ultimate moment capacity along mid-span section parallel to long edge
m	Distributed mass intensity per unit length
n	Number of concentrated masses
p_a	Ambient atmospheric pressure
P_r	Concentrated dynamic force at point *r*
p_{ro}	Peak reflected overpressure
p_s	Side-on overpressure
p_{so}	Peak side-on overpressure
P_t	Total dynamic load (at any instant of time)
$p(x)$	Intensity of distributed dynamic load per unit length
p_{o1}	Peak reflected overpressure on a surface including the effects of clearing

p_{o2}	Intercept of the reflected overpressure history on the y-axis corresponding to the clearing phase
q	Dynamic pressure
q_o	Peak dynamic pressure
R	Standoff distance
R_m	Peak resistance of a structural member against a given load distribution
S	W or *H* whichever is less
T	Elastic time-period of structural member
t_c	Clearance time
t_d	Duration of the equivalent triangular pulse
t_m	Time to the peak displacement of an SDOF system
t_o	Time for positive phase of side-on overpressure
t_t	Transit time
t_{d1}	Time at which clearing starts on a finite reflecting surface (*see* Fig. 47B)
t_{d2}	Total time duration of blast loading on a finite reflecting surface (*see* Fig. 47B)
U	Shock front velocity
u	Peak particle velocity
V_A	Total dynamic reaction along a short edge
V_B	Total dynamic reaction along a long edge
y_E	Displacement corresponding to the elastic limit of an equivalent bilinear representation of idealised resistance-deflection curve
y_{el}	Displacement corresponding to the first onset of yielding (elastic limit) of the idealized resistance-deflection curve of a structural member
y_m	Maximum deflection/deformation permitted in the design of the structure
y_o	Peak static displacement of an SDOF system without inertial effects
W	Span or width of structure across the direction of shock wave propagation
Z	Scaled distance
μ	Ductility ratio = $\frac{y_m}{y_E}$
φ_r	Deflection at point *r* of an assumed deflected shape for concentrated loads

$\varphi_{(x)}$ Deflection at point *x* of an assumed deflected shape for distributed loads

$\varphi_{(x,y)}$ Deflected shape function of blast-loaded component

Q-4 GENERAL PRINCIPLES

Q-4.1 Blast Resistant Design Attributes

The design process of a building against blast should start with planning of building layout and arrangements. The distance between the explosive and the building shall be maximized through a combination of site-access controls, landforms, anti-ram structures, street and site landscaping, and perimeter walls. The building should preferably have short-face oriented towards blast source. Also, it shall be located uphill and placed upstream of wind direction and away from congestion. The structures and their contents that need protection against an explosion event should be decided and all possible scenarios need to be considered which can cause damage or occupant injuries. The overall design of a structure shall aim to incorporate following attributes in the overall planning and design of a facility against explosion threats:

a) *Security* — Discourage potential terrorist attacks on a structure through visible security personnel deployment and defences to reduce chance of such attacks;
b) *Deception* — Disguise the critical area or content of structure so that the focus of the attack is misdirected. Such attacks, even if successfully completed, may not deliver intended impact and realize its goals;
c) *Shielding* — Shield the target structure through physical barriers, including but not limited to blast walls, bollards, anti-ram elements, vehicle and pedestrian entry controls, and landforms. The objective of shielding is to increase the standoff distance between the charge and the target structure and components; and
d) *Design* — The attack must be blunted through appropriate design and detailing of the structure to absorb the energy of the attack and safeguard valuable assets.

Q-4.2 Blast Source

Q-4.2.1 *Charge Shape*

The provisions of this standard shall be applicable for:

a) explosives of spherical shape; and
b) non-spherical charges if the detonation effects are considered at scaled distances greater than 3 $m/kg^{1/3}$.

In case of non-spherical charges at scaled distances smaller than 3 $m/kg^{1/3}$, the provisions of **Q-8.3.3** shall be applicable with explicit modelling of the detonation process.

Q-4.2.2 *Detonation Scenarios*

The standard addresses three scenarios of unconfined explosions based on the location of the detonation with respect to the point of interest:

a) Spherical (free-air) burst (*see* Fig. 24);
b) Hemispherical (surface) burst (*see* Fig. 25); and
c) Air-burst (*see* Fig. 26)

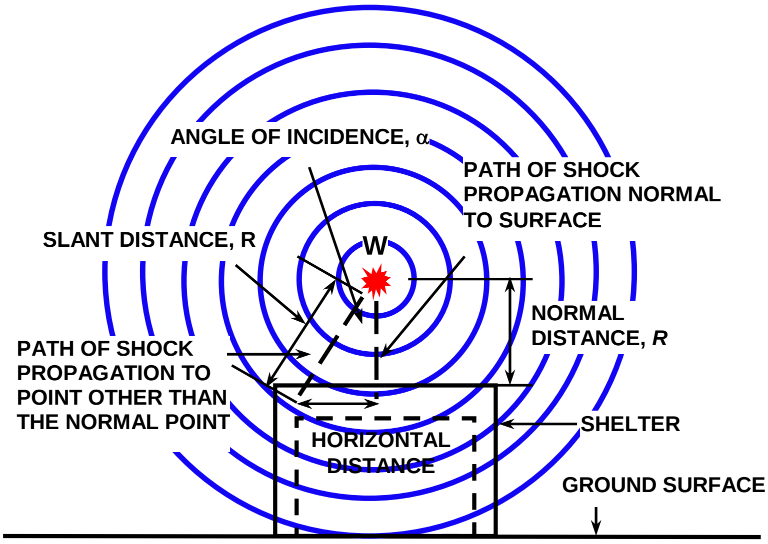


FIG. 24 SPHERICAL (FREE-AIR) BURST

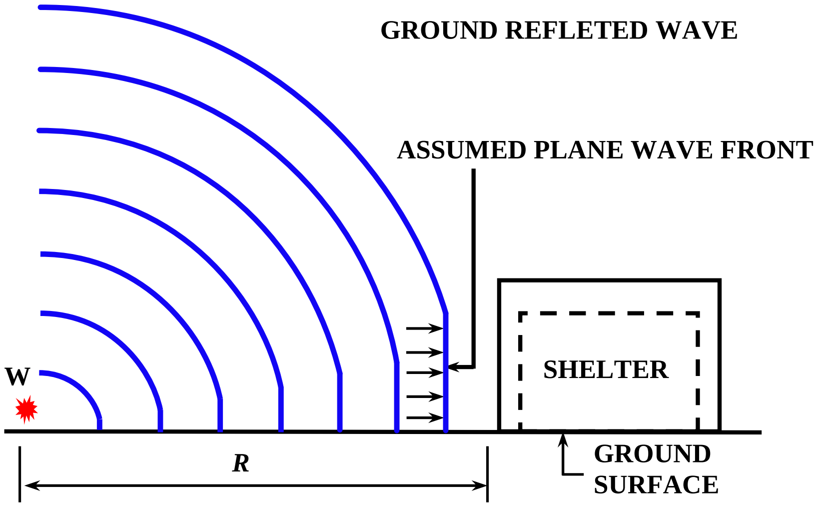


FIG. 25 HEMISPHERICAL (SURFACE) BURST

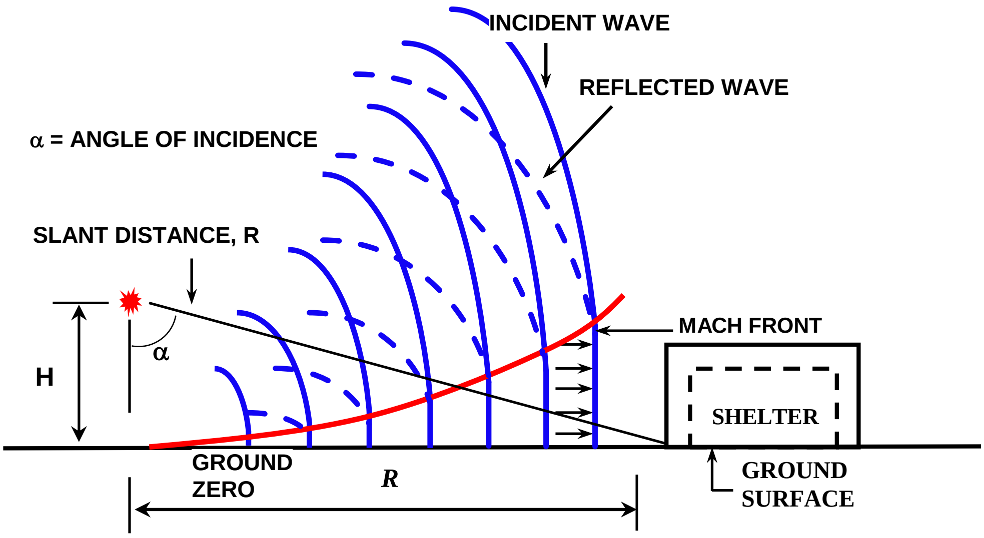
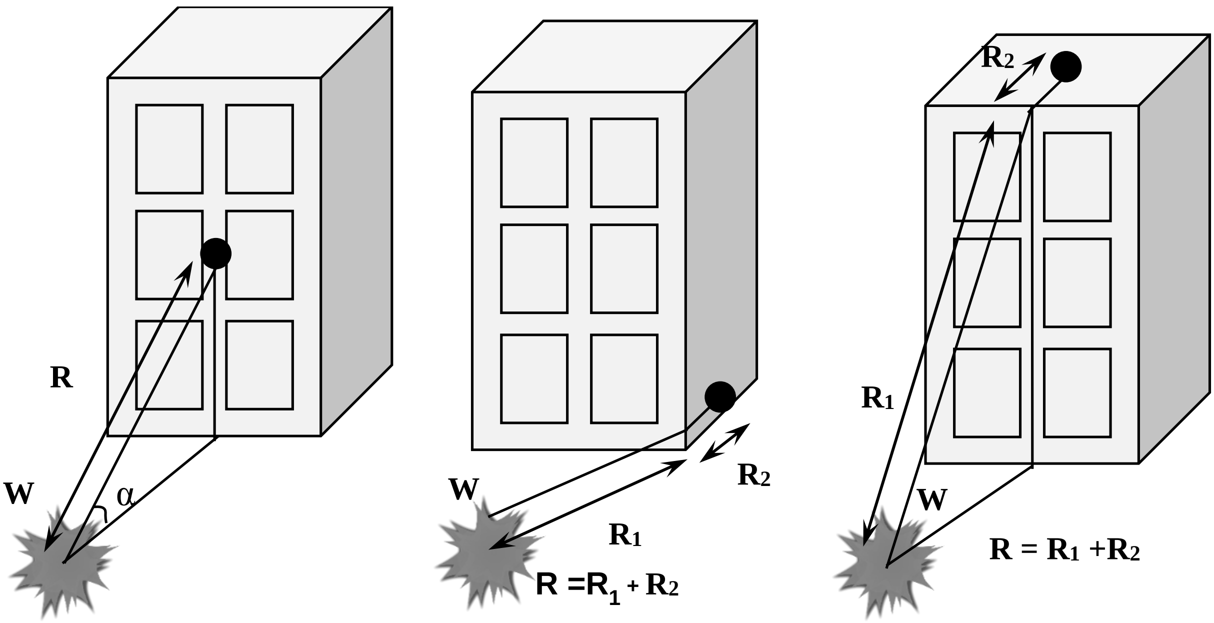


FIG. 26 AIR-BURST

Q-4.2.3 *Standoff Distance*

The standoff distance (*R*) shall be estimated as the distance between the centre of detonation and the point of interest. If the location of interest is not directly in the line of the sight from the centre of detonation, then Fig. 27 shall be referred to estimate the effective standoff distance.



27A FRONT FACE 27B SIDE FACE 27C TOP FACE

FIG. 27 CALCULATION OF STANDOFF DISTANCES AT DIFFERENCE FACES OF A BUILDING STRUCTURE

Q-4.2.4 *Equivalent TNT Charge*

Trinitrotoluene (TNT) shall be used to benchmark all types of explosives. To quantify blast wave parameters (pressure, impulse) from explosives other than TNT, the actual mass of the explosive (M_{EXP}) shall be converted into a TNT-equivalent mass. The TNT equivalent weight (M_{TNT}) of any explosive shall be estimated as:

$$M_{TNT} = \left(\frac{\Delta E_{EXP}}{\Delta E_{TNT}}\right) M_{EXP}$$

Where

ΔE_{EXP} and ΔETNT are the specific energies of the explosive and the TNT, respectively.

The factors presented in Table 54 shall be utilized to obtain equivalent mass of TNT corresponding to the listed explosives.

Table 54 Equivalent TNT Mass of Different Explosives
(*Clause* Q-4.2.4)

Sl No.	**Explosive**	**Density** Mg/m^3	**Equivalent Mass for Pressure**	**Equivalent Mass for Impulse**
(1)	(2)	(3)	(4)	(5)
1)	Ammonia dynamite (50 percent Strength)	NA*	0.90	0.90 ǂ
2)	Ammonia dynamite (20 percent Strength)	NA*	0.70	0.70 ǂ
3)	ANFO (94/6 ammonium nitrate/fuel oil)	NA*	0.87	0.87 ǂ
4)	Composition C-4	1.59 1.59	1.20 1.37	1.19 1.19
5)	Cycloid (75/25 RDX/TNT) (70/30) (60/40)	1.71 1.73 1.74	1.11 1.14 1.04	1.26 1.09 1.16
6)	Gelatine dynamite (50 percent strength)	NA*	0.80	0.80 ǂ
7)	Gelatine dynamite (20 percent strength)	NA*	0.70	0.70 ǂ
8)	HMX	NA*	1.25	1.25 ǂ
9)	Nitrocellulose	1.65 to 1.70	0.50	0.50 ǂ
10)	Nitroglycerine dynamic (50 percent strength)	NA*	0.90	0.90 ǂ
11)	Nitroglycerine (NQ)	1.72	1.00	1.00 ǂ
12)	PETN	1.77	1.27	1.27 ǂ
13)	Picrotol (52/48 Ex D/TNT)	1.63	0.90	0.93
14)	RDX	NA*	1.10	1.10 ǂ

15)	RDX/wax (98/2)	1.92	1.16	1.16 ǂ
16)	RDX/AL/wax (74/21/5)	NA*	1.30	1.30 ǂ
17)	Tetryl	1.73	1.07	1.07 ǂ
18)	Tetrytol (75/25 tetryl/TNT)	1.59	1.06	1.06 ǂ
19)	TNT	1.63	1.00	1.00

NOTE
* NA — Data not available.
ǂ Value is estimated.

Q-4.2.5 *Cube Root Scaling*

Cube root scaling shall be used to relate blast parameters from different sources and standoff distances. The cube-root scaling shall be valid if the conditions specified in **Q-4.2.1** are satisfied.

The blast load parameters can be obtained in scaled form as function of scaled distance, Z, as:

$$Z = \frac{R}{M^{1/3}}$$

where

R = standoff distance of the point of interest from the centre of charge mass; and

M = Mass in equivalent TNT.

The unit for scaled distance shall be m/kg$^{1/3}$, for standoff distance in m and charge mass in kg.

Q-4.3 Performance Expectations

Q-4.3.1 *Categorization of Structures*

Structures shall be categorized based on their importance as per the specification of Table 2.

The categorisation of structures as per Table 55, does not automatically require them to be designed for explosion threats. Only, if the threat assessment carried as per the provisions of **Q-5.1** identifies blast loading due to explosion as a potential threat, the categorisation in Table 55 shall be used to define the importance of a structure.

Table 55 Categories of Structures
(*Clause* Q-4.3.1)

Sl No.	Importance	Structure Type
(1)	(2)	(3)
i)	Low	Residential buildings with 50 or less occupants
ii)	Medium	a) Residential buildings with more than 50 occupants b) Buildings with large congregation, such as schools, public offices, and cinemas c) Industrial buildings with continuous human occupancy
iii)	High	a) Lifeline structures (water, power, communication, transportation, hospital) b) Structures housing essential services that are required for disaster management (for example, emergency relief stores, data centres, governance continuity buildings) c) Financial and commercial establishments of strategic importance

Q-4.3.2 *Structural Performance*

A protection category shall be decided by the building owner based on the importance of the building and required performance. The performance goals for each protection category are listed in Table 56.

Table 56 Protection Category and Associated Performance Goals
(*Clause* Q-4.3.2)

Sl No.	Protection Category	Performance Goal
(1)	(2)	(3)
1)	1	Minimal damage, resumption after brief halt of operations
2)	2	Limited damage, operational after repairs and retrofit
3)	3	Extensive damage, safe replacement, or demolition

Structures of high, medium, and low importance shall be provided with protection categories 1, 2 and 3 respectively, as at least a minimum as specified in Table .

The facility owner may specify a protection category higher than the minimum protection category specified in Table 57 based on the realistic assessment of threat.

Table 57 Protection Category and Associated Category of Structures
(*Clause* Q-4.3.2)

Sl No.	**Building Category**	**Minimum Protection Category**
(1)	(2)	(3)
i)	Low	3
ii)	Medium	2
iii)	High	1

The protection categories specified in shall comply with the deformation limits of structural elements provided in **Q-8.1.4** when using simplified single degree of freedom (SDOF) and multi degree of freedom (MDOF) methods of analysis as per requirements of **Q-8.3.1** and **Q-8.3.2** respectively. Accordingly, all elements shall be designed and detailed to provide deformation limits specified for different protection categories.

If advanced methods of analysis are used as per **Q-8.3.3**, the response and damage in structure and elements for the performance goals of each protection category mentioned in can be independently established. In such case, deformation limits specified in **Q-8.3.1** and **Q-8.3.2** need not to be used.

Q-5 THREAT ASSESSMENT

Q-5.1 Threat Assessment Parameters

A threat assessment shall be carried out to determine all potential sources of detonation.

The threat assessment shall identify two parameters for the purpose of calculation of blast load parameters, namely:

a) charge mass; and
b) standoff distance.

If threat assessment identifies multiple sources of detonation, the effect of blast on the structure shall be assessed considering individual, as well as combinations of all identified detonation threats.

Q-5.2 Design Blast (Reference Explosion)

Q-5.2.1 *Design TNT Equivalence*

If the threat scenario results in identification of one of the detonation sources listed in Table 58, then the associated charge mass may be used for determination of equivalent TNT capacity.

Table 58 Detonation Source and the Associated Charge Mass
(*Clause* Q-5.2.1)

Sl No.	Threat Description	Explosive Capacity (TNT Equivalent)
(1)	(2)	(3)
1)	 Pipe bomb	2.5 kg
2)	 Suitcase	25 kg
3)	 Hatchback	250 kg
4)	 Sedans/SUV	500 kg
5)	 Passenger van/small cargo truck	2 000 kg
6)	 Delivery truck	5 000 kg
7)	 Water tanker/diesel truck/gas truck	15 000 kg

Q-5.2.2 *Standoff Distance*

The standoff distance (R) is site specific and shall be estimated as per the actual geometry and access control of the target structure.

Q-6 BLAST LOAD

Q-6.1 Blast Wave Propagation

The blast loads for design of structures and components shall be determined from blast pressure history.

The variation of pressure at a fixed location away from the centre of detonation shall be obtained using following assumptions:

a) The shock wave due to explosion moves away from the centre of detonation at a speed, U;

b) When the shock wave reaches a point at the arrival time t_a, the pressure rises instantaneously from the ambient pressure (p_a) to a peak pressure. This peak pressure, p_{so}, is known as side-on overpressure or peak incident overpressure;
c) The time needed for the pressure to reach its peak value is very small and for design purposes it is assumed to be zero;
d) The overpressure decreases exponentially from its peak value to the ambient pressure over the positive phase duration (t_o). After the positive phase of the pressure-time profile, the pressure becomes smaller than the ambient value (referred to as negative pressure), and finally returns to the ambient pressure, p_a;
e) The movement of air caused by the shock wave induces dynamic pressure, in addition to blast pressure. Air dynamic pressure q_o, and peak particle velocity u shall be obtained using peak incident pressure, p_{so} (*see* Fig. 32 and Fig. 33); and
f) Shock wave upon reflection from a rigid surface would apply reflected overpressure, p_r, to the reflecting surface. The variation of reflected overpressure with time for normal reflection follows the same trend as the incident overpressure but characterized by significantly higher peak overpressure.

Q-6.2 Decay of Blast Overpressure with Time

The blast overpressure is assumed to vary with time, t according to the following exponential relationship (*see* Fig. 28):

$$p_s(t) = p_o\left(1-\frac{t}{t_o}\right)e^{-b(t/t_o)}$$

where

p_o = peak blast overpressure, which is interpreted as the peak incident and reflected overpressures for the incident and the reflected blast wave, respectively; and

b = decay coefficient; and t_o is the positive phase duration. Special literature is to be referred for the calculation of the blast wave decay parameter, b.

Q-6.3 Blast Overpressure History Parameters

The maximum values of the positive side-on overpressure p_{so}, incident impulse i_s, reflected over pressure p_r, reflected impulse i_r, positive phase duration t_o, shock wave front velocity U, and wave length L_w, for varying scaled distances, are to be obtained for a spherical TNT explosion in free air and a hemispherical TNT explosion on the surface using Fig. 30 and Fig. 31, respectively. Further, air dynamic pressure q_o, and peak particle velocity u, are to be obtained using Fig. 32 and Fig. 33, respectively.

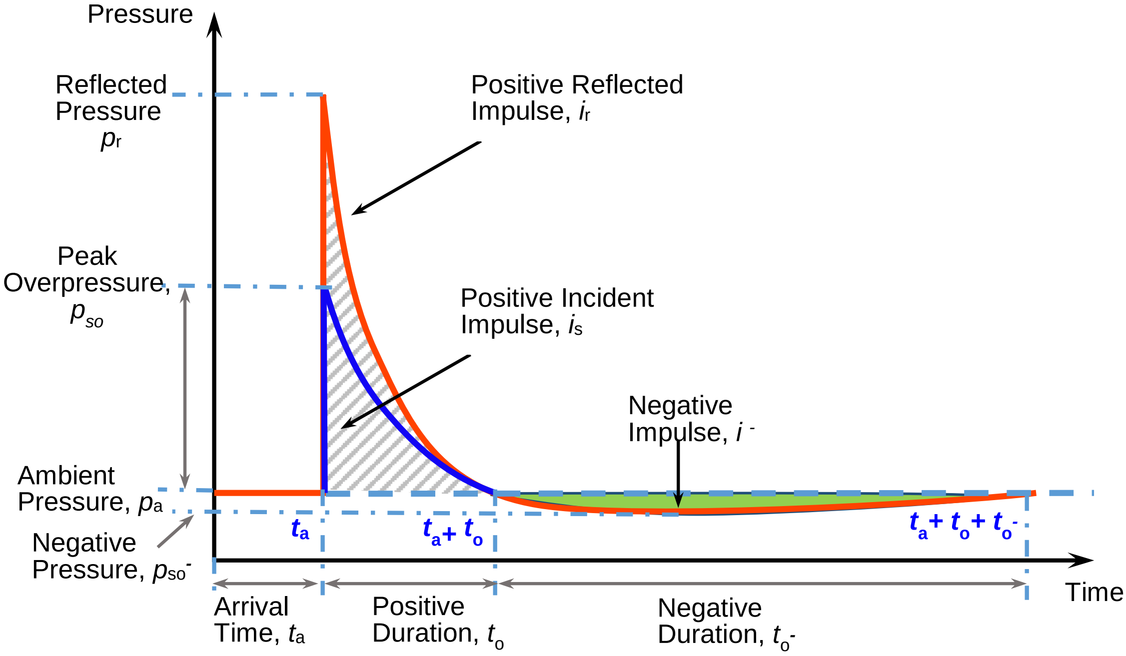


FIG. 28 INCIDENT AND REFLECTED BLAST PRESSURE HISTORY

Q-6.4 Oblique Reflection of Blast Wave

The clauses of this section are applicable to determine the peak reflected overpressure for scenarios where the angle between the wave front and the interacting surface is α (≤ 90°) as shown in Fig. 29. In such case, the peak reflected overpressure (p_{ro}) is to be determined as multiplication of peak incident overpressure (p_{so}) with a reflection coefficient ($C_{r\alpha}$). The reflection coefficient at any angle is to be determined from Fig. 34 for given incident overpressure.

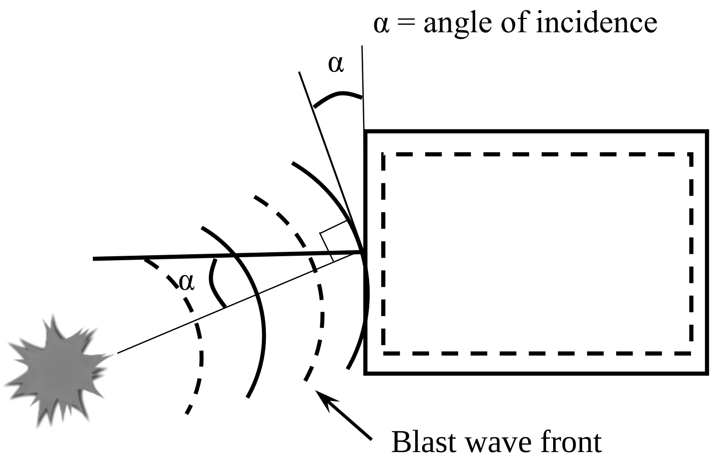


FIG. 29 OBLIQUE REFLECTION OF A BLAST WAVE

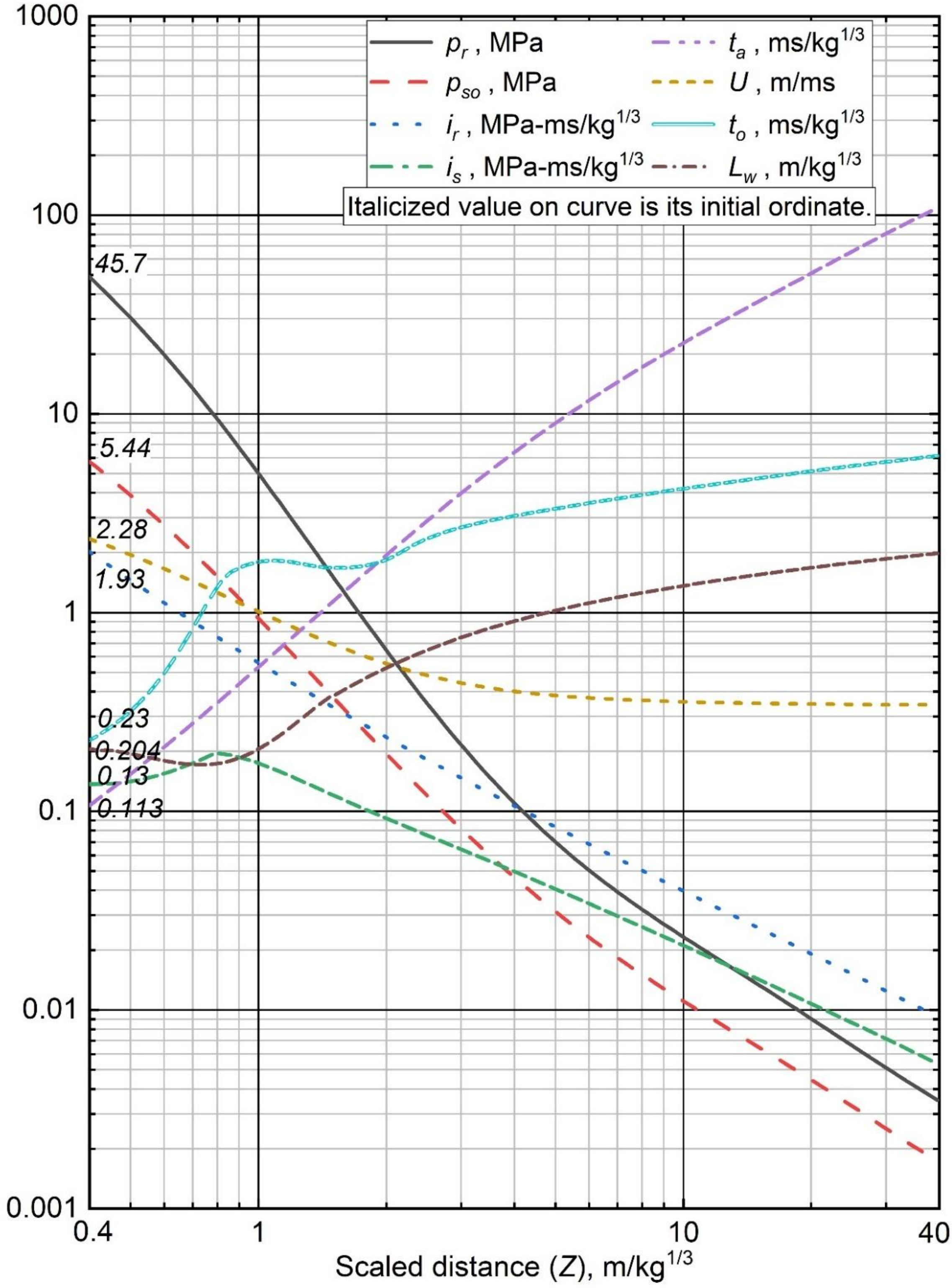


FIG. 30 POSITIVE PHASE SHOCK WAVE PARAMETERS FOR A SPHERICAL TNT EXPLOSION IN FREE AIR

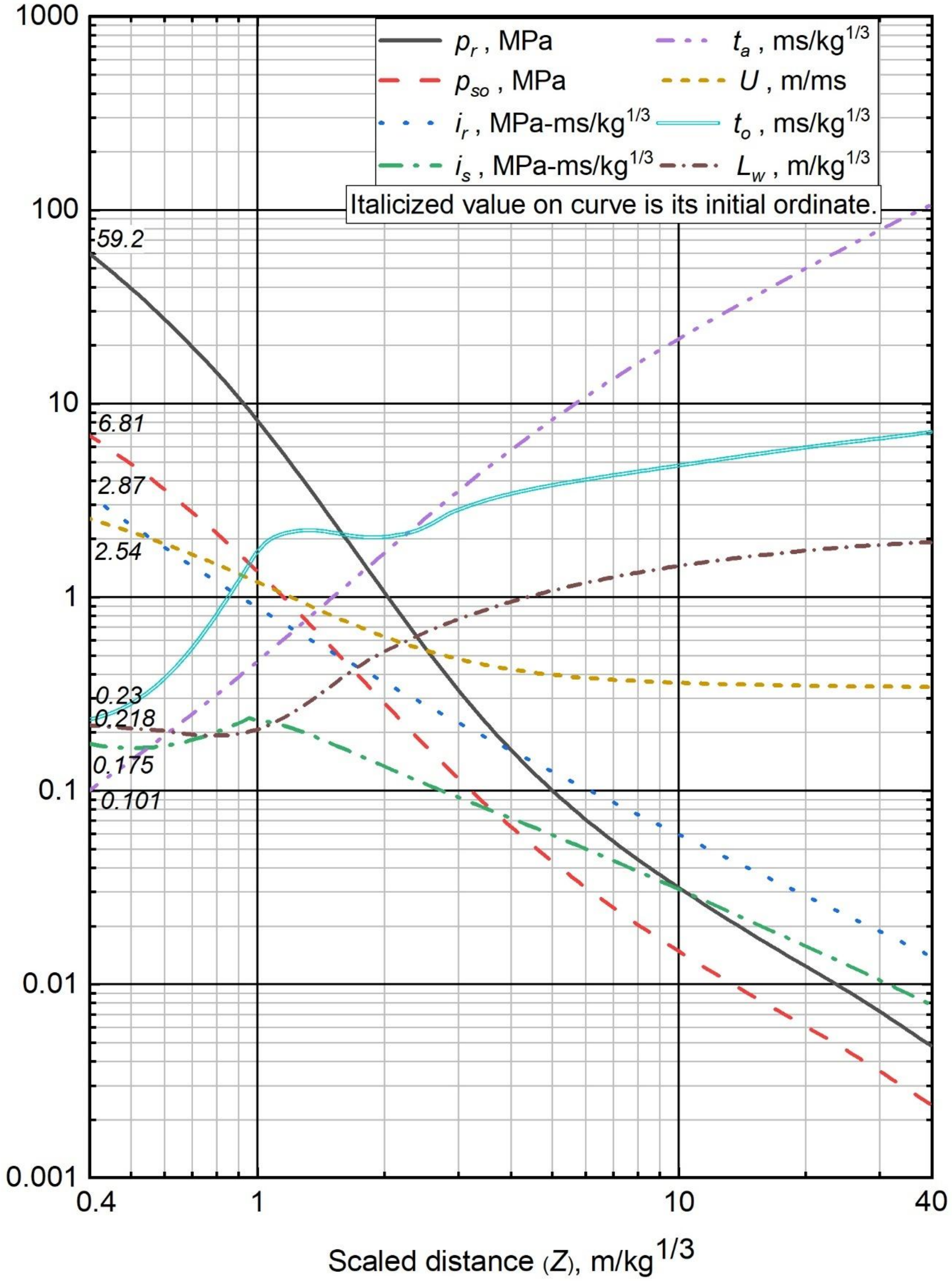


FIG. 31 POSITIVE PHASE SHOCK WAVE PARAMETERS FOR A HEMISPHERICAL TNT EXPLOSION ON SURFACE

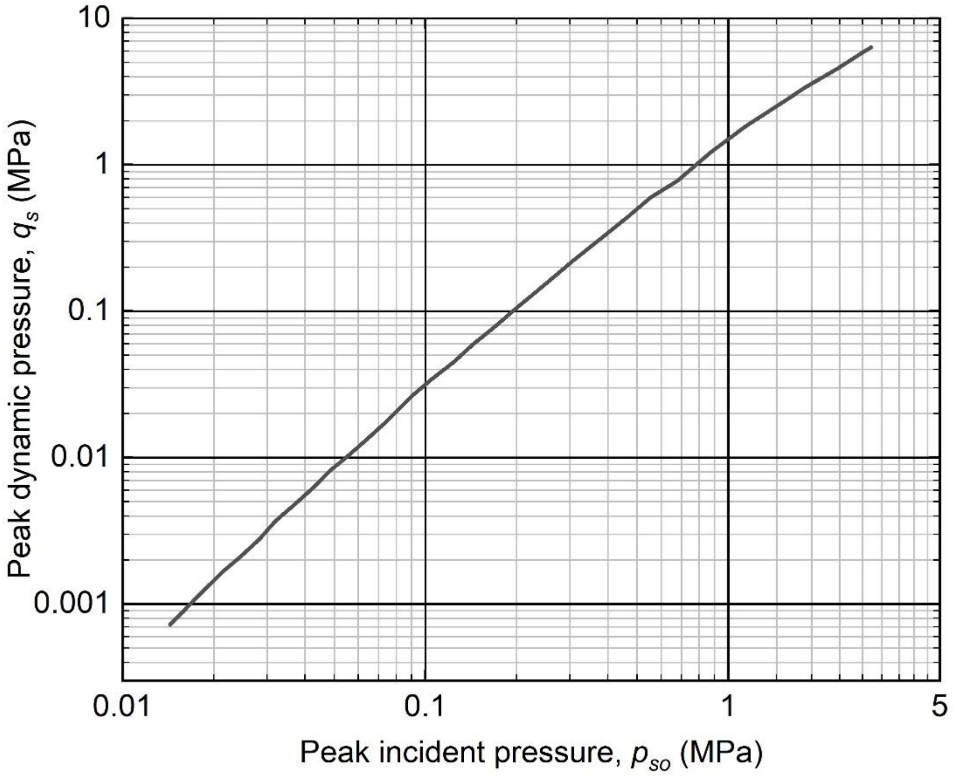


FIG. 32 PEAK DYNAMIC PRESSURE AND PEAK PARTICLE VELOCITY BEHIND A SHOCK FRONT

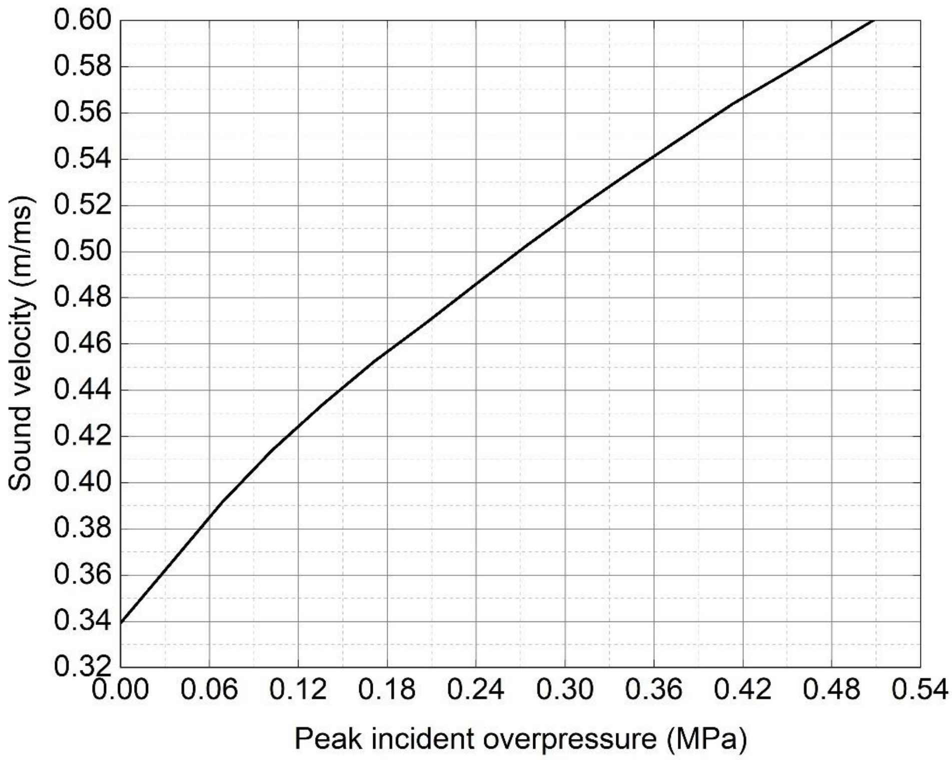


FIG. 33 SOUND VELOCITY IN THE REFLECTED OVERPRESSURE REGION

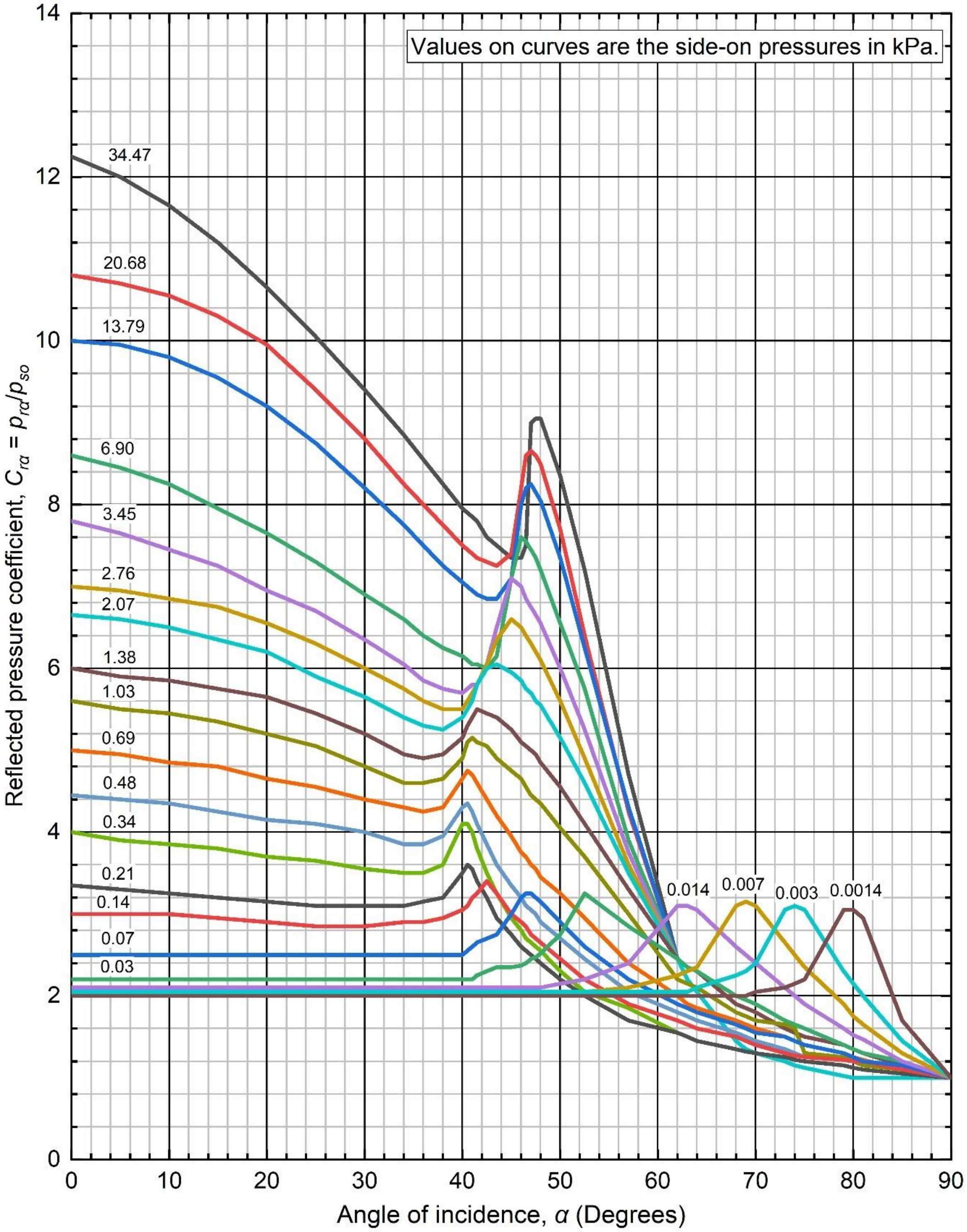


FIG. 34 INFLUENCE OF ANGLE OF INCIDENCE ON THE REFLECTED PRESSURE COEFFICIENT

Q-6.5 Simplified Blast Pressure History

For the purpose of simplified analysis and design, the exponential pressure time relation in the positive phase may be idealised to a triangular pulse e as shown in Fig. 35.

The triangular pulse shall be constructed to have the same peak pressure as in the original exponential blast pressure curve and the positive phase duration (t_d) so adjusted that the area under the curve remains unchanged.

The usage of triangular representation of pressure history shall be governed by relevant provisions in this standard.

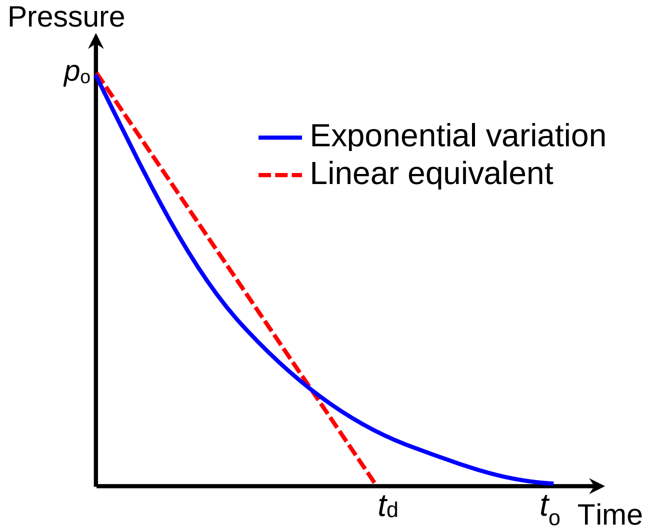


FIG. 35 SIMPLIFIED TRIANGULAR REPRESENTATION OF THE EXPONENTIAL OVERPRESSURE HISTORY

Q-6.6 Ground Shocks

Q-6.6.1 *Ground Shaking Intensity*

The scaled peak particle displacement, peak particle velocity, and soil pressure generated by surface explosion shall be estimated using following equations:

Scaled peak particle displacement,

$$\frac{x}{M^{1/3}} = 60\frac{f}{c}\left(\frac{2.5R}{M^{1/3}}\right)^{1-n}$$

Soil pressure, $p = 160 f \rho c \left(\frac{R}{M^{1/3}}\right)^{-n}$

where

f = coupling factor;
M = explosive weight;

n = attenuation coefficient;
ρ = soil density;
c = soil seismic velocity; and
R = standoff distance.

Table 59 shall be used for taking the values of *c*, *ρc*, and *n*.

The coupling factor *f* shall be taken as 0.14 for explosions above ground.

Table 59 Soil Properties to Calculate Ground Shock Parameters
(*Clause* Q-6.6.1)

Sl No	**Material Description**	**Seismic Velocity,** *c* (m/s)	**Acoustic Impedance,** *ρc* (MPa/m/s)	**Attenuation Coefficient,** *n*
(1)	(2)	(3)	(4)	(5)
i)	Loose, dry sands and gravels with low relative density	182.9	0.27	3 to 3.25
ii)	Sandy loam, loess, dry sands, and backfill	304.8	0.50	2.75
iii)	Dense sand with high relative density	487.7	1.00	2.5
iv)	Wet sandy clay with air voids (greater than 4 percent)	548.6	1.09	2.5
v)	Saturated sandy clays and sands with small amount of air voids (less than 1 percent)	1 524	2.94	2.25 to 2.5
vi)	Heavy saturated clays and clay shales	<1 524	3.40 to 4.07	1.5

Q-6.6.2 *Safe Ground Particle Velocity*

For safety of buried and semi-buried structures, the peak particle velocity, *u*, as calculated using equation provided in **Q-6.6.1** shall not exceed the values given in Table 60.

Table 60 Safe Ground Particle Velocity
(*Clause* Q-6.6.2)

Sl No.	**Type of Rock**	**Safe Ground Particle Velocity** (mm/s)
i)	Soils, weathered or soft rock	50
ii)	Hard rock	70

For safety of unlined underground structures, the peak particle velocity, *u*, as calculated using equation provided in **Q-6.6.1** shall not exceed the values listed in Table 61.

For safety of above ground structures, the peak particle velocity, *u*, measured at the foundation level, as calculated using equation provided in **Q-6.6.1** shall not exceed the values listed in Table 62.

Q-6.6.3 *Decay of Ground Shaking with Distance*

Decay of ground shaking with distance is to be estimated based on principle of mechanics and field measurements

Table 61 PPV Damage Criterion for Unlined Underground Structures
(*Clause* Q-6.6.2)

Sl No.	**Rock Type**	**Unit Weight** kg/m^3	**Compressive Strength** MPa	**Tensile Strength** MPa	**Critical Peak Particle Velocity (m/s)**			
					No Damage	*Slight Damage*	*Intermediate Damage*	*Serious Damage*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Hard rock	2 600 to 2 700	75 to 110	2.1 to 3.4	0.27	0.54	0.82	1.53
		2 700 to 2 900	110 to 180	3.4 to 5.1	0.31	0.62	0.96	1.78
		2 700 to 2 900	180 to 200	5.1 to 5.7	0.36	0.72	1.11	2.09
ii)	Soft rock	2 000 to 2 500	40 to 100	1.1 to 3.1	0.29	0.58	0.90	1.67
		2 000 to 2 500	100 to 160	3.4 to 4.5	0.35	0.70	1.07	1.99

Table 62 Permissible Peak Particle Velocity at the Foundation Level of Structures, in mm/s
(*Clause* Q-6.6.2)

Sl No.	**Type of Structure**	**Dominant Excitation Frequency (Hz)**		
		< 8 Hz	8 Hz to 25 Hz	> 25 Hz
(1)	(2)	(3)	(4)	(5)
i)	Buildings/structures not owned by owners:			
	a) Domestic houses/structure (*kuccha* brick and cement)	5	10	15
	b) Industrial buildings (RCC and framed structures)	10	20	25
	c) Objects of historical importance and sensitive structures	2	5	10
ii)	Buildings belonging to owner with limited span of life			
	a) Domestic houses/structures (kuccha brick and cement)	10	15	25
	b) Industrial buildings (RCC and framed structures)	15	25	50

Q-6.7 Confined Explosions

The provisions given here are applicable for scenarios where detonation of explosives takes place inside closed spaces.

For the purpose of evaluating blast load effects, confined explosions are categorized into:

a) fully vented; and
b) partially vented.

The examples of fully and partially vented structures are shown in Fig. 36.

36A FULLY VENTED   36B PARTIALLY VENTED

FIG. 36 CONFINED EXPLOSION SCENARIO

The variation of pressure at any location inside a closed space due to detonation inside the closed space is obtained using the following assumptions:

a) Surfaces internal structural members are subjected to shock waves from multiple reflections characterized by multiple peaks; and
b) Shock pressure is followed by development of gas pressure of small peak overpressures but significantly longer durations.

The provisions of **Q-6.7** are only applicable when following conditions are satisfied:

a) Structure is of regular geometric shape; and
b) The effect of confined detonation is being evaluated at scaled distances greater than 3 m/kg$^{1/3}$.

Q-6.7.1 *Fully Vented Structures*

Fully vented structures are to be primarily subjected to shock pressures and the effect of gas pressures may be neglected. The reflected overpressure history at any point inside the structure is represented in Fig. 37. The peak pressure and impulse for the first pulse may be obtained using the procedure described in **Q-6.3** as a case of direct reflection of the blast wave propagating from the centre of detonation to the point of interest. The second and the third triangular pulse shall be constructed from the characteristics of the first pulse.

Further simplification of pressure variation in Fig. 37 is allowed if the ratio of total load duration ($5t_a + t_d$) to the time-period of the structure is smaller than 0.1. In such cases, the three pulses of pressure history in Fig. 37 may be combined into a single pulse of peak pressure 1.75 p_{ro} and duration equal to t_d.

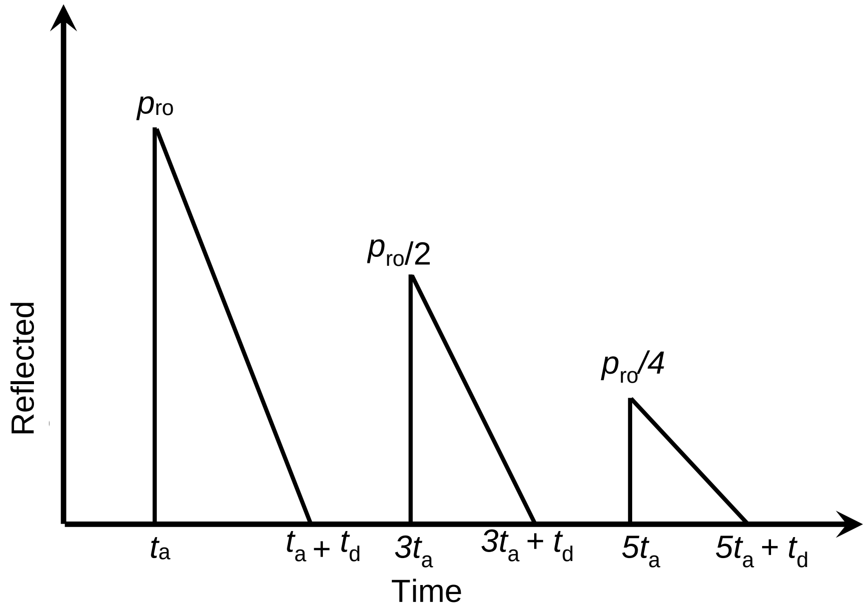


FIG. 37 REFLECTED OVERPRESSURE HISTORY FOR CONFINED DETONATION IN A FULL-VENTED SPACE

Q-6.7.2 *Partially Vented Explosion*

Partially vented explosions are characterized by long duration gas pressures with minor effect of the shock pressures.

Wherever, required, the arrival time of the peak gas pressure is taken as the end time of the shock phase, that is, $5t_a + t_d$, as determined in **Q-6.7.1**.

The blast load effects of confined explosion in partially vented structure is evaluated using advanced numerical technique as per the provisions of **Q-8.3.3**.

The interior surface of a partially vented structure subject to confined explosion may conservatively be designed for the peak gas overpressure, p_g, which needs to be applied statically.

This peak gas overpressure is obtained from the chart in Fig. 38. The use of charts requires following condition to be satisfied:

$$0 \leq A / V_f^{2/3} \leq 0.022$$

where

A = structure's total vent area; and

V_f = free volume which shall be calculated as the total volume excluding the volume of all interior equipment and structural elements.

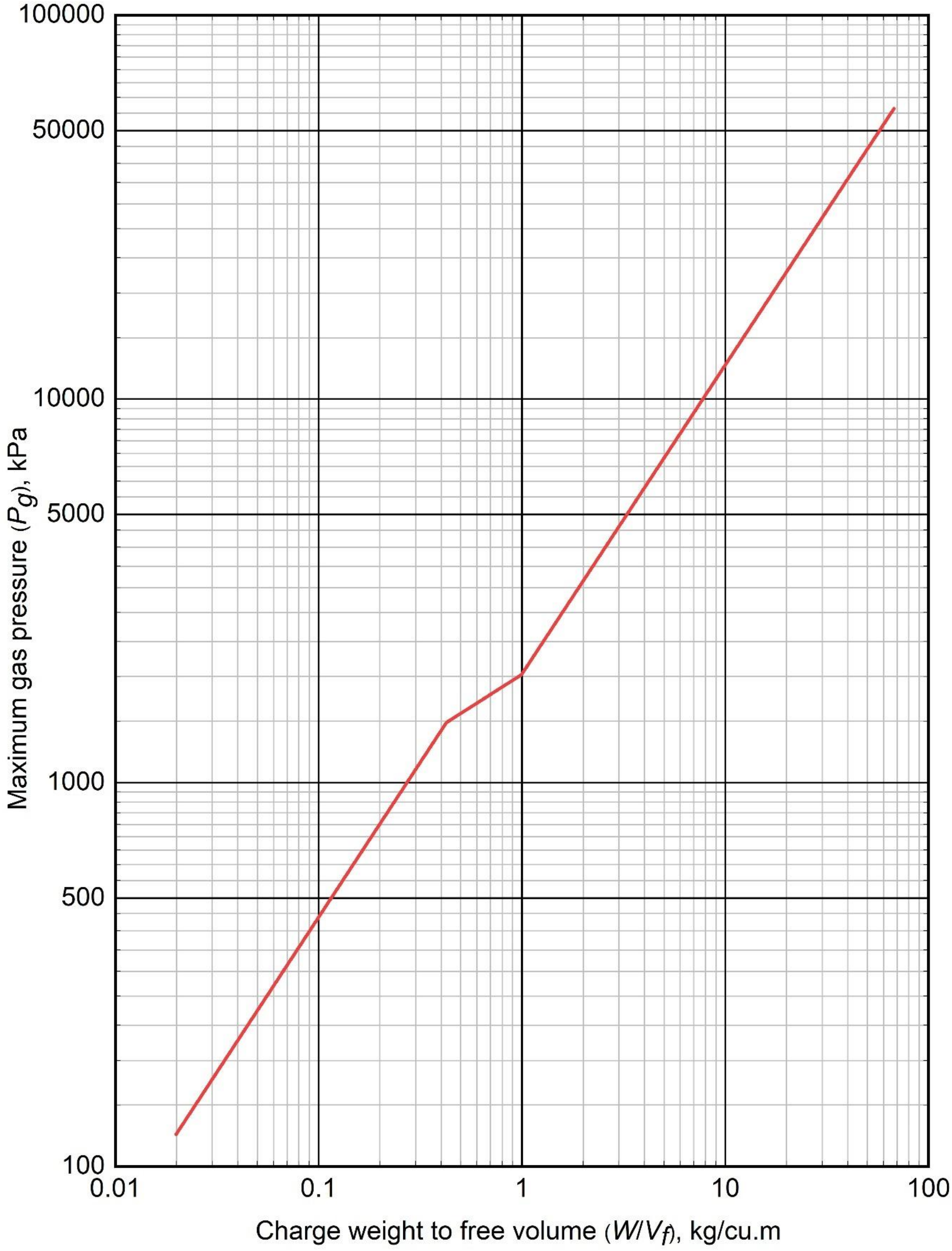


FIG. 38 PEAK GAS PRESSURE PRODUCED BY A TNT DETONATION IN A PARTIALLY VENTED STRUCTURE

Q-7 EFFECTS OF BLAST ON STRUCTURES

Q-7.1 Above Ground Structures

Q-7.1.1 *Types of Structures*

Structures are categorized into following two types for the purpose of determining its interaction with the blast wave and calculating effective blast overpressure history:

a) *Diffraction type structures* — These are the closed structures without openings, with the total area opposing the blast. These are subjected to both the shock wave overpressure $p_{so}$and the dynamic pressures q caused by blast wind; and

b) *Drag type structures* — These are the open structures composed of elements like beams, columns, trusses, etc, which have small projected area opposing the shock wave. These are mainly subjected to dynamic pressures q.

Q-7.1.2 *Closed Rectangular Structures*

Q-7.1.2.1 *Front face*

The peak reflected overpressure at any point on the front face of a closed rectangular structure shall be obtained using the following equation.

$$p_{r\alpha}=C_{r\alpha}p_{so}$$

where

p_{so} = peak side-on pressure that shall be obtained using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively, using the effective scaled distance; and

$C_{r\alpha}$ = reflection co-efficient, which shall be determined from Fig. 34 using the angle of reflection and the side-on pressure.

It is permitted to neglect spatial variation of pressure on the front face by assuming an equivalent uniform reflected overpressure which is equal to the peak reflected overpressure on the front face that is closest to the point of detonation. This, under most scenario, corresponds to the peak reflected overpressure, p_{ro}, at normal incidence (α = 0 degrees).

The reflected overpressure, p_r, on the front face drops from the peak value p_{ro} to zero in time t_{rf}, whichever, is calculated as, $t_{rf} = 2i_{ro}/p_{ro}$, where p_{ro} and i_{ro} shall be obtained from using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively.

The stagnation overpressure, p_s, at the edges of the front face drops from the peak value, $p_{so} + C_d q_o$, to zero in time t_{of}, whichever, is calculated as, $t_{of} = 2i_{so}/p_{so}$, where p_{so} and i_{so} shall be obta*i*ned using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively. The drag coefficient, C_d, shall be taken as 1.0. The peak dynamic pressure, q_o, shall be determined from Fig. 32.

The reflected overpressure acting at a point on the front face drops from the peak value p_{ro} to overpressure (p_{so} + $C_d q$) in clearance time t_c or t_{of}, whichever, is less (*see* Fig. 40).

The drag coefficient, C_d, for different faces of closed rectangular structures located above ground shall be taken as per Table 63.

The clearing time, t_{cp}, for a point on the front wall is given by

$$t_{cp} = \frac{3S}{U}$$

where

S = H or W/2 whichever, is less (*see* Fig. 39); and

U = shock front velocity to be obtained from using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively.

The average clearing time, t_c, required to clear the front face of reflection effects from the roof down and inwards from the sides shall be calculated using the following expression:

$$t_c = \frac{4HW}{C_r(W+2H)}$$

where

C_r = velocity of sound in the reflected overpressure region that shall be determined using Fig. 33.

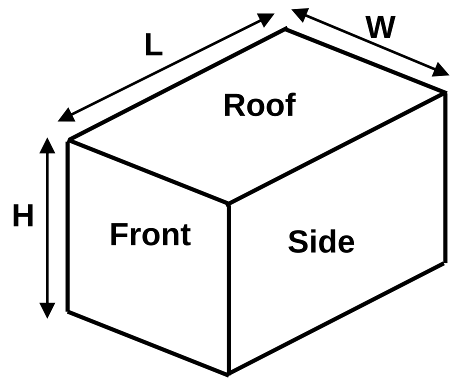


FIG. 39 ABOVE GROUND RECTANGULAR STRUCTURE

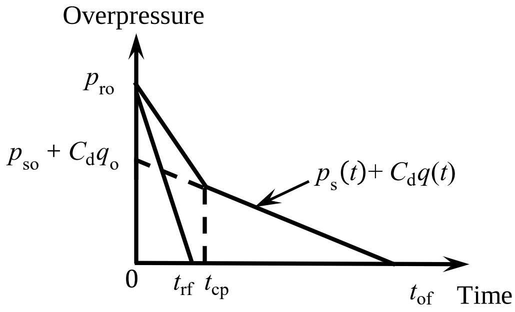


FIG. 40 REFLECTED OVERPRESSURE AT A POINT ON THE FRONT SURFACE OF FINITE REFLECTING SURFACE

The average net loading on the front face (*W* × *H*) as a function of time is shown in Fig. 41 depending on whether t_c is smaller than or equal to t_{of}. The pressures p_{ro}, p_{so} and *q*, the impulses t_{ro} and I_{so} are for the actual explosion that are determined according to **Q-6.3**.

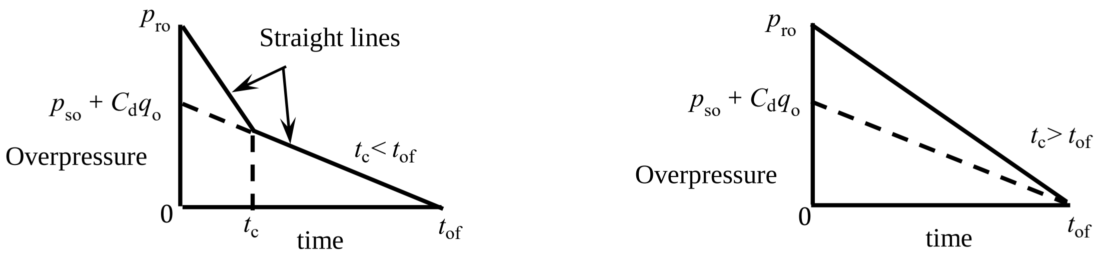


FIG. 41 REFLECTED OVERPRESSURE HISTORY ON THE FRONT FACE OF A CLOSED RECTANGULAR STRUCTURE

Q-7.1.2.2 *Rear face*

Using the pressures for the actual explosion, the average loading on the rear face (*W* × *H*) in Fig. 39 shall be taken as shown in Fig. 42, where the time has been taken from the instant the shock first strikes the front face. The time intervals of interest are the following:

$\frac{L}{U}$ = the travel time of shock from front to rear face; and

$\frac{4S}{U}$ =pressure rise time on the back face.

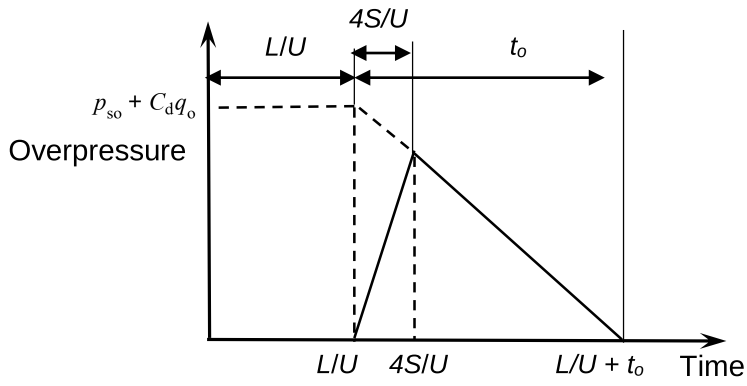


FIG. 42 OVERPRESSURE VERSUS TIME FOR REAR FACE

Q-7.1.2.3 *Roof and side walls*

The average pressure versus time curve for roof and side walls is given in Fig. 43A when t_d is greater than the transit time $t_t = L/U$. When t_t is greater than t_d, the load on roof and side walls may be considered as a moving triangular pulse having the peak value of overpressure $p_{so}+C_d q_o$ and time t_o as shown in Fig. 43B.

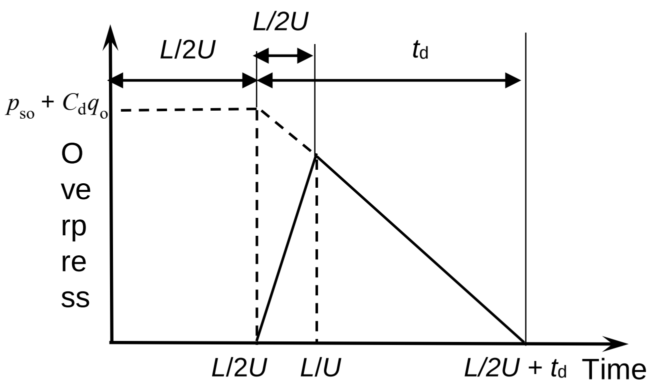


FIG. 43A AVERAGE PRESSURE DIAGRAM $t_d > t_t$

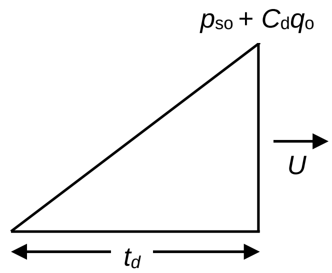


FIG. 43B MOVING PRESSURE PULSE FOR $t_t > t_d$

FIG. 43 OVERPRESSURE VERSUS TIME FOR ROOF AND SIDE WALLS

Table 63 Drag Coefficient C_d
(*Clause* Q-7.1.2.1)

Sl No.	Shape of Element	Drag Coefficient C_d	Remarks
(1)	(2)	(3)	(4)
i)	Front vertical face	1.0	For closed rectangular structures located above ground
ii)	Roof, near and side faces for:		
	a) q_o = 0 kPa to 170 kPa	- 0.4	
	b) q_o = 170 kPa to 350 kPa	- 0.3	
	c) q_o = 350 kPa to 1 MPa	- 0.2	

Q-7.1.2.4 *Overturning of structure*

The average net load as a function of time which tends to cause sliding and overturning of the building is obtained by subtracting the loading on back face from that on the front face.

Q-7.1.3 *Closed Cylindrical Arch-Shape Structures*

Q-7.1.3.1 *Gable ends*

The loading may be taken same as for front and rear faces of above ground rectangular structure.

Q-7.1.3.2 *Curved surface*

The direction of shock wave propagation is taken transverse to the ridge of the structure and since the usual arch spans are large so that the transit time t_t is greater than the positive phase time t_d, average loading condition cannot be assumed. Therefore, the loading on curved surface may be taken as a moving triangular pulse as shown in Fig. 43B.

Q-7.1.3.3 *Closed dome structures*

The loading on a domical structure may be taken as a moving triangular pressure pulse as shown in Fig. 43B. The variation of pressure transverse to the direction of propagation of the pulse may be considered symmetrical varying according to cosine θ where the angle θ is measured from longitudinal vertical section of the dome, as shown in Fig. 44.

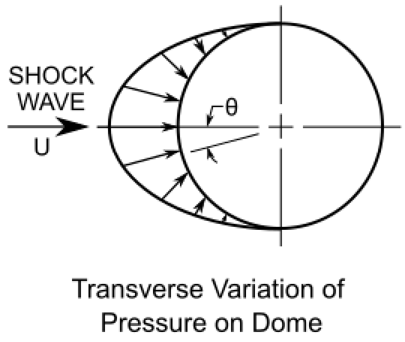


FIG. 44 TRANSVERSE VARIATION OF PRESSURE ON DOME

Q-7.2 Blast Load on Below-Ground Structures

Q-7.2.1 *Types of Structures*

The below-ground structures are classified into buried and semi-buried structures depending upon the earth cover and slopes of earth berms. The buried structures are subjected only to the general overpressure p_{so}, the reflected and dynamic pressures being neglected. The semi-buried structures are subjected to partial dynamic pressures besides the general overpressure. Both are acted upon by air-induced ground shock also.

For assessing the effects of above-ground blasts on below-ground structures, dynamic finite element analysis can be performed considering the flexibility of soil and interactions of blast induced ground waves with the structure.

Q-8 RESPONSE OF STRUCTURAL ELEMENTS

Q-8.1 Properties of Materials

The nominal properties of materials are obtained as per relevant codes and guidelines. The change in material properties due to high strain rates associated with blast response are to be incorporated using the provisions in **Q-8.1.1** and **Q-8.1.2**.

Q-8.1.1 *Elastic Modulus and Poisson's Ratio*

The elastic modulus and Poisson's ratio of the constituent materials used in the structural elements shall be estimated based on relevant standards as applicable to that material.

No increase in elastic modulus and Poisson's ratio is to be considered for the calculation of blast response of structures.

Q-8.1.2 *Dynamic Strength*

Q-8.1.2.1 *Design strength of structural steel*

The average dynamic increase factors of strength for structural, carbon, mild, weldable or rivet steels and high strength alloy steels are listed in Table 64 and Table 65.

Table 64 Dynamic Increase Factors for Mild Steels
(*Clause* Q-8.1.2.1)

Sl No.	**Material**	**Dynamic Increase Factors**
(1)	(2)	(3)
i)	Yield strength in bending and shear	1.25
ii)	Yield strength in tension and compression	1.20
iii)	Ultimate strength	1.00

Table 65 Dynamic Increase Factors for High Strength Steels
(*Clause* Q-8.1.2.1)

Sl No.	**Material**	**Dynamic Increase Factors**
(1)	(2)	(3)
i)	Yield strength in bending and shear	1.10
ii)	Yield strength in tension and compression	1.05
iii)	Ultimate strength	1.00

Q-8.1.2.2 *Design strength of reinforced concrete*

The average dynamic strength increase factors for concrete are listed in Table 66 and for reinforcing steel are listed in Table 67.

Table 66 Dynamic Increase Factors for Reinforced Concrete
(*Clause* Q-8.1.2.2)

Sl No.	**Strength**	**Dynamic Increase Factors**
(1)	(2)	(3)
i)	Flexure	1.15
ii)	Compression	1.10
iii)	Diagonal tension, direct shear, and bond	1.00

Table 67 Dynamic Increase Factors for Reinforcing Steel
(*Clause* Q-8.1.2.2)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in flexure	1.2
ii)	Yield strength in compression, diagonal tension, direct shear, and bond	1.1
iii)	Ultimate strength in flexure	1.1
iv)	Ultimate strength in compression, diagonal tension, direct shear, and bond	1.0

Q-8.1.2.3 *Design strength for masonry or plain concrete*

The average dynamic strength increase factors for masonry and plain concrete are listed in Table 68.

Table 68 Dynamic Increase Factors for Masonry and Plain Concrete
(*Clause* Q-8.1.2.3)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Flexure	1.15
ii)	Compression	1.10
iii)	Diagonal tension and direct shear, and bond	1.00

Q-8.1.3 *Design Strength*

For obtaining design strength, the resistance of various elements is to be estimated using the strength increased by the dynamic increase factors mentioned in **9.1.2**.

Q-8.1.4 *Deformation Limits*

Structural elements subjected to blast loading can be designed subject to certain allowable dynamic ductility ratios and support rotations.

The blast resistance of a flexural member is to be expressed in terms of maximum permissible deflection. Support rotation and ductility is to be used to characterize the blast resistance of reinforced concrete and structural steel members, respectively.

The support rotation shall be defined as per Fig. 45.

The maximum permissible deflection defines the energy absorption capacity of the element which is equal to the area under the resistance versus deflection curve. For a given blast scenario, greater the permissible deflection, lesser will be the maximum resistance required in the member.

The allowable deformation parameters are included in Table 69.

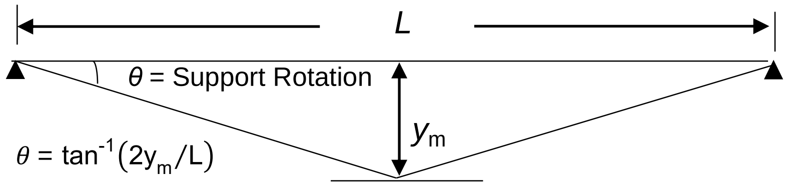


FIG. 45 SUPPORT ROTATION IN A FLEXURAL MEMBER

Table 69 Allowable Deformation Parameters
(*Clause* Q-8.1.4)

Sl No.	Material	Low Damage	Medium Damage	High Damage
(1)	(2)	(3)	(4)	(5)
i)	Ductility ratio in structural steel members subjected to bending and direct stresses	3	10	20
ii)	Support rotation in reinforced concrete members subjected to bending and direct stresses (in degrees)	1	2	4

Q-8.2 Modelling of Structural Members

Q-8.2.1 *Geometry, Material and Boundary Conditions*

The methods of modelling are applicable to structural members of regular prismatic sections.

Q-8.2.2 *Equivalent Single Degree of Freedom (SDOF) Model*

The structural element or system may be replaced by an equivalent single spring-mass system having effective stiffness k_E and effective mass equal to $K_{LM}M_t$, where M_t is the actual mass of the member under consideration and K_{LM} is a load-mass factor depending upon the stiffness and mass distribution in the member and its boundary conditions. The equivalent system is defined so that the deflection of the equivalent single mass is the same as that of some significant point in the given structure. The effective stiffness k_E is defined with respect to the deflection of this point.

The load-mass factor K_{LM} is equal to the ratio of mass factor K_M to the load factor K_L. The factors are evaluated on the basis of an assumed deflected shape of the structure as given below:

$$K_{\mathrm{M}} = \frac{1}{M_{\mathrm{t}}}\left\{\sum_{r=1}^{n} M_{\mathrm{r}}\emptyset_{\mathrm{r}}^{2} + \int m\emptyset^{2}(x)\,dx\right\}$$

$$K_{\mathrm{L}} = \frac{1}{P_{\mathrm{t}}}\left\{\sum_{r=1}^{j} p_{\mathrm{r}}\emptyset_{\mathrm{r}} + \int p(x)\emptyset(x)dx\right\}$$

where

M_t = total actual mass;
n = number of concentrated masses;
M_r = concentrated mass at point r ;
$\phi_r \phi_{\mathrm{r}}$ = deflection at point r of an assumed deflected shape for concentrated loads;
m = distributed mass intensity per unit length;
$\phi_{(X)}$ = deflection at point x of an assumed deflected shape for distributed loads;
P_t = total dynamic load (at any instant of time);
j = number of concentrated load points;
P_r = concentrated dynamic force at point r; and
$p_{(X)}$ = intensity of distributed dynamic load per unit length.

Q-8.2.2.1 *Effective single SDOF parameters*

Values of the factors k_E and K_{LM} for few structural members are given in Table 70 to Table 72. The effective SDOF parameters for other type of structural members shall be calculated by assuming an appropriate deflected shape as per **Q-8.2.2.2**.

Q-8.2.2.2 *Deflected shape*

The deflected shape is suitably chosen to resemble as far as possible the true deflected shape taking into consideration whether the structure or member remains elastic or goes into the plastic range. It may be taken the same as due to static application of the dynamic load on the structure. The deflected shape is normalized such that $\varphi(x)=1$ at the point with respect to which the effective stiffness k_E is defined.

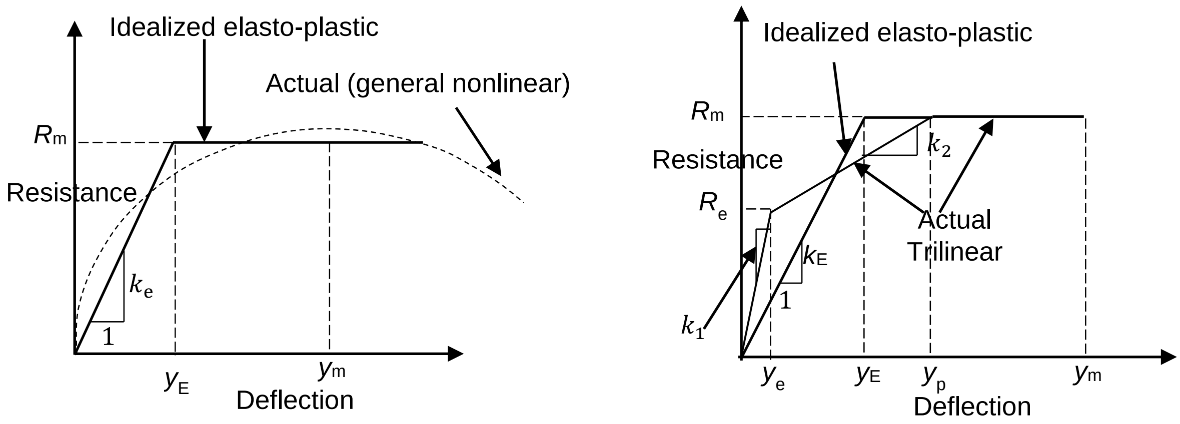
Q-8.2.2.3 *Effective time-period*

The effective time period *T* of the structural member may be calculated from the equation:

$$T = 2\pi \sqrt{\frac{K_{LM}\, M_t}{k_E}}$$

Q-8.2.3 *Resistance Function*

The resistance versus deflection diagram of a structural element shall be idealized as elasto-plastic (*see* Fig. 46) by keeping the area under the actual and idealized curves about the same up to the maximum permissible deflection defined as per **Q-8.1.4**.



46A GENERAL NONLINEAR 46B TRILINEAR

FIG. 46 IDEALIZATION OF RESISTANCE DEFLECTION DIAGRAM

A trilinear resistance versus deflection diagram may be simplified to its elasto-plastic (bilinear) equivalent of same peak resistance (R_m) and the elastic parameters calculated using following equations:

$$y_E = y_e + y_p \left(1 - \frac{R_m}{R_e}\right)$$

$$k_E = \frac{R_m}{y_E}$$

Q-8.3 Methods of Analysis

These provisions apply to structures, components, and systems subject to near-field and far-field detonations but not for contact or near-contact detonations.

Q-8.3.1 *Analysis of Members*

Blast loaded structural components of a structure may be analysed independently provided it can be demonstrated that such independent member analysis would yield conservative performance of the member and the structure.

Q-8.3.1.1 *Equivalent SDOF analysis*

The equivalent SDOF analysis may be used to determine response of structural members subject to blast loads when the following criteria are satisfied:

a) The equivalent mass, stiffness, and damping parameters in the model must capture the dynamic response and failure mode of the structural member being analysed; and
b) A single flexural deformation mode controls the dynamic response of the structural system.

The equivalent SDOF analysis shall make use of following parameters:

a) Blast-pressure history obtained using **Q-6.3**;
b) The effective time-period calculated using **Q-8.2.2.3**;
c) Resistance function of the structural element established using provisions of **Q-8.2.3**; and
d) Maximum permissible deflection provided in **Q-8.1.4**.

Q-8.3.1.2 *Blast overpressure history*

The exponential blast pressure history of Fig. 28 shall be idealized to a triangular pulse (Fig. 47A) as per provisions of **Q-6.5** for the purpose of obtaining equivalent SDOF blast loads. Such idealization may neglect the effect of negative phase, unless otherwise it is expected to influence the flexure response of the structural member.

The positive phase duration of the triangular loading shall be calculated as:

$$t_d = \frac{2 i_{ro}}{p_{ro}}$$

If the influence of clearing on the blast pressure history applied to the front face of the structural element is found to be significant, then the equivalent SDOF blast loads shall be idealized to Fig. 47B using **Q-7.1.2**.

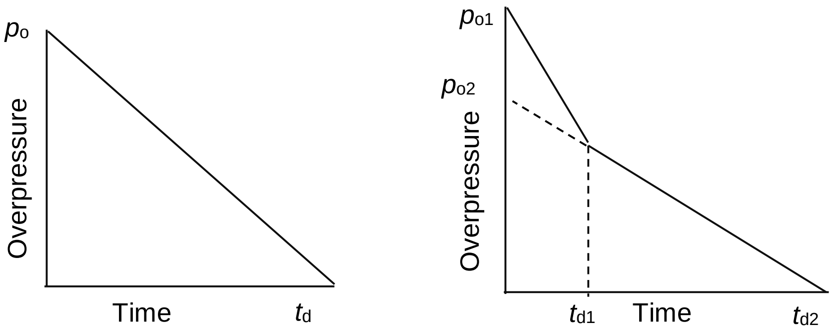


FIG. 47A LINEAR FIG. 47B BILINEAR

FIG. 47 IDEALIZED BLAST PRESSURE HISTORIES

When the ratio of time duration t_{d1}, or t_{d2}, to the natural period of the element is less than 0.1, the problem may be considered as an impulse problem taking the area under the pressure versus time curve as impulse per unit area. In such a case, the shape of pressure-time curve is not important.

Q-8.3.1.3 *Uniform blast pressure distribution*

A uniform distribution of blast pressure over the length of the structural member may be considered provided one of the following conditions are satisfied:

a) The minimum scaled distance to the structural component is greater than 1.2 $m/kg^{1/3}$; and
b) The variation of peak reflected pressures over the middle two-third of the component span length is less than 25 percent.

For uniform distribution of blast pressure, the pressure value calculated at the closest point on the structure from the centre of detonation may be considered over the whole reflecting surface.

Q-8.3.1.4 *Non-uniform blast pressure distribution*

For members subjected to spatially non-uniform blast loads, the effect of non-uniformity on the response shall be considered.

These provisions shall be applicable for non-uniform blast pressure distribution, provided the conditions outlined in **Q-9.3.1.3** are not satisfied and the minimum scaled distance to the structural component is between 0.4 $m/kg^{1/3}$ and 1.2 $m/kg^{1/3}$.

An equivalent impulse weighted using deflected shape function shall be considered for the structural component as per following expression:

$$i_e = \frac{\int_0^L \int_0^H i(x,y)\phi(x,y)dxdy}{\int_0^L \int_0^H \phi(x,y)dxdy}$$

where

i_e = equivalent impulse of uniformly distributed load;
L = length of loaded area;
H = width of loaded area;
$\emptyset(x,y)$ = deflected shape function of blast-loaded component; and
i(x,y) = non-uniform impulse applied to loaded area.

The simplified triangular representation of the pressure time history shall be constructed as per provisions of **Q-6.5** by calculating the positive loading phase duration using the equivalent uniform impulse (i_e) and the peak overpressure at the midspan or geometric centre of the reflecting surface.

A safety factor of 1.25 shall be used if a non-uniform blast load is represented as an equivalent uniform impulse in a blast analysis.

Q-8.3.1.5 *Elastic SDOF response*

For elastic analysis (ductility ratio μ ≤ 1.0) of structures, the effective stiffness k_E and load mass factor K_{LM} shall be used as given in Table 70 to Table 72.

For elastic response of an SDOF system, the peak response (y_m) and the time to peak response (t_m) may be obtained using following expressions:

$$\frac{y_m}{y_0} = 2 - \frac{t_m}{t_d}; \qquad \text{when } t<t_d$$

$$t_m = \frac{2}{\omega}\tan^{-1}(\omega t_d) \qquad \text{when } t<t_d$$

$$\frac{y_m}{y_0} = \frac{1}{\omega t_d}\sqrt{2(1-\cos\omega t_d - \omega t_d \sin\omega t_d) + (\omega t_d)^2}; \text{ when } t > t_d$$

$$t_m = \frac{1}{\omega}\tan^{-1}\left(\frac{1-\cos\omega t_d}{\sin\omega t_d - \omega t_d}\right) \text{ when } t>t_d$$

where

y_m = peak elastic displacement;
y_o = peak static displacement; and
t_m = time to the peak displacement.

Q-8.3.1.6 *Inelastic SDOF response*

For elasto-plastic analysis, k_E shall be used as given in Table 70 to Table 72 but value of K_{LM} may be chosen in between the elastic and plastic cases depending upon the ductility factor.

Based on the elasto-plastic resistance-deflection curve shown in Fig. 46 and triangular pressure-time curve shown in Fig. 47A, the ratio of resistance R_m, required to the peak dynamic load (F_1) is given in Fig. 48 for various values of t_d/T and μ.

The time to peak deflection (t_m)shall be obtained from Fig. 49 for various values of td/T and μ.

When the time ratio, t_d/T, is less than 0.1, the ratio k may be computed from the following equation:

$$k = \frac{\pi}{\sqrt{2\mu-1}} \cdot \frac{t_d}{T}$$

When the pressure-time diagram is given by Fig. 47B, the following equation shall be satisfied:

$$\frac{P_{01}}{R_m} k_1 + \frac{P_{02}}{R_m} k_2 = 1$$

where

R_m = the required resistance; and

$k_{1,} k_2$ = the values of ratios k for ductility ratio μ and time ratios t_{d1}/T and t_{d2}/T respectively.

For elasto-plastic design of fixed slabs, the modified value of k_E is to be worked out in accordance with Fig. 46B using the stiffness values of slab in elastic and elasto-plastic cases as given in Table 72 and $K_{LM}K_{LM}$ is to be suitably chosen depending upon the ductility factor.

The value of modular ratio shall be taken the same as in static design for calculating EI .

For calculating moment of inertia I of reinforced concrete sections, the concept of effective transformed area shall be used.

For analysis of reinforced concrete sections when the simplified elasto-plastic resistance function is used, the elastic stiffness shall be based on effective moment of inertia that shall be calculated as the average of the gross moment of inertia of the transformed section and the cracked moment of inertia.

The gross and cracked moment of inertias shall be obtained from Fig. 50.

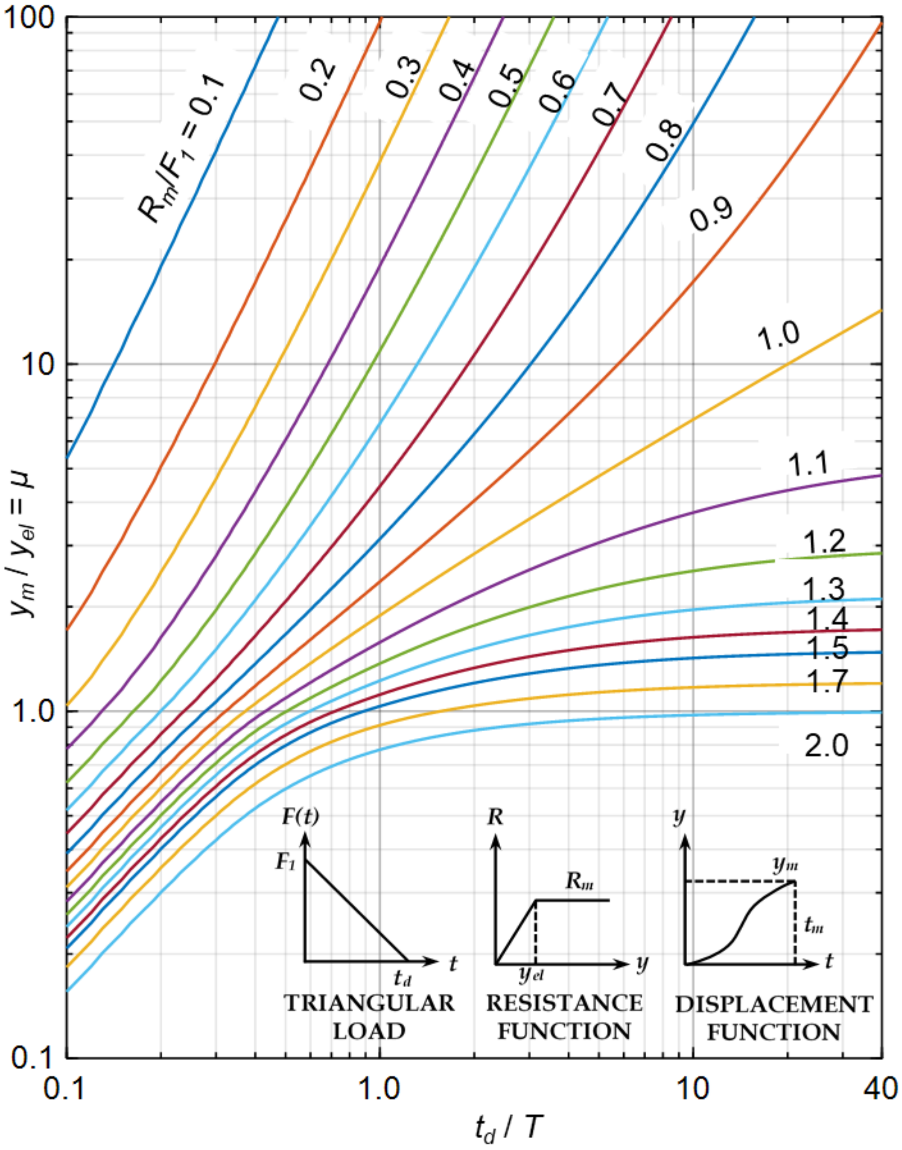


FIG. 48 PEAK DUCTILITY DEMAND OF SDOF SYSTEM SUBJECT TO TRIANGULAR PULSE LOADING

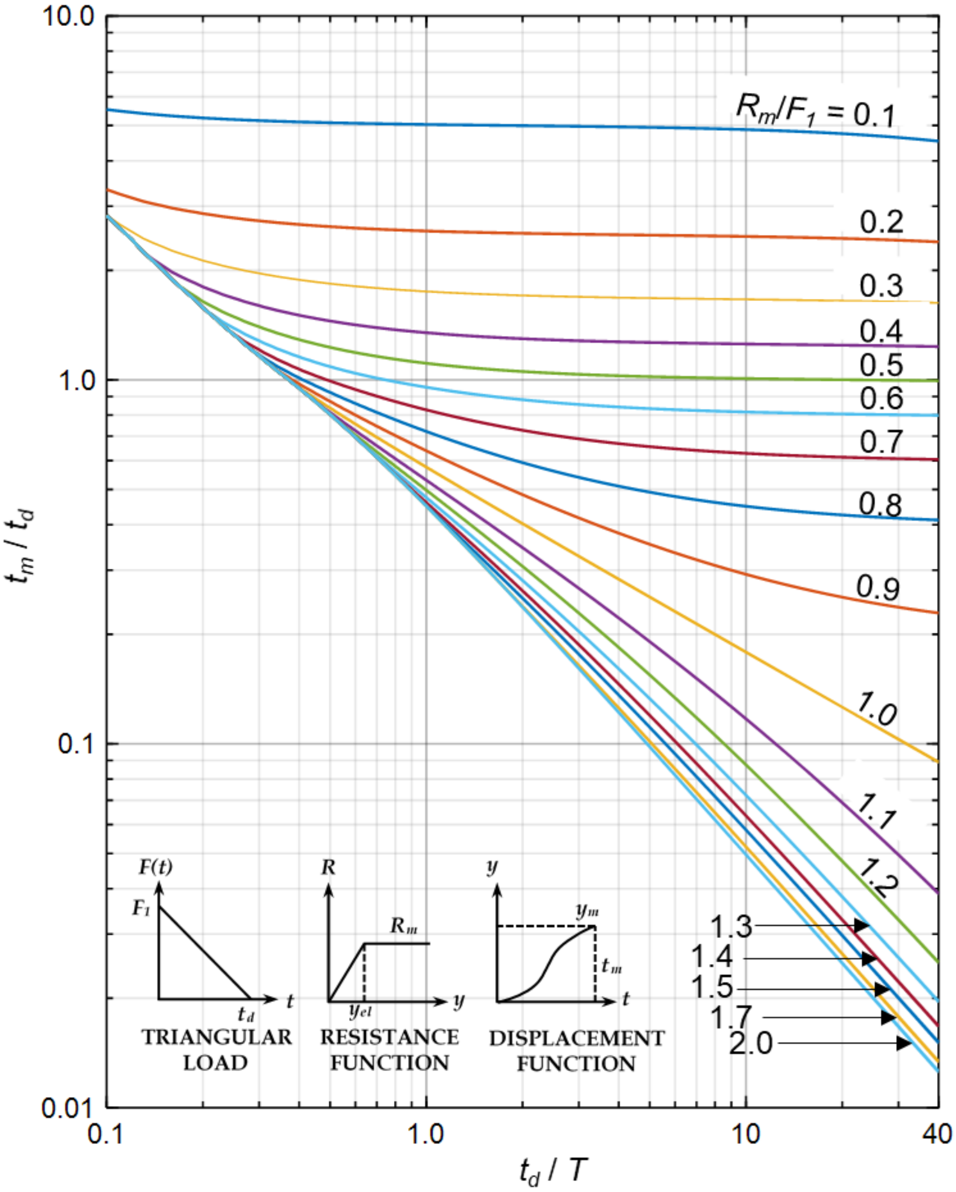


FIG. 49 TIME TO PEAK DUCTILITY DEMAND OF SDOF SYSTEM SUBJECT TO TRIANGULAR PULSE

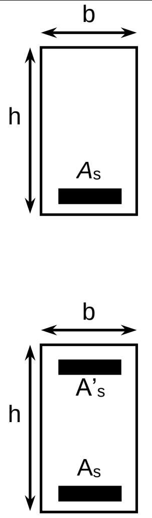
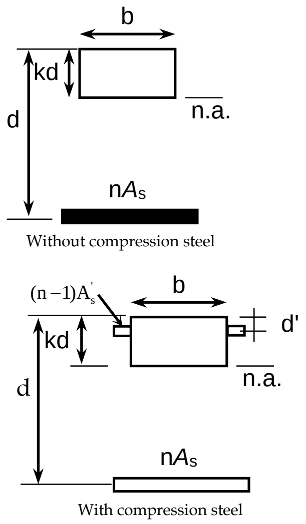
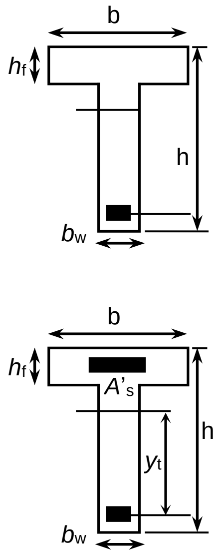
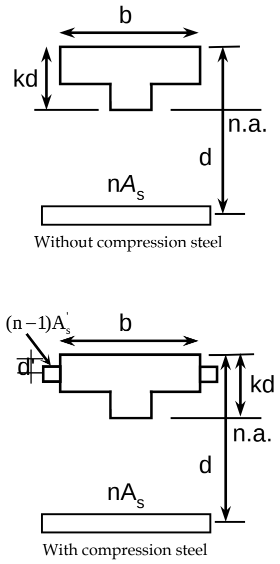
<table>
<tr><th>Gross Section</th><th>Cracked Transformed Section</th><th>Gross and Cracked Moment of Inertia</th></tr>
<tr>
<td>
</td>
<td>
</td>
<td>$n = \frac{E_s}{E_c}$
$B = \frac{b}{(nA_s)}$
$I_g = \frac{bh^3}{12}$
Without compression steel:
$kd = (\sqrt{2dB + 1} - 1)/B$
$I_{cr} = bk^3d^3/3 + nA_s(d - kd)^2$
With compression steel:
$r = (n - 1)A'_s/(nA_s)$
$kd = \left[\sqrt{2dB(1+rd'/d) + (1 + r)^2} - (1 + r)\right]/B$
$I_{cr} = bk^3d^3/3 + nA_s(d - kd)^2 + (n - 1)A'_s(kd - d')^2$</td>
</tr>
<tr>
<td>
</td>
<td>
</td>
<td>$n = \frac{E_s}{E_c}$
$C = b_w/(nA_s)$, $f = h_f(b - b_w)/(nA_s)$
$y_t = h - 1/2\left[(b - b_w)h_f^2 + b_wh^2\right]/[(b - b_w)h_f + b_wh]$,
$I_g = (b - b_w)h_f^3/12 + b_wh^3/12 + (b - b_w)h_f(h - h_f/2 - y_t)^2 + b_wh(y_t - h/2)^2$
Without compression steel:
$kd = \left[\sqrt{C(2d + h_ff)+(1 + f)^2} - (1 + f)\right]/C$
$I_{cr} = (b - b_w)h_f^3/12 + b_wk^3d^3/3 + (b - b_w)h_f(kd - h_f/2)^2 + nA_s(d - kd)^2+(n - 1)A_s(kd - d')2$
With compression steel:
$kd = \left[\sqrt{C(2d + h_ff + 2rd')+(f + r + 1)^2} - (f + r + 1)\right]/C$
$I_{cr} = (b - b_w)h_f^3/12 + b_wk^3d^3/3 + (b - b_w)h_f(kd - h_f/2)^2 + nA_s(d - kd)^2 + (n - 1)A'_s(kd - d')2$</td>
</tr>
</table>

FIG. 50 CRACKED MOMENT OF INERTIA OF PRISMATIC SECTIONS

Q-8.3.2 *MDOF Analysis*

The multi degree of freedom (MDOF) analysis is permitted for structural systems which cannot be analysed using the simplified SDOF analysis, provided the expected failure mechanism under blast load can adequately be represented using the MDOF model representation.

Q-8.3.2.1 The numerical model of MDOF system may comprise of distributed elements or combined lumped mass and stiffness elements.

Q-8.3.2.2 The spring in the lumped-mass stiffness should adequately represent the member resistance-deflection behaviour by incorporating appropriate constitutive relationship.

Q-8.3.2.3 The lumped-mass stiffness system should be able to properly account for the dynamic behaviour and failure modes at the structure and the element level.

Q-8.3.2.4 The use of structural damping is permitted for MDOF system when the expected response is within elastic range. These structural damping values for elastic response shall conform to the typical values used for different type of structures.

Q-8.3.2.5 The energy dissipation during inelastic response should be accounted by using appropriate nonlinear member resistance-deflection curves. Supplemental damping in form of structural damping is not permitted for nonlinear range of response.

Q-8.3.2.6 The time-step for MDOF analysis shall be determined based on accuracy and convergence requirement. The time-step should be sufficiently small such that it captures the rise and decay of the applied blast load time history.

Q-8.3.3 *Finite Element Analysis*

Advanced numerical analysis techniques, such as explicit dynamic finite element analysis (FEA), may be used to analyse structures subject to blast loading, if the methods described in **Q-8.3.1** and **Q-8.3.2** cannot be reliably used to obtain the response. Such methods shall be required if one or more of the following conditions are encountered:

a) Detonation involves scaled distances smaller than 0.4 $m/kg^{1/3}$;
b) Detonation of non-spherical charge masses at scaled distances smaller than 3 $m/kg^{1/3}$;
c) The blast wave propagation involves interaction with surrounding environment before it reaches the location of interest;
d) The variation in geometric and material properties of structural elements or the structure cannot appropriately be represented using the simplified methods; and
e) The interaction between the blast wave and structure is of significance and the assumption of rigid reflecting surface is not valid.

The FEA shall consider the modelling of non-ductile failure modes and failure surfaces that cannot be considered in the simplified SDOF and the MDOF methods of analysis.

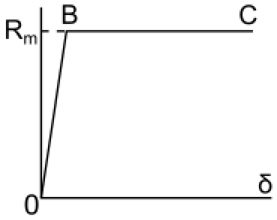
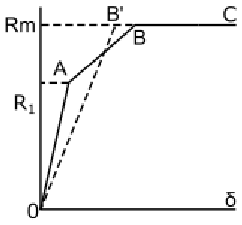
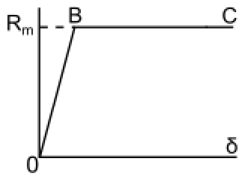
The time-step of analysis need to be decided based on size of the smallest element of interest with appropriate consideration to the accuracy and convergence of the response.

The interaction of blast load with the surrounding shall be considered using computational fluid dynamics (CFD) methods.

Where the shape and size of the charge mass is expected to play a major role in the detonation process and the resulting blast load, the detonation process itself may be modelled.

The FEA methods shall not be subjected to the response limits typically used for structural members in the simplified SDOF and the MDOF analysis.

Table 70 Transformation Factors for Beams and One-way Slabs
(*Clauses* Q-8.2.2.1, Q-8.3.1.5 *and* Q-8.3.1.6)

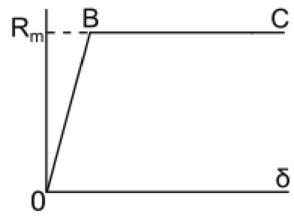
Resistance Curve, End-Conditions of Beams or One-way Slabs	**Dynamic Loading**	**Strain Range**	**Load-Mass Factor** K_{LM}		**Maximum Resistance** R_m	**Effective Spring Constant** k_E	**Dynamic Reaction**
			Concentrated Mass	Uniform Mass			
	Uniform $P_t = pL$	Elastic	–	0.78	$8M_p/L$	$384EI/5L^3$	$0.39\,R_m + 0.11\,P_t$
		Plastic	–	0.66	$8M_p/L$	–	$0.38\,R_m + 0.12\,P_t$
	Concentrated at mid-span $P_t = P$	Elastic	1.0	0.49	$4M_p/L$	$48EI/L^3$	$0.78\,R_m - 0.28\,P_t$
		Plastic	1.0	0.33	$4M_p/L$	–	$0.75\,R_m - 0.25\,P_t$
	Concentrated at third points P/2 each $P_t = P$	Elastic	0.87	0.60	$6M_p/L$	$56.4EI/L^3$	$0.62\,R_m - 0.12\,P_t$
		Plastic	1.0	0.56	$6M_p/L$	–	$0.52\,R_m - 0.02\,P_t$
 Fixed at both ends	Uniform $P_t = pL$	Elastic (OA)	–	0.77	$12M_{ps}/L$	$384EI/L^3$	$0.36\,R_1 + 0.14\,P_t$
		Elastic (OB')	–	0.78	$(8/L)(M_{ps} + M_{pm})$	$307EI/L^3$	$0.39\,R_m + 0.11\,P_t$
		Plastic	–	0.66	$(8/L)(M_{ps} + M_{pm})$	–	$0.38\,R_m + 0.12\,P_t$
	Concentrated at mid-span $P_t = P$	Elastic (OB)	1.0	0.37	$(4/L)(M_{ps} + M_{pm})$	$192EI/L^3$	$0.71\,R_m - 0.21\,P_t$
		Plastic	1.0	0.33	$(4/L)(M_{ps} + M_{pm})$	–	$0.75\,R_m - 0.25\,P_t$
Fixed at one end and simply supported at the other	Uniform $P_t = pL$	Elastic (OA)	–	0.78	$(8/L)(M_{ps})$	$185EI/L^3$	$V_1 = 0.26R_1 + 0.12P_t$ $V_2 = 0.43R_1 + 0.19P_t$
		Elastic (OB')	–	0.78	$(4/L)(M_{ps} +$	$160EI/L^3$	$0.39\,R_m + 0.11$
		Plastic	–	0.66	$(4/L)(M_{ps} +$	–	$0.38\,R_m + 0.12$

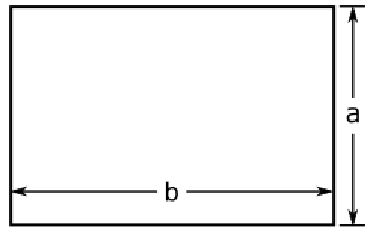
	Concentrated at mid-span $P_t = P$	Elastic (OA)	1.0	0.43	$(16/3L)M_{ps}$	$107EI/L^3$	$V_1 = 0.54R_1 + 0.14P_t$ $V_2 = 0.25R_1 + 0.07P_t$
		Elastic (OB')	1.0	0.49	$(2/L)(M_{ps} + 2M_{pm})$	$106EI/L^3$	$0.78\ R_m - 0.28\ P_t \pm M_{ps}/L$
		Plastic	1.0	0.33	$(2/L)(M_{ps} + 2M_{pm})$	–	$0.75\ R_m - 0.25\ P_t \pm M_{ps}/L$

L	=	span;
p	=	dynamic pressure;
P_t	=	dynamic load;
M_p	=	plastic moment of section;
M_{pm}	=	M_p at mid-span
M_{ps}	=	M_p at support;
R_m	=	maximum resistance; and
k_E	=	effective spring constant.

Table 71 Transformation Factors for Two-way Slabs — Four Sides, Simply Supported, Uniformly Loaded

(*Clauses* Q-8.2.2.1, Q-8.3.1.5 *and* Q-8.3.1.6)

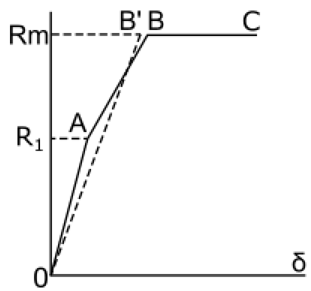


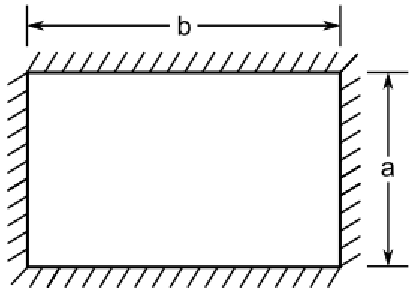


Strain Range	a/b	Load Mass Factor	Max Resistance	Spring Constant	Dynamic Reactions	
		K_{LM}	R_m	k_E	V_A	V_B
Elastic (OB)	1.0	0.68	$12(M_{pfa} + M_{pfb})/a$	$252EI/a^2$	$0.07P_r + 0.18R_m$	$0.07P_r + 0.18R_m$
	0.9	0.70	$(12M_{pfa} + 11M_{pfb})/a$	$230EI/a^2$	$0.06P_r + 0.16R_m$	$0.08P_r + 0.20R_m$
	0.8	0.71	$(12M_{pfa} + 10.3M_{pfb})/a$	$212EI/a^2$	$0.06P_r + 0.14R_m$	$0.08P_r + 0.22R_m$
	0.7	0.73	$(12M_{pfa}+9.8M_{pfb})/a$	$201EI/a^2$	$0.05P_r + 0.13R_m$	$0.08P_r + 0.24R_m$
	0.6	0.74	$(12M_{pfa} + 9.3M_{pfb})/a$	$197EI/a^2$	$0.04P_r + 0.11R_m$	$0.09P_r + 0.26R_m$
	0.5	0.75	$(12M_{pfa} + 9.0M_{pfb})/a$	$201EI/a^2$	$0.04P_r + 0.09R_m$	$0.09P_r + 0.28R_m$
Plastic (BC)	1.0	0.51	$12(M_{pfa} + M_{pfb})/a$	-	$0.09P_r + 0.16R_m$	$0.09P_r + 0.16R_m$
	0.9	0.51	$(12M_{pfa} + 11M_{pfb})/a$	-	$0.08P_r + 0.15R_m$	$0.09P_r + 0.18R_m$
	0.8	0.54	$(12M_{pfa} + 10.3M_{pfb})/a$	-	$0.07P_r + 0.13R_m$	$0.10P_r + 0.20R_m$
	0.7	0.58	$(12M_{pfa} + 9.8M_{pfb})/a$	-	$0.06P_r + 0.12R_m$	$0.10P_r + 0.22R_m$
	0.6	0.58	$(12M_{pfa} + 9.3M_{pfb})/a$	-	$0.05P_r + 0.10R_m$	$0.10P_r + 0.25R_m$
	0.5	0.59	$(12M_{pfa} + 9.0M_{pfb})/a$	-	$0.04P_r + 0. 08R_m$	$0.11P_r + 0.27R_m$

M_{pfa} = total positive ultimate moment capacity along mid-span section parallel to short edge;
M_{pfb} = total positive ultimate moment capacity along mid-span section parallel to long edge;
I = moment of inertia per unit width of slab;
P_t = dynamic load;
V_A = total dynamic reaction along a short edge; and
V_B = total dynamic reaction along a long edge.

Table 72 Transformation Factors for Two-way Slabs — Fixed four sides, Uniform loaded
(*Clauses* Q-8.2.2.1, Q-8.3.1.5 *and* Q-8.3.1.6)





Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Strain Range	Spring Constant k_E	Dynamic Reactions	
						V_A	V_B
Elastic (OA)	1.0	0.63	29.2 M^o_{psb}	810 EI/a^2	$0.10P_t + 0.15R_1$	$0.10P_t + 0.15R_1$	1.0
	0.9	0.68	27.4 M^o_{psb}	742 EI/a^2	$0.09P_t + 0.14R_1$	$0.10P_t + 0.17R_1$	0.9
	0.8	0.69	26.4 M^o_{psb}	705 EI/a^2	$0.08P_t + 0.12R_1$	$0.11P_t + 0.19R_1$	0.8
	0.7	0.71	26.2 M^o_{psb}	692EI/a^2	$0.07P_t + 0.11R_1$	$0.11P_t + 0.21R_1$	0.7
	0.6	0.71	27.3 M^o_{psb}	724 EI/a^2	$0.06P_t + 0.09R_1$	$0.12P_t + 0.23R_1$	0.6
	0.5	0.72	30.2 M^o_{psb}	806 EI/a^2	$0.05P_t + 0.08R_1$	$0.12P_t + 0.25R_1$	0.5
Elasto-plastic (OB')	1.0	0.67	$(1/a)[12(M_{pfa}+ M_{psa})+ 12(M_{pfb}+ M_{psb})]$	252 EI/a^2	$0.07P_t + 0.18R_m$	$0.07P_t + 0.18R_m$	1.0
	0.9	0.70	$(1/a)[12(M_{pfa}+ M_{psa})+ 11(M_{pfb}+ M_{psb})]$	230 EI/a^2	$0.06P_t + 0.16R_m$	$0.08P_t + 0.20R_m$	0.9
	0.8	0.71	$(1/a)[12(M_{pfa}+ M_{psa})+ 10.3(M_{pfb}+ M_{psb})]$	212 EI/a^2	$0.06P_t + 0.14R_m$	$0.08P_t + 0.22R_m$	0.8
	0.7	0.73	$(1/a)[12(M_{pfa}+ M_{psa})+ 9.8(M_{pfb}+ M_{psb})]$	201 EI/a^2	$0.05P_t + 0.13R_m$	$0.08P_t + 0.24R_m$	0.7
	0.6	0.74	$(1/a)[12(M_{pfa}+ M_{psa})+ 9.3(M_{pfb}+ M_{psb})]$	197 EI/a^2	$0.04P_t + 0.11R_m$	$0.09P_t + 0.26R_m$	0.6
	0.5	0.75	$(1/a)[12(M_{pfa}+ M_{psa})+ 9.0(M_{pfb}+ M_{psb})]$	201 EI/a^2	$0.04P_t + 0.09R_m$	$0.09P_t + 0.28R_m$	0.5
Fully plastic (BC)	1.0	0.51	$(1/a)[12(M_{pfa}+ M_{psa})+ 12(M_{pfb}+ M_{psb})]$	–	$0.09P_t + 0.16R_m$	$0.09P_t + 0.16R_m$	1.0
	0.9	0.51	$(1/a)[12(M_{pfa}+ M_{psa})+ 11(M_{pfb}+ M_{psb})]$	–	$0.08P_t + 0.15R_m$	$0.09P_t + 0.18R_m$	0.9
	0.8	0.54	$(1/a)[12(M_{pfa}+ M_{psa})+ 10.3(M_{pfb}+ M_{psb})]$	–	$0.07P_t + 0.13R_m$	$0.10P_t + 0.20R_m$	0.8
	0.7	0.58	$(1/a)[12(M_{pfa}+ M_{psa})+ 9.8(M_{pfb}+ M_{psb})]$	–	$0.06P_t + 0.12R_m$	$0.10P_t + 0.22R_m$	0.7
	0.6	0.58	$(1/a)[12(M_{pfa}+ M_{psa})+ 9.3(M_{pfb}+ M_{psb})]$	–	$0.05P_t + 0.10R_m$	$0.10P_t + 0.25R_m$	0.6
	0.5	0.59	$(1/a)[12(M_{pfa}+ M_{psa})+ 9.0(M_{pfb}+ M_{psb})]$	–	$0.04P_t + 0.08R_m$	$0.11P_t + 0.27R_m$	0.5

Range (OA)	=	moment at centre of long edge just becomes plastic;
Range (OB')	=	moment at supports and mid-span sections just becomes plastic;
M^{o}_{psb}	=	negative ultimate moment capacity per unit width at centre of long edge;
M_{psa}	=	total negative ultimate moment capacity along a short edge support;
M_{psb}	=	total negative ultimate moment capacity along a long edge support;
M_{pfa}, M_{pfb}, I, P_t	=	(*see* Table 5);
V_A	=	total dynamic reaction along a short edge; and
V_B	=	total dynamic reaction along a long edge.

Q-9 LOAD COMBINATIONS FOR DESIGN

Q-9.1 Partial Safety Factor

A partial safety factor of unity shall be assumed for blast loads.

Q-9.2 Live Loads

Live load on floors shall be considered as per good practice [6-1(33)]. No live load shall be considered on roof at the time of blast.

Q-9.3 Load Combination

Wind or earthquake forces shall not be assumed to occur simultaneously with blast effects. Effects of temperature and shrinkage shall be neglected.

Following load combinations for limit state design shall be considered:

$1.0DL + 0.5\,IL + 1.0B$
$0.9DL + 1.0B$

where

DL = design dead load;
IL = design imposed live load; and
B = design blast load.

Q-10 For design of structures for blast effects of explosions above ground, *see also* good practice [6-1(30)].

For safety of structures during underground blasting, *see also* good practice [6-1(31)].

ANNEX R
(*Clause* 9.2)

SUMMARY OF DISTRICTS HAVING SUBSTANTIAL MULTI-HAZARD RISK AREAS

State	*Name of Districts Having Substantial Multi-hazard Prone Area*			
	E.Q. and Flood	Cyclone and Flood	E.Q., Cyclone and Flood	E.Q. and Cyclone
(1)	(2)	(3)	(4)	(5)
Andhra Pradesh	Adilabad, Karim Nagar, Khammam	Krishna, Nellore, Srikakulam, Visakhapatnam, Vizianagram	East Godavari, Guntur, Prakasam, West Godavari	-
Assam	All 22 districts listed in Table 53 could have M.S.K IX or more with flooding	No cyclone, but speed can be 50 m/s in districts of Table 53 causing local damage except Dhubri	-	-
Bihar	All 25 districts listed in Table 53	-	-	-
Goa	-	-	-	North and South Goa
Gujarat	Banaskantha, Danthe GS, Gandhinagar, Kheda, Mahesana, Panchmahals, Vadodara	-	Ahmedabad, Bharuch, Surat, Valsad	Amreli, Bhavnagar, Jamnagar, Rajkot, Junagad, Kachcha
Haryana	All 8 districts listed in Table 53	-	-	-
Kerala	Idduki, Kottayam, Palakkad, Pathanamthitta	-	Alappuzha, Ernakulum, Kannur, Kasargod, Kollam, Kozhikode,	-

State	*Name of Districts Having Substantial Multi-hazard Prone Area*			
	E.Q. and Flood	Cyclone and Flood	E.Q., Cyclone and Flood	E.Q. and Cyclone
(1)	(2)	(3)	(4)	(5)
			Malappuram, Thiruvanathapuram, Thrissur	
Maharashtra	-	-	-	Mumbai, Rayagad, Ratnagiri, Sindhudurg, Thane
Orissa	-	Ganjam	Baleshwar, Cuttack, Puri	Dhenkanal
Punjab	All 12 districts listed in Table 53	-	-	-
Uttar Pradesh and Uttarakhand	All 50 districts listed in Table 53	-	-	-
West Bengal	Birbhum, Darjeeling, Jalpaiguri, Cooch Behar, Malda, Murshidabad, West Dinajpur	-	Bardhaman, Kolkata, Hugli, Howrah, Midnapore, Nadia, North and South 24 Parganas	Bankura
Union Territories	Delhi	-	Yanam (Py)	Diu
India	**139 Districts**	**6 Districts**	**29 Districts**	**16 Districts**

Table 53 Multi-Hazard Prone Districts
(*Annex* R)

Assam

Barpeta, Bongaigaon, Cachar[1)], Darrang, Dhemaji, Dhuburi, Dibrugarh, Goalpara, Golaghat, Hailaknadi[1)], Jorhat, Kamrup, Karbianglong, Karimganj[1)], Kokrajhar, Lakhimpur, Morigaon, Nagaon, Nalbari, Sibsagar, Sonitpur, Tinsukia

Bihar[2)]
Araria, Begusarai, Bhagalpur, Bhojpur, Darbhanga, Gopalganj, Katihar, Khagaria, Kishanganj, Madhepura, Madhubani, Munger, Muzaffarpur, Nalanda, Nawada, Paschim Champaran, Patna, Purbachamparan, Purnia, Samastipur, Saran, Saharsa, Sitamarhi, Siwan, Vaishali

Haryana[3)]
Ambala, Bhiwani, Faridabad, Gurgaon, Hisar, Jind, Kurukshetra, Rohtak

Punjab[4)]
Amritsar, Bathinda, Faridkot, Firozpur, Gurdaspur, Hoshiarpur, Jalandhar, Kapurthala, Ludhiana, Patiala, Rupnagar, Sangrur

Uttar Pradesh and Uttarakhand[5)]
Agra, Aligarh, Allahabad, Azamgarh, Bahraich, Ballia, Barabanki, Bareilly, Basti, Bijnor, Budaun, Bulandshahr, Deoria, Etah, Etawah, Faizabad, Farrukhabad, Fatehpur, Firozabad, Ghaziabad, Ghazipur, Gonda, Gorakhpur, Hardoi, Haridwar, Jaunpur, Kanpur (Dehat), Kanpur (Nagar), Kheri, Lucknow, Maharajganj, Mainpuri, Mathura, Mau, Meerut, Mirzapur, Mordabad, Muzaffarnagar, Nainital, Pilibhit, Partapgarh, Raebareli, Rampur, Saharanpur, Shahjahanpur, Siddarth Nagar, Sitapur, Sultanpur, Unnao, Varanasi

[1)] Districts liable to cyclonic storm but no storm surge
[2)] No cyclonic storm in Bihar
[3)] No cyclonic storm in Haryana
[4)] No cyclonic storm in Punjab
[5)] No cyclonic storm in Uttar Pradesh and Uttarakhand

LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfillment of the requirements of the Code. The latest version of a standard shall be adopted at the time of enforcement of the Code. The standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Part/Section.

	IS No.	*Title*
(1)	875	Code of practice for design loads (other than Earthquake) for buildings and structures
	(Part 1) : 1987	Part 1 Dead loads-unit weights of building material and stored materials (*second revision*)
(2)	8888 : 2020	Requirements of low income housing for Urban areas guide (*second revision*)
(3)	807 : 2006	Code of practice for design, erection and testing (structural portion) of cranes and hoists (*second revision*)
	3177 : 2020	Code of practice for electric overhead traveling cranes and gantry cranes other than steelwork cranes (*third revision*)
(4)	14732 : 2000	Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0.063 to 1 Hz)
(5)	15498 : 2023	Guidelines for improving the cyclone resistance of low rise houses and other buildings/structures
(6)	4326 : 2013	Earthquake resistant design and construction of buildings – Code of Practice (*third revision*)
	13827 : 1993	Improving earthquake resistance of earthen buildings – Guidelines
	13828 : 1993	Improving earthquake resistance of low strength masonry buildings –Guidelines
	13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces – Code of practice (*first revision*)
	13935 : 2009	Repair and seismic strengthening of buildings – Guidelines

	15988 : 2013	Seismic evaluation and strengthening of existing reinforced concrete building – Guidelines
(7)	456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
	800 : 2007	Code of practice for general construction in steel (second revision)
(8)	13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces – Code of practice (*first revision*)
(9)	800 : 2007	Code of practice for general construction in steel (second revision)
(10)	456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
(11)	456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
	1343 : 2012	Code of practice for prestressed concrete (*second revision*)
(12)	IS 1893 (Part 4) : 2015	Criteria for earthquake resistant design of structures: Industrial structures including stack-like structures (*first revision*)
(13)	456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
	800 : 2007	Code of practice for general construction in steel (second revision)
	1343 : 2012	Code of practice for prestressed concrete (*second revision*)
	1905 : 1987	Code of practice for structural use of unreinforced masonry (*third revision*)
	2974	Code of practice for design and construction of machine foundations:
	(Part 1) : 1982	Foundation for reciprocating type machines
	(Part 2) : 1980	Foundations for impact type machines (Hammer foundations)
	(Part 3) : 1992	Foundations for rotary type machines (Medium and high frequency)
	(Part 4) : 1979	Foundations for rotary type machines of low frequency
	(Part 5) : 1987	Foundations for impact machines other than hammer (Forging and stamping press, pig breaker, drop crusher and jolter)
(14)	2974	Code of practice for design and construction of machine foundations:
	(Part 1) : 1982	Foundation for reciprocating type machines
	(Part 2) : 1980	Foundations for impact type machines (Hammer foundations)
	(Part 3) : 1992	Foundations for rotary type machines (Medium and high frequency)

	(Part 4) : 1979	Foundations for rotary type machines of low frequency
	(Part 5) : 1987	Foundations for impact machines other than hammer (Forging and stamping press, pig breaker, drop crusher and jolter)
(15)	6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
(16)	1888 : 1982	Method of load test on soils (*second revision*)
(17)	1498 : 1970	Classification and identification of soils for general engineering purposes (*first revision*)
	2131 : 1981	Method of standard penetration test for soils (*first revision*)
(18)	1893	Criteria for earthquake resistant design of structures:
	(Part 1) : 2016	General Provisions and Buildings
	(Part 2) : 2014	Liquid retaining tanks
	(Part 3) : 2014	Bridges and retaining walls
	(Part 4) : 2015	Industrial structures including stack-like structures (*first revision*)
(19)	1905 : 1987	Code of practice for structural use of unreinforced masonry (*third revision*)
(20)	4326 : 2013	Earthquake resistant design and construction of buildings – Code of Practice (*third revision*)
(21)	3414 : 1968	Code of practice for design and installation of joints in buildings
(22)	875 (Part 5) : 1987	Code of practice for design loads (other than Earthquake) for buildings and structures Part 5 Special loads and load combinations *(second revision)*
(23)	1642 : 2013	Code of practice fire safety of buildings (general): Details of construction (*second revision*)
(24)	14458 (Part 1) : 1998	Guidelines for retaining wall for hill area: Part 1 Selection of type of wall
(25)	14458 (Part 2) : 1997	Guidelines for retaining wall for hill area: Part 2 Design of retaining/breast walls

(26)	14458 (Part 3) : 1998	Guidelines for retaining wall for hill area: Part 3 Construction of dry stone
(27)	14496 (Part 2) : 1998	Guidelines for preparation of landslide-Hazard zonation maps in mountainous terrains: Part 2 Macro-zonation
(28)	14680 : 1999	Guidelines for landslide control
(29)	2131 : 1981	Method of standard penetration test for soils (*first revision*)
(30)	4991 : 1968	Criteria for blast resistant design of structures for explosions above ground
(31)	6992 : 1973	Criteria for safety and design of structures subject to underground blasts
(32)	18315 : 2023	Estimation of Cyclonic Factor (k_4) – Guidelines
(33)	875 (Part 2) : 1987	Code of practice for design loads (Other Than Earthquake) for buildings and structures: Part 2 Imposed Loads (*second revision*)
(34)	17951 : 2023	Method for determination of performance relating to safety of external building envelope impacted by wind borne debris