भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110002
Phone: + 91 11 23230131, 23233375, 23239402 Extn 4402; Fax: + 91 11 23235529

व्यापक परिचालन मसौदा

हमारा संदर्भ : सीईडी 46/टी-7 **28 अगस्त 2015**
तकनीकी समिति :राष्ट्रीय भवन निर्माण संहिता विषय समिति , सीईडी 46

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प्राप्तकर्ता :

1 **सिविल इंजीनियरी विभाग परिषद् के सभी सदस्य**
2 **राष्ट्रीय भवन निर्माण संहिता विषय समिति, सीईडी 46 व मृदा एवं नींव के लिए पैनल, सीईडी 46:P5 के सभी सदस्य**
3 **रूचि रखने वाले अन्य निकाय ।**

महोदय/महोदया,

निम्नलिखित मसौदा संलग्न है:

प्रलेख संख्या	शीर्षक
सीईडी 46(8021)WC	**भारत की राष्ट्रीय भवन निर्माण संहिता का मसौदा : भाग 6 संरचनात्मक डिजाइन, अनुभाग 2 मृदा एवं नींव [SP7(भाग 6/अनुभाग 2) का तीसरा पुनरीक्षण]**

कृपया इस मसौदे का अवलोकन करें और अपनी सम्मतियाँ यह बताते हुए भेजें कि यदि यह मसौदा भारत की राष्ट्रीय भवन निर्माण संहिता के भाग के रूप में प्रकाशित हो तो इस पर अमल करने में आपके व्यवसाय अथवा कारोबार में क्या कठिनाइयाँ आ सकती हैं ।

सम्मतियाँ भेजने की अंतिम तिथि : **28 सितंबर 2015**।

यदि कोई सम्मति हो तो कृपया अधोहस्ताक्षरी को उपरिलिखित पते पर संलग्न फोर्मेट में भेजें । हो सके तो कृपया अपनी सम्मतियाँ ई-मेल द्वारा **sanjaypant@bis.org.in** पर भेजें ।

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 ।

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धन्यवाद ।

भवदीय,
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(बी.के. सिन्हा)
प्रमुख (सिविल इंजीनियरी)

संलग्न: उपरिलिखित

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110002
Phone: + 91 11 23230131, 23233375, 23239402 Extn 4402; Fax: + 91 11 23235529

DRAFT IN WIDE CIRCULATION

<u>DOCUMENT DESPATCH ADVICE</u>

Reference	Date
CED 46/T-7	**28 August 2015**

TECHNICAL COMMITTEE:
NATIONAL BUILDING CODE SECTIONAL COMMITTEE, CED 46

ADDRESSED TO:

1. All Members of Civil Engineering Division Council, CEDC
2. All Members of National Building Code Sectional Committee, CED 46 and its Panel for Soils and Foundations, CED 46:P5
3. All other interests.

Dear Sir/Madam,

Please find enclosed the following draft:

Doc. No.	Title
CED 46 (8021)WC	**Draft National Building Code of India: Part 6 Structural Design, Section 2 Soils and Foundations [*Third Revision of* SP 7(Part 6/Section 2)]**

Kindly examine the 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 part of the National Building Code of India.

Last Date for comments: **<u>28 September 2015</u> .**

Comments if any, may please be made in the format as attached, and mailed to the undersigned at the above address. You are requested to send your comments preferably through e-mail to **sanjaypant@bis.org.in**.

In case no comments are received or comments received are of editorial nature, you may kindly permit us to presume your approval for the above document as finalized. However, in case of comments of technical 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.

This document is also hosted on BIS website **www.bis.org.in**.

Thanking you,

Yours faithfully,

Sd/-

(B. K. Sinha)
Head (Civil Engg)

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 in MS WORD format to sanjaypant@bis.org.in shall be appreciated**.]

Doc. No.: CED 46(8021)WC **BIS Letter Ref**: CED 46/T-7 **Dated**: 28 August 2015

Title: Draft National Building Code of India: Part 6 Structural Design,
Section 2 Soils and Foundations
[*Third Revision of* SP 7(Part 6/Section 2)]

Name of the Commentator/ Organization: ______________________________________

Clause No. with Para No. or Table No. or Figure No. commented *(as applicable)*	**Comments / Modified Wordings**	**Justification of Proposed Change**

Doc: CED 46 (8021)WC
August 2015

Draft NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 2 Soils and Foundations

[*Third Revision of* SP 7(Part 6/Section 2)]

BUREAU OF INDIAN STANDARDS

C O N T E N T S

IMPORTANT EXPLANTORY NOTE FOR USERS OF THE CODE

In this Part/Section of the 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 this Part/Section may 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 this 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 this Section, the number appearing in the first column indicates the number of the reference in this Section. For example:

a) Accepted standard [6-2(12)] refers to the Indian Standard given at serial number (12) of the above list given at the end of this Section 2 of Part 6, that is, IS 1888:1982 ‘Method of load tests on soils (*second revision*)’ and the other standards listed therein.

b) Good practice [6-2(3)] refers to the Indian Standard given at serial number (2) of the above list given at the end of this Section 2 of Part 6, that is, IS 10042:1981 'Code of practice for site-investigations for foundation in gravel boulder deposits'.

BUREAU OF INDIAN STANDARDS

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Draft NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 2 Soils and Foundations

[*Third Revision of* SP 7(Part 6/Section 2)]

ICS: 01.120; 91.040.01

National Building Code Sectional Committee, CED 46	Last Date for Comments: **28 September 2015**

National Building Code Sectional Committee, CED 46

FOREWORD

This Section deals with the structural design aspects of foundations and mainly covers the design principles involved in different types of foundations.

This Section was published in 1970, and subsequently revised in 1983 and 2005. In the first revision design considerations in respect of shallow foundation were modified, provisions regarding pier foundation were added and provisions regarding raft foundation and pile foundation were revised and elaborated. In the second revision of 2005, design considerations in respect of shallow foundations were again modified, method for determining depth of fixity, lateral deflection and maximum moment of laterally loaded piles were modified and reference was made to ground improvement techniques.

As a result of experience gained in implementation of 2005 version of the Code and feed back received as well as revision of standards in the field of soils and foundations, a need to revise this Section was felt. This revision has therefore been prepared to take into account these developments. The significant changes incorporated in this revision include:

a) The scope of this Section has been extended to cover design of foundations on rock.

b) Definitions of various terms have been modified as per the prevailing engineering practice and new terms have been added, particularly those relating to pile foundation and ground improvement.

c) The clause on site investigation has been completely reviewed and number of modifications have been included, such as, new methods of soil investigation have been added; depth of exploration for pile foundations has been added; new sub-clauses on vertical interval for field tests and site investigation report have been added; etc.

d) Method for assessment of liquefaction potential of a site has been included.

e) Permissible differential settlements and tilt (angular distortion) for shallow foundations in soils as given in Table 4 has been modified.

f) Procedures for calculation of bearing capacity, structural capacity, factor of safety, lateral load capacity, overloading, etc, in case of pile foundations have also been modified to bring them at par with the present practices.

g) Design parameters with respect to adhesion factor, earth pressure coefficient, modulus of subgrade reaction, etc, for design of pile foundations have also been revised to make them consistence with the outcome of modern research and construction practices.

j) Provision has been made for use of any established dynamic pile driving formulae, instead of recommending any specific formula, to control the pile driving at site, giving due consideration to limitations of various formulae.

k) Other modifications have also been incorporated in provisions relating to pile foundations to bring them in coherence with the revised Indian Standards on pile foundations.

l) A reference to spun piles, which are used in deep marshy soils where conventional pile installation beyond 50 m is difficult, has been included. However, much detail is not available in the country for the purpose of codal provisions which may be shared by the users of spun pipes in the country based on their experience in use of these piles, for formulation of guidelines thereon.

k) The clause on ground improvement techniques has been elaborated and a table on summary of soil improvement methods has been added.

For detailed information regarding structural analysis and soil mechanics aspects of individual foundations, reference should be made to standard textbooks and available literature.

The information contained in this section is mainly based on the following Indian Standards:

IS 1080:1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
IS 1892:1979	Code of practice for sub-surface investigation for foundations (*first revision*)
IS 1904:1986	Code of practice for design and construction of foundations in soils: General requirements (*third revision*)
IS 2911(Part 1/Sec 1):2010	Design and construction of pile foundations — Code of practice: Part 1 Concrete piles: Section 1 Driven cast *in-situ* concrete piles (*second revision*)
IS 2911(Part 1/Sec 2):2010	Design and construction of pile foundations — Code of practice: Part 1 Concrete piles: Section 2 Bored cast *in-situ* piles (*second revision*)
IS 2911(Part 1/Sec 3):2010	Design and construction of pile foundations — Code of practice: Part 1 Concrete piles : Section 3 Driven precast concrete piles (*second revision*)
IS 2911(Part 1/Sec 4):2010	Design and construction of pile foundations — Code of practice: Part 1 Concrete piles : Section 4 Precast concrete piles in prebored holes (*first revision*)
IS 2911(Part 3):1980	Code of practice for design and construction of pile foundations: Part 3 Under-reamed piles (*first revision*)
IS 2950 (Part 1):1981	Code of practice for design and construction of raft foundations: Part 1 Design (*second revision*)

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to the 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 2:1960 `Rules for rounding off numerical values (revised)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Section.

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August 2015

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Draft NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 2 Soils and Foundations

[*Third Revision of* SP 7(Part 6/Section 2)]

ICS: 01.120; 91.040.01

National Building Code Sectional Committee, CED 46	Last Date for Comments: **28 September 2015**

1 SCOPE

1.1 This Section covers geotechnical design (principles) of all building foundations such as shallow foundations, like, continuous strip footings, combined footings, raft foundations, deep foundations like pile foundations and other foundation systems to ensure safety and serviceability without exceeding the permissible stresses of the materials of foundations and the bearing capacity of the supporting soil/rock.

1.2 It also covers provisions relating to preliminary work required for construction of foundations and protection of excavation.

2 TERMINOLOGY

2.0 For the purpose of this section, the following definitions shall apply.

2.1 General

2.1.1 *Clay* – An aggregate of microscopic and submicroscopic particles derived from the chemical decomposition and disintegration of rock constituents. It is plastic within a moderate to wide range of water content. The particles are less than 0.002 mm in size.

2.1.2 *Clay, Firm* – A clay which at its natural water content can be moulded by substantial pressure with the fingers and can be excavated with a spade.

2.1.3 *Clay, Soft* – A clay which at its natural water content can be easily moulded with the fingers and readily excavated.

2.1.4 *Clay, Stiff* – A clay which at its natural water content cannot be moulded with the fingers and requires a pick or pneumatic spade for its removal.

2.1.5 *Foundation* – That part of the structure which is in direct contact with and transmits loads to the ground.

2.1.6 *Gravel* – Cohesionless aggregates of angular rounded or semi-rounded, fragments of more or less unaltered rocks or minerals, 50 percent or more of the particles having size greater than 4.75 mm and less than 80 mm.

2.1.7 *Peat* – A fibrous mass of organic matter in various stages of decomposition generally dark brown to black in colour and of spongy consistency.

2.1.8 *Sand* – Cohesionless aggregate of rounded, sub-rounded, angular, subangular or flat fragments of more or less unaltered rock or minerals, 50 percent or more of particles greater than 0.075 mm or less than 4.75 mm in size.

2.1.9 *Sand, Coarse* – Sand which contains 50 percent or more of particles of size greater than 2 mm and less than 4.75 mm.

2.1.10 *Sand, Fine* – Sand which contains 50 percent of particles of size greater than 0.075 mm and less than 0.425 mm.

2.1.11 *Sand, Medium* – Sand which contains 50 percent of particles of size greater than 0.425 mm and less than 2.0 mm.

2.1.12 *Silt* – A fine grained soil with little or no plasticity. The size of particles ranges from 0.075 mm to 0.002 mm.

2.1.13 *Soft Rock* – A rocky cemented material which offers a high resistance to picking up with pick axes and sharp tools but which does not normally require blasting or chiselling for excavation.

2.1.14 *Soil, Black Cotton* – Inorganic clays of medium to high compressibility. They form a major soil group in India. They are predominately montmorillonitic in structure and yellowish black or blackish grey in colour. They are characterized by high shrinkage and swelling properties.

2.1.15 *Soil, Coarse Grained* – Soils which include the coarse and largely siliceous and unaltered products of rock weathering. They possess no plasticity and tend to lack cohesion when in dry state.

2.1.16 *Soil, Fine Grained* – Soils consisting of the fine and altered products of rock weathering, possessing cohesion and plasticity in their natural state, the former even when dry and both even when submerged. In these soils, more than half of the material by weight is smaller than 75-micron IS Sieve size.

2.1.17 *Total Settlement* – The total downward movement of the foundation unit under load.

2.2 Ground Improvement

2.2.1 *Ground Improvement* – Enhancement of the in-place properties of the ground by controlled application of technique suited to subsoil conditions.

2.2.2 *Injection* – Introduction of a chemical/cementitious material into a soil mass by application of pressure.

2.2.3 *Preloading* – Application of loads to achieve improvement of soil properties prior to imposition of structural loads.

2.2.4 *Soil Densification* – A technique to densify cohesionless soils by imparting shocks or vibrations.

2.2.5 *Soil Reinforcement* – Rods, strips or fabrics incorporated within soil mass to impart resistance to tensile, shear and compressive forces.

2.3 Shallow Foundation

2.3.1 *Back Fill* – Materials used or reused to fill an excavation.

2.3.2 *Bearing Capacity, Safe* – The maximum intensity of loading that the soil will safely carry with a factor of safety against shear failure of soil irrespective of any settlement that may occur.

2.3.3 *Bearing Capacity, Ultimate* – The intensity of loading at the base of a foundation which would cause shear failure of the supporting soil.

2.3.4 *Bearing Pressure, Allowable (Gross or Net)* – The intensity of loading which the foundation will carry without undergoing settlement in excess of the permissible value for the structure under consideration but not exceeding safe bearing capacity.

The net allowable bearing pressure is the gross allowable bearing pressure minus the surcharge intensity.

> NOTE - The concept of 'gross' and `net' used in defining the allowable bearing pressure could also be extended to safe bearing capacity, safe bearing pressure and ultimate bearing capacity.

2.3.5 *Factor of Safety (with Respect to Bearing Capacity)* – A factor by which the ultimate bearing capacity (net) must be reduced to arrive at the value of safe bearing capacity (net).

2.3.6 *Footing* – A structure constructed in brick work, masonry or concrete under the base of a wall or column for the purpose of distributing the load over a larger area.

2.3.7 *Foundation, Raft* – A substructure supporting an arrangement of columns or walls in a row or rows transmitting the loads to the soil by means of a continuous slab, with or without depressions or openings.

2.3.8 *Made-up Ground* – Refuse, excavated soil or rock deposited for the purpose of filling a depression or raising a site above the natural surface level of the ground.

2.3.9 *Offset* – The projection of the lower step from the vertical face of the upper step.

2.3.10 *Permanent Load* – Loads which remain on the structure for a period, or a number of periods, long enough to cause time dependent deformation/settlement of the soil.

2.3.11 *Shallow Foundation* – A foundation whose width is generally equal to or greater than its depth. The shearing resistance of the soil in the sides of the foundation is generally neglected.

2.3.12 *Spread Foundation* – A foundation which transmits the load to the ground through one or more footings.

2.4 Pile Foundation

2.4.1 *Allowable Load* — The load which may be applied to a pile after taking into account its ultimate load capacity**,** group effect, the allowable settlement, negative skin friction and other relevant loading conditions including reversal of loads, if any.

2.4.2 *Anchor Pile* — An anchor pile means a pile meant for resisting pull or uplift forces.

2.4.3 *Batter Pile (Raker Pile)* – The pile which is installed at an angle to the vertical using temporary casing or permanent liner.

2.4.4 *Bored Cast in-situ Pile* — Piles formed by boring a hole in the ground by percussive or rotary method with the use of temporary/permanent casing or drilling mud and subsequently filling the hole with reinforced concrete.

2.4.5 *Bored Compaction Pile* – A bored cast *in-situ* pile with or without bulb(s) in which the compaction of the surrounding ground and freshly filled concrete in pile bore is simultaneously achieved by a suitable method. If the pile is with bulb(s), it is known as under-reamed bored compaction pile.

2.4.6 *Bored Pile* – A pile formed with or without casing by excavating or boring a hole in the ground and subsequently filling it with plain or reinforced concrete.

2.4.7 *Cut-Off Level* — It is the level where a pile is cut-off to support the pile caps or beams or any other structural components at that level.

2.4.8 *Driven Cast in-situ Pile* — The pile formed within the ground by driving a casing of uniform diameter, or a device to provide enlarged base and subsequently filling the hole with reinforced concrete. For displacing the subsoil the casing is driven with a plug or a shoe at the bottom. When the casing is left permanently in the ground, it is termed as cased pile and when the casing is taken out, it is termed as uncased pile. The steel casing tube is tamped during its extraction to ensure proper compaction of concrete.

2.4.9 *Initial Load Test* — A test pile is tested to determine the load carrying capacity of the pile by loading either to its ultimate load or to twice the estimated safe load.

2.4.10 *Precast Concrete Piles in Prebored Holes* — A pile constructed in reinforced concrete in a casting yard and subsequently lowered into prebored holes and the annular space around the pile grouted.

2.4.11 *Precast Driven Pile* – The pile constructed in concrete in a casting yard and subsequently driven into the ground when it has attained sufficient strength.

2.4.12 *Efficiency of a Pile Group* – It is the ratio of the actual supporting value of a group of piles to the supporting value arrived at by multiplying the pile resistance of an isolated pile by their number in the group.

2.4.13 *Negative Skin Friction* – Negative skin friction is the force developed through the friction between the pile and the soil in such a direction as to increase the loading on the pile, generally due to drag of a consolidating soft layer around the pile resting on a stiffer bearing stratum such that the surrounding soil settles more than the pile.

2.4.14 *Ultimate Load Capacity* – The maximum load which a pile can carry before failure, that is, when the founding strata fails by shear as evidenced from the load settlement curve or the pile fails as a structural member.

2.4.15 *Under-Reamed Pile* – A bored cast *in-situ* or bored compaction concrete pile with enlarged bulb(s) made by either cutting or scooping out the soil or by any other suitable process.

An under-reamed pile having more than one bulb is termed as multi under-reamed piles. The piles having two bulbs may be called double under-reamed piles.

2.4.16 *Pile Spacing* — The spacing of piles means the centre to centre distance between adjacent piles.

2.4.17 *Safe Load* — It is the load derived by applying a factor of safety on the ultimate load capacity of the pile/pile group or as determined from load test.

2.4.18 *Working Load* — The load assigned to a pile as per design.

2.4.19 *Working Pile* — A pile forming part of the foundation system of a given structure.

3 SITE INVESTIGATION

3.1 General

Site investigation is essential in determining the physical, chemical and engineering properties of subsoil to arrive at the required foundation system. However, in areas which have already been developed, information should be obtained regarding the existing local knowledge, records of trial pits, boreholes, etc, in the vicinity, and the behaviour of the existing structures, particularly those of a similar nature to those proposed. This information may be made use for design of foundation of lightly loaded structures of not more than two storeys and also for deciding scope of further investigation for other structures.

3.1.1 If the existing information is not sufficient or is inconclusive, the proposed site should be explored in detail as per good practice [6-2(1)] so as to obtain a knowledge of the type, uniformity, consistency, thickness, sequence and dip of the strata, hydrology of the area and also the engineering properties. In the case of lightly loaded structures of not more than two storeys the tests required to obtain the above information are optional, mainly depending on site conditions.

3.1.1.1 Site reconnaissance would help in deciding future programme of field investigations, that is, to assess the need for preliminary or detailed investigations. Information on some of these may be obtained from topographical maps, geological maps, pedological and soil survey maps, seismic maps and aerial photographs.

Geological maps of the area give valuable information of the site conditions. The general topography will often give some indications of the soil conditions and their variations. In certain cases the earlier uses of the land (like quarries, agricultural land, buried canals, etc) may have a very important bearing on the proposed new structures.

The presence of harmful chemicals in the subsoil can be easily identified by geology or by reconnaissance.

Data regarding removal of overburden by excavation, erosion or landslides should be obtained. This gives an idea of the amount of pre-consolidation the soil strata has undergone. Similarly, data regarding recent fills is also important to study the consolidation characteristics of the fill as well as the original strata. Data regarding swelling or expansive soils will be useful information.

3.1.1.2 *Ground-water conditions*

The ground water level fluctuates and will depend upon the permeability of the strata and the head causing the water to flow. The water level in streams and water courses, if any and wells in the vicinity give useful indications of the ground-water levels.

3.1.1.3 Rock out crops if any may also be indicated during reconnaissance.

3.2 Methods of Site Exploration

3.2.1 *General*

Subsurface explorations should generally be carried out in two stages, that is, preliminary and detailed.

3.2.1.1 *Preliminary exploration*

The investigation at this stage is to explore the feasibility aspects. The scope of preliminary exploration is restricted to the determination of depths, thickness, extent and composition of each soil stratum, location of rock and ground water and also to obtain approximate information regarding strength in compressibility of the various strata.

The number of boreholes to be explored depends upon the importance of the structure, the aerial extent of facility, the topography and the nature of sub strata conditions. Less than five exploratory boreholes, few sounding tests, geophysical methods and often few trial pits are generally adequate in the case of Preliminary Exploration. In a situation of liquefaction possibility, exploratory boreholes with SPT and sampling at very close depth intervals are must, while static cone penetration tests with or without pore water pressure measurement are very useful. The cross hole shear tests and laboratory cyclic shear tests are useful in assessing the liquefaction potential.

NOTE- A method for assessment of liquefaction potential of a site is given in Annex A.

3.2.1.2 *Detailed exploration*

Detailed investigation follow preliminary investigation and should be planned on the basis of data obtained during review as the investigations progress. The scope of detailed exploration is ordinarily restricted to the determine shear strength and compressibility of all types of soils, density, density index, natural moisture content and permeability. It is also necessary to determine the preconsolidation pressure of the strata from oedometer tests and consolidation characteristics beyond the preconsolidation pressure. The detailed investigation includes boring programme, detailed sampling and laboratory tests to determine the physical, chemical and engineering properties of soil and rock.

Field tests which may be performed are in-situ vane shear tests and plate load tests.

3.2.1.3 The common methods of subsoil exploration are given below:

a) The most commonly used exploration procedure is boring in which different field testing procedures, like, standard penetration tests (SPT), field vane shear tests (VST), undisturbed and disturbed sampling (UDS and DS), ground water level observations and field classification are employed.

b) *Open trial pits* – The method consists of excavating trial pits and thereby exposing the subsoil surface thoroughly, enabling undisturbed samples to be taken from the sides and bottom of the trial pits. This is suitable for all types of formations, but should be used for small depths (up to 3 m). In the case of cuts which cannot stand below water table, proper bracing should be given. The data is useful for borrow pits for use as borrow soil.

c) *Auger boring* – The auger is either power or hand operated with periodic removal of the cuttings. Auger boring can be adopted in soft to stiff cohesive soils above water table. Augers shall be of helical or post hole type which may be manually or power operated. While boring, care shall be taken to minimize the disturbance to the deposits below the bottom of the bore hole. The cuttings brought up by the auger shall be carefully examined in the field and the description of all the strata shall be duly recorded. No water shall be introduced from the top while conducting auger boring.

d) *Shell and auger boring* – Both manual and mechanized rig can be used for vertical borings. The tool normally consists of augers for soft to stiff clays, shells for very stiff and hard clays, and shells or sand pumps for sandy strata attached to sectional boring rods.

e) *Wash boring* – In wash boring, the soil is loosened and removed from the bore hole by a stream of water or drilling mud is worked up and down or rotated in the bore hole. The water or mud flow carries the soil up the annular space between the wash pipe and the casing and it overflows at ground level where the soil in suspension is allowed to settle in a pond or tank and the fluid is re-circulated as required. samples of the settled out soil can be retained for identification purposes but this process is often unreliable. However, accurate identification can be obtained if frequent sampling is resorted to using undisturbed sample tubes.

f) *Sounding/Probing including standard penetration test, dynamic and static cone penetration test*

g) *Geophysical methods like electrical resistivity method and seismic method*

h) *Percussion boring*

j) *Rotary boring* – In this system, boring is effected by the cutting action of a rotating bit and a mud-laden fluid or grout is pumped continuously down hollow drill rods and side of the hole.

k) *Core drilling* – To obtain the core samples, core drilling using double table barrel, preferably using hydraulic feed to avoid disturbance to the rock core, is used.

m) *Pressure meter test* (*PMT*) – In this, a uniform radial stress is applied to the borehole and consequent deformation is measured from which limiting pressures and modulus are obtained. This test can also be used where the sampling in hard strata becomes difficult or where there is no UDS collected. For industrial structures, 80 bars capacity energy suffice. However for nuclear projects, higher capacity up to 200 bars may be required.

n) *Plate load test* – This test is conducted at shallow depths with an assumption that it directly determines the bearing capacity. Small size steel plates or relatively large RCC pads are used for conducting this test.

NOTE- While this procedure may be adequate for light or less important structures under normal conditions, relevant laboratory tests or field tests are essential in the case of unusual soil types and for all heavy and important structures. Plate load test, though useful in obtaining the necessary information about the soil with particular reference to design of foundation has some limitations. The test results reflect only the character of the soil located within a depth of less than twice the width of the bearing plate. Since the foundations are generally larger than the test plates, the settlement and shear resistance will depend on the properties of a much thicker stratum. Moreover this method does not give the ultimate settlements particularly in case of cohesive soils. Thus the results of the test are likely to be misleading, if the character of the soil changes at shallow depths, which is not uncommon. A satisfactory load test should, therefore, include adequate soil exploration {*see* good practice [6-2(2)]} with due attention being paid to any weaker stratum below the level of the footing. Another limitation is the concerning of the effect of size of foundation. For clayey soils the bearing capacity (from shear consideration) for a larger foundation is almost the same as that for the smaller test plate. But in dense sandy soils the bearing capacity increases with the size of the foundation. Thus tests with smaller size plate tend to give conservative values in dense sandy soils. It may, therefore, be necessary to test with plates of at least three sizes and the bearing capacity results extrapolated for the size of the actual foundation (minimum dimensions in the case of rectangular footings).

p) *Laboratory investigation* – This gives numerical values for strength, stiffness and consolidation parameters apart from the description and classification of different soil layers.

q) *In-situ permeability test* – In case of high water table and construction extending below the water table, prior lowering of water table would be necessary. In such situations, this test may be required. Hydraulic feed of 20 T capacity helps to obtain data even from hard soils. For small projects IUT hydraulic rig may suffice.

r) Static cone penetration test – Apart from the above methods there are few other sounding procedures like Light Cone Penetration Tests (LCPT) and Dynamic Probing Tests (DPT), in which similar sounding techniques described in the case of DCPT are used, probably more effectively.

3.2.2 *Number and Disposition of Test Locations*

The number and disposition of various tests shall depend upon type of structure/buildings and the soil strata variations in the area. General guidelines are, however, given below:

a) For a compact building site covering an area of about 0.4 hectare, one bore hole or trial pit in each corner and one in the centre should be adequate.

b) For smaller and less important buildings, even one bore hole or trial pit in the centre will suffice.

c) For very large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of boreholes or trial pits. For various commercial, industrial, infrastructure, power plants, cement plants, petrochemical plants, steel plants, pump house on shore or offshore, etc, number of bore holes and/or trial pits should be decided considering importance of structure and type as well as uniformity of strata. As a guide, dynamic or static cone penetration tests may be performed at every 100 m by dividing the area in a grid pattern and the number of bore holes or trial pits may be decided by examining the variation in the penetration curves. The cone penetration tests may not be possible at sites having generally bouldery strata. In such cases, geophysical methods should be resorted to.

3.2.3 *Depth of Exploration*

The depth of exploration required depends on the type of proposed structure, its total weight, the size, shape and disposition of the loaded areas, soil profile, and the physical properties of the soil that constitutes each individual stratum. The borings should be extended to strata of adequate bearing capacity and should penetrate all deposits which are unsuitable for foundation purposes, such as, unconsolidated fill, peat, organic silt and very soft and compressible clay. Normally, it should be one and a half times the width of the footing below foundation level. In certain cases, it may be necessary to take at least one borehole or cone test or both to twice the width of the foundation. If a number of loaded areas are in close proximity the effect of each is additive. In such cases, the whole of the area may be considered as loaded and exploration should be carried out up to one and a half times the lower dimension. In weak soils, the exploration should be continued to a depth at which the loads can be carried by the stratum in question without undesirable settlement and shear failure. In any case, the depth to which seasonal variations affect the soil should be regarded as the minimum depth for the exploration of sites. But where industrial processes affect the soil characteristics, this depth may be more. The presence of fast growing and water seeking trees also contributes to the weathering processes.

NOTE — Examples of fast growing and water seeking trees are Banyan (*Ficus bengalensis*), Pipal (*Ficus religiosa*) and Neem (*Azadirachta indica*).

3.2.4.1 An estimate of the variation with depth of the vertical normal stress in the soil arising from foundation loads may be made on the basis of elastic theory. The net loading intensity at any level below a foundation may be obtained approximately by assuming a spread of load of two vertical to one horizontal from all sides of the foundations, due allowance being made for the overlapping effects of load from closely spaced footings. As a general guidance, the depth of exploration at the start of the work may be decided as given in Table 1, which may be modified as exploration proceeds, if required. However, for industrial plant and other main structures, the depth of exploration may be decided depending upon importance of structure, loading conditions and type as well as uniformity of strata. For piles or piers, the depth shall be at least $2/3L$ below the anticipated tip (that is, termination of pile) or 5.0 m in rock with rock quality designation (RQD) ≥ 50 percent, whichever is earlier.

3.2.4 *Vertical Interval for Field Tests*

Normally, once the type of foundation is decided as shallow footings based on initial findings, the test intervals shall be close to almost continuous at least for a depth equal to 1.5 times the foundation width.

Similarly, in the case of pile foundations, the test intervals shall be significantly small within a depth range of eight times the pile diameter above the pile tip and five times the pile diameter below the pile tip until incompressible stratum is encountered and confined geologically.

In case of housing colonies, the depth of borehole may be kept as 40 m or 5.0 m in rock with rock quality designation RQD ≥ 25 percent, whichever is earlier.

In the case of ground improvement programme like pre-loading, stone columns, compaction etc, the field tests and sampling shall be almost continuous so that presence of thin soil layers of different texture can be identified.

3.3 Choice of Method for Site Exploration

The choice of the method depends on the following factors.

3.3.1 *Nature of Ground*

a) *Soils* — In clayey soils, borings are suitable for deep exploration and pits for shallow exploration. In case of soft sensitive clayey soils, field vane shear test may be carried out with advantage.

   In sandy soils, boring is easy but special equipments such as, Bishop or Osterberg piston samplers should be used for taking undisturbed samples below the water table. where necessary, some form of ground water lowering is used. Standard penetration test, dynamic cone penetration test and static cone penetration test are used to assess engineering properties.

b) *Gravel-boulder deposits* — In the deposits where gravel-boulder proportion is large (>30 percent), the sub-soil strata should be explored by open trial pits of about 5 m × 5 m but in no case less than 2 m × 2 m. The depth of excavation may be up to 6 m. For determining strata characteristics, in-situ tests should be preferred. For shear characteristics and allowable soil pressure dynamic cone penetration tests, load tests on cast in-situ footing and in-situ shear tests that is, boulder-boulder test or concrete-boulder test are more appropriate. For detailed information on these tests reference may be made to good practice [6-2(3)]. Depending on the structure, if required, the strata may be explored by drilling bore hole using suitable method.

c) *Rocks* — Core drillings are suitable in hard rocks and pits in soft rocks. Core borings are suitable for the identification of types of rock, but they provide limited data on joints and fissures. NX borehole camera is useful to photograph the stratification in drilled boreholes. For obtaining core, double tube core barrel with hydraulic feed is preferred. Triple tube core barrel may be used in fragmented or failure or sheared rock. Depending upon the design requirements, large diameter drilling may be explored if feasible.

3.3.2 *Topography*

In hilly country, the choice between vertical openings (for example, borings and trial pits) and horizontal openings (for example, headings) may depend on the geological structure, since steeply inclined strata are most effectively explored by headings and horizontal strata by trial pits or borings. Swamps and areas overlain by water are best explored by borings which may require use of a floating craft.

3.3.3 *Cost*

The cost of investigation varies depending upon project size, stratigraphy, location, marshy areas' access or approaches, etc. The data to be obtained for founding purpose, with a view to reducing or minimizing the surprises, is however, important. It also helps in solutions in respect of selection of appropriate type of foundation, construction materials, etc. For deep exploration, borings are usual, as deep shafts are costly. For shallow exploration in soil, the choice between pits and borings will depend on the nature of the ground and the information required by shallow exploration in rock; the cost of bringing a core drill to the site will be justified only if several holes are required; otherwise, trial pits will be more economical.

3.4 Sampling and Testing

3.4.1 *Methods of Sampling*

a) *Disturbed samples* — These are taken by methods which modify or destroy the natural structure of the material though with suitable precautions the natural moisture content can be preserved.

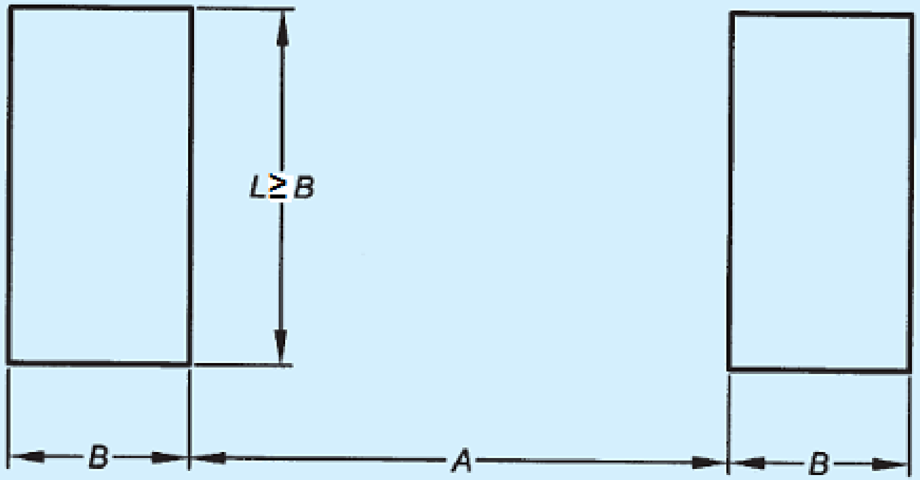
Table 1 Depth of Exploration
(*Clause* 3.2.4.1)

Sl No.	Type of Foundation	Depth of Exploration, *D*
(1)	(2)	(3)
i)	Isolated spread footing or raft	One and a half times the width (*B*) (*see* Fig. 1)
ii)	Adjacent footings with clear spacing less than twice the width	One and a half times the length (*L*) of the footing (*see* Fig. 1)
iii)	Adjacent rows of footings	*See* Fig. 1
iv)	Pile and well foundations	To a depth of one and a half times the width of structure from the bearing (toe of pile or bottom of well)
v)	a) Road cuts	Equal to the bottom width of the cut
	b) Fill	Two metres below ground level or equal to the height of the fill whichever is greater

b) *Undisturbed samples (UDS)* — These are taken by methods which preserve the structure and properties of the material. Thin walled tube samples may be used for undisturbed samples in soils of medium strength and tests for the same may be carried out in accordance with good practice [6-2(1)]. Minimum one UDS shall be obtained from each cohesive layer. For thick layers, the UDS shall be taken at every 3m intervals. UDS need not be taken from cohesion-less soils (silty sand, sand and gravel)

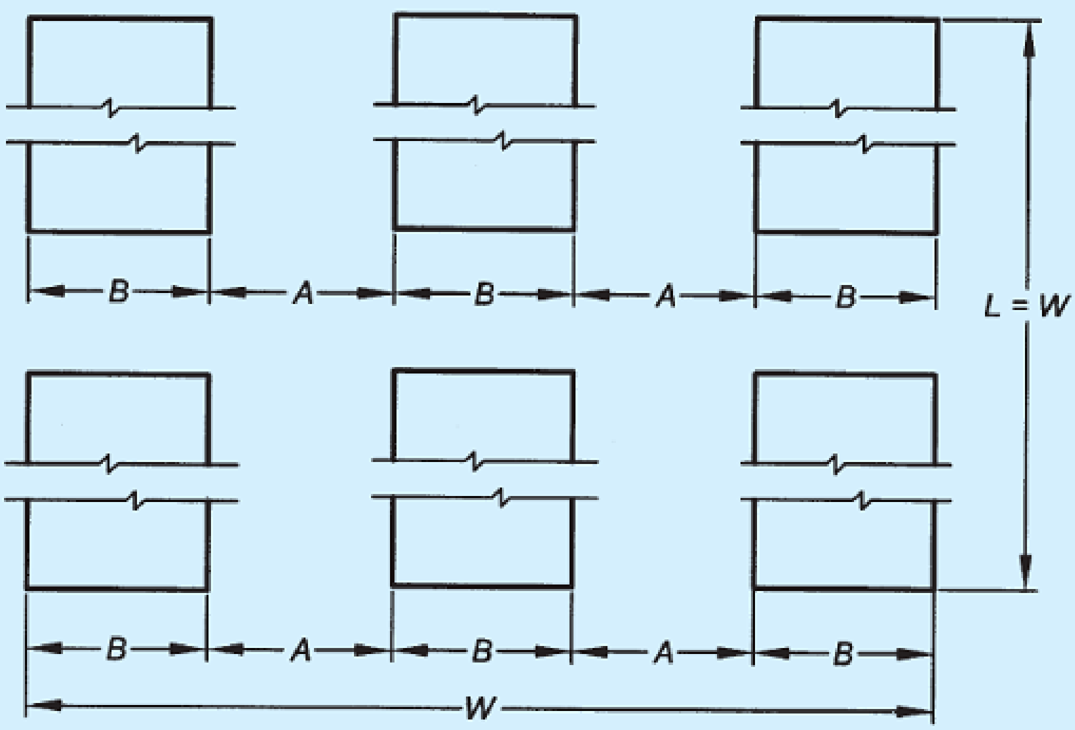
NOTE — In case of loose sandy soils and soft soils, specially below water table it may not be possible to take undisturbed sample, in which case other suitable methods may be adopted for exploration.

c) *Representative samples* — These samples have all their constituent parts preserved, but may or may not be structurally disturbed. Washed samples from rotary boring should not be collected.



$D = 1\frac{1}{2}B$ FOR $A \geq 4B$

$D = 1\frac{1}{2}L$ FOR $A < 2B$



$D = 4\frac{1}{2}B$ FOR $A < 2B$

$D = 3B$ FOR $A > 2B$

$D = 1\frac{1}{2}B$ FOR $A \geq 4B$

FIG. 1 DEPTH OF EXPLORATION

3.4.1.1 The methods usually employed are:

Nature of Ground (1)	*Type of Sample* (2)	*Method of Sampling* (3)
Soil	Disturbed	Chunk samples Auger samples (for example, in clay) Shell samples (for example, in sand)
	Undisturbed	Chunk samples Tube samples
Rock	Disturbed	Wash samples from percussion or rotary drilling
	Undisturbed	Core barrel sampling

3.4.2 *Soil Samples*

a) *Disturbed soil samples* — The mass of sample generally required for testing purposes is given in Table 2.

b) *Undisturbed soil samples* — Undisturbed sampling tube shall be minimum 100 mm diameter with minimum length/diameter ratio of 0 to 2.

3.4.3 *Rock Samples*

a) *Disturbed samples* — The sludge from percussion borings, or from rotary borings which have failed to yield a core, may be taken as a disturbed sample.

b) *Undisturbed samples*

1) *Block samples* — Such samples taken from the rock formation shall be dressed to a size convenient for packing to about 90 mm × 75 mm × 50 mm.

2) *Core samples — Cores of rock shall be taken by means of rotary drills fitted with a coring bit with core retainer*; *see also* good practice [6-2(4)]

3.4.4 *Water Samples*

Water samples have to be collected for chemical analysis. In cases, where mud circulation is used for advancing and stabilizing the boreholes, water samples are to be collected only after 24 h of completion of the borehole. The other option is to collect water samples from open wells in the close proximity. The sample shall be collected in a plastic container (about 500 ml) with airtight cover.

3.4.5 *Protection, Handling and Labelling of Samples*

Care should be taken in protecting, handling and subsequent transport of samples and in their full labelling, so that samples can be received in a fit state for examination and testing, and can be correctly recognized as coming from a specified trial pit or boring.

3.4.6 *Examination and Testing of Samples*

3.4.6.1 The following tests shall be carried out in accordance with accepted standard [6-2(5)].

a) *Tests on Undisturbed and Disturbed Samples*
   1) Visual and engineering classification;
   2) Sieve analysis and hydrometer analysis;
   3) Liquid, plastic, and shrinkage limits;
   4) Specific gravity;
   5) Chemical analysis (Sulphate, chloride and pH content of soil and ground water);
   6) Swell pressure and free swell index determination;
   7) Proctor compaction test; and
   8) California bearing ratio.

b) *Test on Undisturbed Samples*
   1) Bulk density and moisture content;
   2) Relative density (for sand);
   3) Unconfined compression test;
   4) Box shear test (in case of cohesionless and c-Φ soil);
   5) Triaxial shear tests (depending on the type of soil and field conditions on undisturbed or remoulded samples):
      i) Unconsolidated undrained,
      ii) Consolidated undrained test with the measurement of pore water pressure, and
      iii) Consolidated drained;
   6) Consolidation test; and
   7) Laboratory permeability test.

c) *Test on Rock Samples*
   1) Visual classification;
   2) Water absorption, porosity and density;
   3) Specific gravity;
   4) Hardness;
   5) Slake durability;
   6) Unconfined compression test (both saturated and at in-situ water content);

7) Point load strength index; and
8) Deformability test (both saturated and dry samples).

Note — These test may be reduced according to engineering requirements.

Table 2 Mass of Soil Sample Required
[*Clause* 3.4.2(a)]

Sl No.	**Purpose of Sample**	**Type**	**Mass of Sample Required** kg
(1)	(2)	(3)	(4)
i)	Soil identification, natural moisture content tests, mechanical analysis and index properties Chemical tests	Cohesive soil	1
		Sands and gravels	3
ii)	Compaction tests	Cohesive soils and sands	12.5
		Gravely soils	25
iii)	Comprehensive examination of construction materials including stabilization	Cohesive soils and sands	25 to 50
		Gravely soils	50 to 100

NOTES

1 The schedule for laboratory tests shall include the representative of the sub strata layer for testing physical, engineering and chemical properties of soil and rock.

2 Additional tests if any required for assessing borrow material may also be included.

3.5 Soil Investigation Report

The soil investigation report shall generally include the following:

a) Scope of the investigation,
b) Method of drilling of borehole,
c) Sampling of boreholes,

d) Visual description of soil samples,
e) Ground Levels along with Ground water table level,
f) Test results (both laboratory and field),
g) Photographs of samples collected with descriptions,
h) Photo graphs of laboratory test samples after testing,
j) General observations of vicinity (streams, foundation system adopted, etc), and
k) Longitudinal and transverse cross section profiles of substrata.

4 CLASSIFICATION AND IDENTIFICATION OF SOILS

The classification and identification of soils for engineering purposes shall be in accordance with accepted standard [6-2(6)].

5 MATERIALS

5.1 Cement, coarse aggregate, fine aggregate, lime, *SURKHI,* steel, timber and other materials go into the construction of foundations shall conform to the requirements of Part 5 'Building Materials'.

5.2 Protection Against Deterioration of Materials

Where a foundation is to be in contact with soil, water or air, that is, in a condition conducive to the deterioration of the materials of the foundation, protective measures shall be taken to minimize the deterioration of the materials.

5.2.1 *Concrete*

The concrete used for construction shall be in accordance with Part 6 'Structural Design', Section 5 'Concrete'.

5.2.2 *Timber*

Where timber is exposed to soil, it shall be treated in accordance with good practice [6-2(7)].

6 GENERAL REQUIREMENTS FOR FOUNDATIONS/SUB-STRUCTURES FOR BUILDINGS

6.1 Types of Foundations

Types of foundations for buildings covered in this Section are:

a) Shallow Foundations (*see* **7**)
   1) Pad or spread and strip foundations (*see* **7.2**),
   2) Raft foundations (*see* **7.4**),
   3) Ring foundations (*see* **7.5**), and

4) Shell foundations (*see* **7.6**).

b) Deep Foundations
   1) Pile Foundations
      i) Driven cast *in-situ* concrete piles (*see* **8**),
      ii) Bored cast *in-situ* concrete piles (*see* **8**),
      iii) Precast driven concrete piles (*see* **9**),
      iv) Precast concrete piles in prebored holes (*see* **10**),
      v) Under-reamed concrete piles (*see* **11**),
      vi) Timber piles (*see* **12**), and
      vii) Spun piles.

      NOTE - Spun piles are used in deep marshy soils as conventional pile installation beyond 50 m in such soils is very difficult.

c) Other Foundations/Sub-structures/Foundations for Special Structure
   1) Pier foundations (*see* **13.1**),
   2) Diaphragm walls (*see* **13.2**), and
   3) Machine Foundations (*see* **13.2**).

6.2 Depth of Foundations

6.2.1 The depth to which foundations should be carried depends upon the following principal factors:

a) The securing of adequate allowable capacity.
b) In the case of clayey soils, penetration below the zone where shrinkage and swelling due to seasonal weather changes, and due to trees and shrubs are likely to cause appreciable movements.
c) In fine sands and silts, penetration below the zone in which trouble may be expected from frost.
d) The maximum depth of scour, wherever relevant, should also be considered and the foundation should be located sufficiently below this depth.
e) Other factors such as ground movements and heat transmitted from the building to the supporting ground may be important.

6.2.2 All foundations shall extend to a depth of at least 500 mm below natural ground level. On rock or such other weather resisting natural ground, removal of the top soil may be all that is required. In such cases, the surface shall be cleaned and, if necessary, stepped or otherwise prepared so as to provide a suitable bearing and thus prevent slipping or other unwanted movements.

6.2.3 Where there is excavation, ditch pond, water course, filled up ground or similar condition adjoining or adjacent to the subsoil on which the structure is to be erected and which is likely to impair the stability of structure, either the foundation of such structure shall be carried down to a depth beyond the detrimental influence of such conditions, or retaining walls or similar works shall be constructed for the purpose of shielding from their effects.

6.2.4 A foundation in any type of soil shall be below the zone significantly weakened by root holes or cavities produced by burrowing animals or works. The depth shall also be enough to prevent the rainwater scouring below the footings.

6.2.5 Clay soils, like black cotton soils, are seasonally affected by drying, shrinkage and cracking in dry and hot weather, and by swelling in the following wet weather to a depth which will vary according to the nature of the clay and the climatic condition of the region. It is necessary in these soils, either to place the foundation bearing at such a depth where the effects of seasonal changes are not important or to make the foundation capable of eliminating the undesirable effects due to relative movement by providing flexible type of construction or rigid foundations. Adequate load counteracting against swelling pressures also provide satisfactory foundations.

6.3 Foundation at Different Levels

6.3.1 Where footings are adjacent to sloping ground or where the bottoms of the footings of a structure are at different levels or at levels different from those of the footings of adjoining structures, the depth of the footings shall be such that the difference in footing elevations shall be subject to the following limitations:

a) When the ground surface slopes downward adjacent to a footing, the sloping surface shall not intersect a frustum of bearing material under the footing having sides which make an angle of 30° with the horizontal for soil and horizontal distance from the lower edge of the footing to the sloping surface shall be at least 600 mm for rock and 900 mm for soil (*see* Fig. 2).

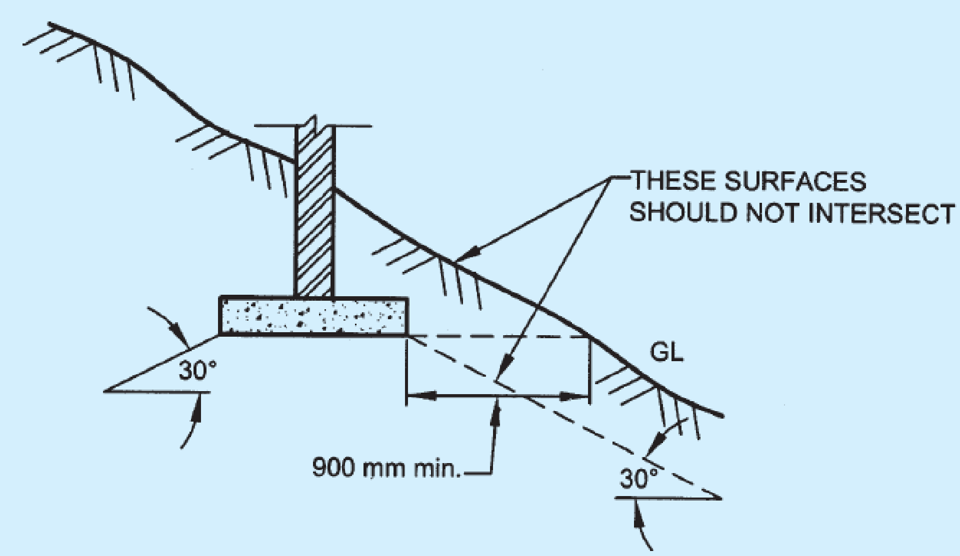


FIG. 2 FOOTING IN SLOPING GROUND

b) In the case of footings in granular soil, a line drawn between the lower adjacent edges of adjacent footings shall not have a steeper slope than one vertical to two horizontal (*see* Fig. 3).

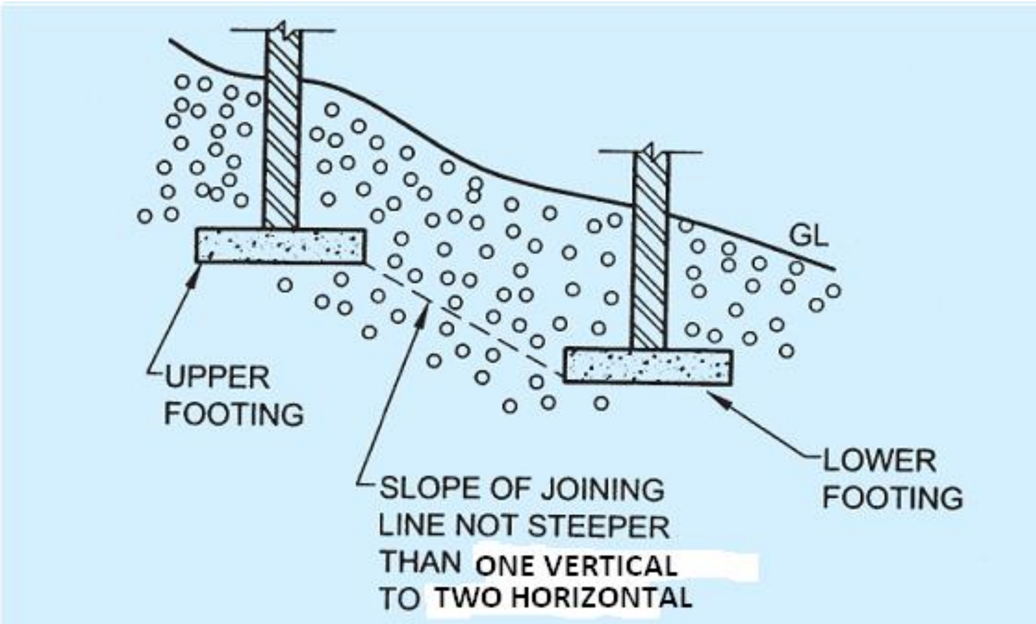


FIG. 3 FOOTING IN GRANULAR SOIL

c) In case of footing in clayey soils a line drawn between the lower adjacent edge of the upper footing and the upper adjacent edge of lower footing shall not have a steeper slope than one vertical to two horizontal (Fig. 4).

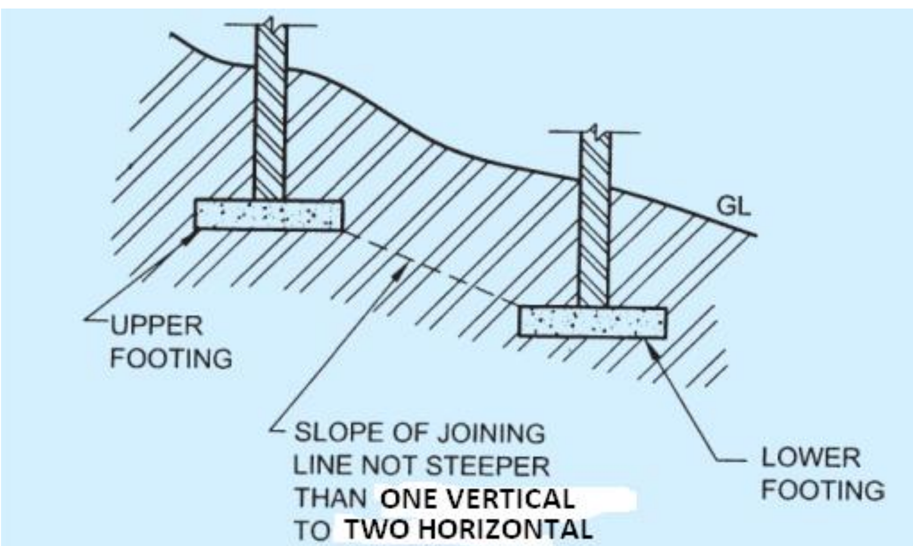


FIG. 4 FOOTING IN CLAYEY SOIL

6.3.2 The requirement given in **6.3.1** shall not apply in the condition where adequate provision is made for the lateral support (such as with retaining walls) of the material supporting the higher footing.

6.4 Effect of Seasonal Weather Changes

During periods of hot, dry weather a deficiency of water develops near the ground surface and in clay soils, that is associated with a decrease of volume or ground shrinkage and the development of cracks. The shrinkage of clay will be increased by drying effect produced by fast growing and water seeking trees. The range of influence depends on size and number of trees and it increase during dry periods. In general, it is desirable that there shall be a distance of at least 8 m between such trees. Boiler installations, furnaces, kilns, underground cables and refrigeration installations and other artificial sources of heat may also cause increased volume changes of clay by drying out the ground beneath them, the drying out can be to a substantial depth. Special precautions either in the form of insulation or otherwise should be taken. In periods of wet weather, clay soils swell and the cracks lend to close, the water deficiency developed in the previous dry periods may be partially replenished and a subsurface zone or zones deficient in water may persist for many years. Leakage from water mains and underground sewers may also result in large volume changes. Therefore, special care must be taken to prevent such leakages.

6.5 Effect of Mass Movements of Ground in Unstable Areas

In certain areas mass movements of the ground are liable to occur from causes independent of the loads applied by the foundations of structures. These include mining subsidence, landslides on unstable slopes and creep on clay slopes.

6.5.1 *Mining Subsidence*

In mining areas, subsidence of the ground beneath a building or any other structure is liable to occur. The magnitude of the movement and its distribution over the area are likely to be uncertain and attention shall, therefore, be directed to make the foundations and structures sufficiently rigid and strong to withstand the probable worst loading condition and probable ground movements. Long continuous buildings should be avoided in such areas and large building in such area should be divided into independent sections of suitable size, each with its own foundations. Expert advice from appropriate mining authority should be sought.

> **NOTE –** For prediction of subsidence in coal mines, guidelines as given in the good practice [6-2(8)] may be referred.

6.5.2 *Landslide Prone Areas*

The construction of structures on slopes which are suspected of being unstable and are subject to landslip shall be avoided.

On sloping ground on clay soils, there is always a tendency for the upper layers of soil to move downhill, depending on type of soil, the angle of slope, climatic conditions, etc. In some cases, the uneven surface of the slope on a virgin ground will indicate that the area is subject to small land slips and, therefore, if used for foundation, will obviously necessitate special design consideration.

Where there may be creep of the surface layer of the soil, protection against creep may be obtained by following special design considerations.

On sloping sites, spread foundations shall be on a horizontal bearing and stepped. At all changes of levels, they shall be lapped at the steps for a distance at least equal to the thickness of the foundation or twice the height of the step, whichever is greater. The steps shall not be of greater height than the thickness of the foundation, unless special precautions are taken.

Cuttings, excavations or sloping ground near and below foundation level may increase the possibility of shear failure of the soil. The foundation shall be well beyond the zone of such shear failure.

If the probable failure surface intersects a retaining wall or other revetment, the latter shall be made strong enough to resist any unbalanced thrust. In case of doubt as to the suitability of the natural slopes or cuttings, the structure shall be kept well away from the top of the slopes, or the slopes shall be stabilized.

Cuttings and excavations adjoining foundations reduce stability and increase the likelihood of differential settlement. Their effect should be investigated not only when they exist but also when there is possibility that they are made subsequently.

Where a structure is to be placed on sloping ground, additional complications are introduced. The ground itself, particularly if of clay, may be subject to creep or other forms of instability, which may be enhanced if the strata dip in the same direction as the ground surface. If the slope of the ground is large, the overall stability of the slope and substructure may be affected. These aspects should be carefully investigated.

6.6 Precautions for Foundations on Inclined Strata

In the case of inclined strata, if they dip towards a cutting of basement, it may be necessary to carry foundation below the possible slip planes, land drainage also requires special consideration, particularly on the uphill side of a structure to divert the natural flow of water away from the foundations.

6.7 Strata of Varying Thickness

Strata of varying thickness, even at appreciable depth, may increase differential settlement. Where necessary, calculations should be made of the estimated settlement from different thickness of strata and the structure should be designed accordingly.

When there is large change of thickness of soft strata, the stability of foundation may be affected. Site investigations should, therefore, ensure detection of significant variations in strata thickness.

6.8 Layers of Softer Material

Some soils and rocks have thin layers of softer material between layers of harder material, which may not be detected unless thorough investigation is carried out. The softer layers may undergo marked changes in properties if the loading on them is increased or decreased by the proposed construction or affected by related changes in ground water conditions. These should be taken into account.

6.9 Spacing Between Existing and New Foundation

The deeper the new foundation and the nearer to the existing it is located, the greater the damage is likely to be. The minimum horizontal spacing between existing and new footings shall be equal to the width of the wider one. While the adoption of such provision shall help minimizing damage to adjacent foundation, an analysis of bearing capacity and settlement shall be carried out to have an appreciation of the effect on the adjacent existing foundation.

6.10 Preliminary Work for Construction

6.10.1 The construction of access roads, main sewers and drains should preferably be completed before commencing the work of foundations; alternatively, sufficient precautions shall be taken to protect the already constructed foundations during subsequent work.

6.10.2 *Clearance of Site*
Any obstacles, including the stump of trees, likely to interfere with the work shall be removed. Holes left by digging, such as those due to removal of old foundation, uprooted trees, burrowing by animals, etc, shall be back-filled with soil and well compacted.

6.10.3 *Drainage*

If the site of a structure is such that surface water shall drain towards it, land may be dressed or drains laid to divert the water away from the site.

6.10.4 *Setting Out*

Generally the site shall be levelled before the layout of foundations are set out. In case of sloping terrain, care shall be taken to ensure that the dimensions on plans are set out correctly in one or more horizontal planes.

6.10.5 The layout of foundations shall be set out with steel tapes. Angles should be set out with theodolites in the case of important and intricate structures where the length of area exceeds 16 m. In other cases these shall be set out by measurement of sides. In rectangular or square setting out, diagonals shall be checked to ensure accuracy. The setting out of walls shall be facilitated by permanent row of pillars, parallel to and at a suitable distance beyond the periphery of the building. The pillars shall be located at junctions of cross walls with the peripheral line of pillars. The centre lines of the cross walls shall be extended to and permanently erected on the plastered tops of the corresponding sets of pillars. The datum lines parallel to and at the known fixed distance from the centre lines of the external walls also be permanently worked on the corresponding rows of pillars to pillars to serve as checks on the accuracy of the work as it proceeds. The tops of these pillars shall be at the same level and preferably at the plinth or floor level. The pillars shall be of sizes not less than 250 mm wide and shall be bedded sufficiently deep into ground so that they are not disturbed.

6.11 Protection of Excavation

6.11.1 The protection of excavation during construction of timbering and dewatering operations, where necessary, shall be done in accordance with [6-2(9)].

6.11.2 After excavation, the bottom of the excavation shall be cleared of all loose soil and rubbish and shall be levelled, where necessary. The bed shall be wetted and compacted by heavy rammers to an even surface.

6.11.3 Excavation in clay or other soils that are liable to be effected by exposure to atmosphere shall, wherever possible, be concreted as soon as they are dug. Alternatively the bottom of the excavation shall be protected immediately by 8 cm thick layer of cement concrete not leaner than mix 1:5:10 over which shall come the foundation concrete; or in order to obtain a dry hard bottom, the last excavation of about 10 cm shall be removed only before concreting.

6.11.4 The refilling of the excavation shall he done with care so as not to disturb the constructed foundation, and shall be compacted in layers not exceeding 15 cm thick with sprinkling of minimum quantity of water necessary for proper compaction.

6.12 Alterations During Construction

a) Where during construction the soil or rock to which foundation is to transfer loads is found not to be the type or in the condition assumed, the foundation shall be redesigned and constructed for the existing type or conditions and the Authority notified.
b) Where a foundation bears on gravel, sand or silt and where the highest level of the ground water is or likely to be higher than an elevation defined by bearing surface minus the width of the footing, the bearing pressure shall be altered in accordance with Note 4 in Table 3.

c) Where the foundation has not been placed or located as indicated earlier or is damaged or bears on a soil whose properties may be adversely changed by climatic and construction conditions, the error shall be corrected, the damaged portion repaired or the design capacity of the affected foundation recalculated to the satisfaction of the Authority.
d) Where a foundation is placed, and if the results of a load test so indicate, the design of the foundation shall be modified to ensure structural stability of the same.

7 SHALLOW FOUNDATIONS

7.1 Design Information

For the satisfactory design of foundations, the following information is necessary:

a) The type and condition of the soil or rock to which the foundation transfers the loads;
b) The general layout of the columns and load-bearing walls showing the estimated loads, including moments and torques due to various loads (dead load, imposed load, wind load, seismic load) coming on the foundation units;
c) The allowable bearing pressure of the soils;
d) The changes in ground water level, drainage and flooding conditions and also the chemical conditions of the subsoil water, particularly with respect to its sulphate content;
e) The behaviour of the buildings, topography and environment/ surroundings adjacent to the site, the type and depths of foundations and the bearing pressure assumed; and
f) Seismic zone of the region.

7.2 Design Considerations

7.2.1 *Design Loads*

The foundation shall be proportioned for the following combination of loads:

a) Dead load + imposed load; and
b) Dead load + imposed load + wind load or seismic loads, whichever is critical.

For details, reference shall be made to Part 6 Structural Design, Section 1 Loads, Forces and Effects.

NOTE - For shallow foundations on coarse grained soils, settlements shall be estimated corresponding to **7.2.1 (b)** and for foundations on fine grained soils, the settlement shall be estimated corresponding to permanent loads only. Permanent loads shall be in accordance with good practice [6-2(10)] IS 1904.

7.2.2 *Allowable Bearing Pressure*

The allowable bearing pressure shall be taken as either of the following, whichever is less:

a) The safe bearing capacity on the basis of shear strength characteristics of soil, or
b) The bearing pressure that the soil can take without exceeding the permissible settlement (*see* **7.2.3**).

7.2.2.1 *Bearing capacity by calculation*

Where the engineering properties of the soil are available, that is, cohesion, angle of internal friction, density, etc, the bearing capacity shall be calculated from stability considerations of shear; factor of safety of 2.5 shall be adopted for safe bearing capacity. The effect of interference of different foundations should be taken into account. The procedure for determining the ultimate bearing capacity and bearing pressure of shallow foundations based on shear and allowable settlement criteria shall be in accordance with good practice [6-2(11)]. Depth factor correction is to be applied only when backfilling is done with proper compaction.

7.2.2.2 *Field method for determining allowable bearing pressure*

Where appropriate, plate load tests can be performed and allowable pressure determined as per accepted standard [6-2(12)]. The allowable bearing pressure for sandy soils may also be obtained by loading tests. When such tests cannot be done, the allowable bearing pressure for sands may be determined using penetration test.

7.2.2.3 Where the bearing materials directly under a foundation over-lie a stratum having smaller safe bearing capacity, these smaller values shall not be exceeded at the level of such stratum.

7.2.2.4 *Effect of wind and seismic force*

7.2.2.4.1 Where the bearing pressure due to wind is less than 25 percent of that due to dead and imposed loads, it may be neglected in design. Where this exceeds 25 percent foundations may be so proportioned that the pressure due to combined dead, imposed and wind loads does not exceed the allowable bearing pressure by more than 25 percent.

7.2.2.4.2 When earthquake forces are considered for the computation of design loads, the permissible increase in allowable bearing pressure of pertaining soil shall be as given in Table 3, depending upon the type of foundation of the structure and the type of soil.

In soil deposits consisting of submerged loose sand and soils falling under classification SP with standard penetration *N*-values less than 15 in seismic Zones III, IV, V (*see* Part 6 'Structural Design', Section 1 'Loads, Forces and Effects') and less than 10 in seismic Zone II, the vibration caused by earthquake may cause liquefaction or excessive total and differential settlements. Such sites should preferably be avoided while locating new settlements or important projects. Otherwise, this aspect of the problem needs to be investigated and appropriate methods of ground improvement/compaction or stabilization adopted to achieve suitable *N*-values as indicated in Note 3 under Table 3. Alternatively deep pile foundation may be provided and taken to depths well into the layer which is not likely to liquefy. Marine clays and other sensitive clays are also known to liquefy due to collapse of soil structure and will need special treatment according to site condition.

NOTE - Liquefaction potential of a site may be assessed in accordance with Annex A.

7.2.2.5 *Bearing capacity of buried strata*

If the base of a foundation is close enough to a strata of lower bearing capacity, the latter may fail due to excess pressure transmitted to it from above. Care should be taken to see that the pressure transmitted to the lower strata is within the prescribed safe limits. When the footings are closely spaced, the pressure transmitted to the underlying soil will overlap. In such cases, the pressure in the overlapped zones will have to be considered. With normal foundations, it is sufficiently accurate to estimate the bearing pressure on the underlying layers by assuming the load to be spread at a slope of 2 (vertical) to 1 (horizontal).

7.2.3 *Settlement*

The permissible values of total and differential settlement for a given type of structure may be taken as given in Table 4. Total settlements of foundation due to net imposed loads shall be estimated in accordance with good practice [6-2(14)]. The following causes responsible for producing the settlement shall be investigated and taken into account.

a) *Causes of Settlement*

1) Elastic compression of the foundation and the underlying soil,
2) Consolidation including secondary compression,
3) Ground water lowering - Specially repeated lowering and raising of water level in loose granular soils tend to compact the soil and cause settlement of the footings. Prolonged lowering of the water table in fine grained soils may introduce settlement because of the extrusion of water from the voids. Pumping water or draining water by tiles or pipes from granular soils without an adequate mat of filter

material as protection may, in a period of time, carry a sufficient amount of fine particles away from the soil and cause settlement.

4) Seasonal swelling and shrinkage of expansive clays.
5) Ground movement on earth slope, for example, surface erosion, slow creep or landslides.
6) Other causes, such as adjacent excavation, mining, subsidence and underground erosion.

Table 3 Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

(*Clauses* 7.2.2.4 *and* 8.2.7)

Sl No.	**Foundation**	**Type of Soil Mainly Constituting the Foundation**		
		Type I – Rock or hard Soil : Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GP, GW, GC, SW, and SC)[1] having N[2] above 30, where N is the standard penetration value	Type II - Medium Soils: All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP[1]) with $N > 15$	Type III Soft Soils: All soils other than SP[1] with N <10 and SP with N > 15
(1)	(2)	(3)	(4)	(5)
1	Piles passing through any soil but resting on soil type I	50	50	50
2	Piles not covered under item 1	-	25	25
3	Raft Foundations	50	50	50
4	Combined isolated RCC footing with tie Beams	50	25	25
5	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	-
6	Well foundations	50	25	25

NOTES

1 The allowable bearing pressure shall be determined in accordance with good practice [6-2(11)] and [6-2(12)].
2 If any increase in bearing pressure has already been permitted for forces other than seismic forces , the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.
3 Desirable minimum field values of *N* - If soils of smaller *N*-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.
4 The preferred values of *N* (corrected values) at the founding level may be as given below to avoid liquefaction and the allowable bearing pressure shall be determined in accordance with good practice [6-2(11)] and [6-2(12)].

Seismic Zone	Depth below ground level (in metres)	*N* Values	Remark
III, IV & V	≤ 5	15	For values of depths between 5 metres and 10 meters, linear interpolation is recommended
	≥ 10	25	
II (for important structures only)	≤ 5	10	
	≥ 10	20	

5 The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy. Assessment of liquefaction potential of soil shall be made in accordance with Annex A.

6 Accepted standards [6-2(6)] and [6-2(13)] may also be referred.

7 Isolated R.C.C. footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with *N* < 10.

1) *See* accepted standard [6-2(6)].
2) *See* accepted standard [6-2(13)].

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b) *Causes of Differential Settlements*

1) Geological and physical non-uniformity or anomalies in type, structure, thickness, and density of the soil medium (pockets of sand in clay, clay lenses in sand, wedge like soil strata, that is, lenses in soil), an admixture of organic matter, peat, mud;
2) Non-uniform pressure distribution from foundation to the soil due to non-uniform loading and incomplete loading of the foundations;
3) Water regime at the construction site,
4) Overstressing of soil at adjacent site by heavy structures built next to light ones;

5) Overlap of stress distribution in soil from adjoining structures;
6) Unequal expansion of the soil due to excavation for footing;
7) Non-uniform development of extrusion settlements; and
8) Non-uniform structural disruptions or disturbance of soil due to freezing and thawing, swelling and softening and drying of soils.

7.2.4 *Shallow Foundations on Rocks*

Estimation of the safe bearing pressures of rocks for shallow foundations based on strength, allowable settlement and classification criteria; and also design and construction of shallow foundations on rocks shall be carried out in accordance with the good practice [6-2(15)].

7.3 Pad or Spread and Strip Foundations

7.3.1 In such type of foundations wherever the resultant of the load deviates from the centre line by more than 1/6 of its least dimension at the base of footing, it should be suitably reinforced.

7.3.2 For continuous wall foundations (plain or reinforced) adequate reinforcement should be provided particularly at places where there is abrupt change in magnitude of load or variation in ground support.

7.3.3 On sloping sites the foundation should have a horizontal bearing and stepped and lapped at changes of levels for a distance at least equal to the thickness of foundation or twice the height of step whichever is greater. The steps should not be of greater height than thickness of the foundations.

7.3.4 *Ground Beams*

The foundation can also have the ground beam for transmitting the load. The ground beam carrying a load bearing wall should be designed to act with the wall forming a composite beam, when both are of reinforced concrete and structurally connected by reinforcement. The ground beam of reinforced concrete structurally connected to reinforced brick work can also be used.

Table 4 Permissible Differential Settlements and Tilt (Angular Distortion) for Shallow Foundations in Soils

(*Clause* 7.1.3)

Sl No.	Type of Structure	Isolated Foundations						Raft Foundations					
		Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
		Maximum settlement mm	Differential settlement mm	Angular distortion	Maximum settlement mm	Differential settlement mm	Angular distortion	Maximum settlement mm	Differential settlement mm	Angular distortion	Maximum settlement mm	Differential settlement mm	Angular distortion
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	For steel structure	50	.003 3*L*	1/300	50	.003 3*L*	1/300	75	.003 3*L*	1/300	100	.003 3*L*	1/300
ii)	For reinforced concrete structures	75	.001 5*L*	1/500	75	.002*L*	1/500	75	.003 3*L*	1/300	125	.003 3*L*	1/300
iii)	For multistoreyed buildings												
	a) RC or steel framed buildings with panel walls	60	.002*L*	1/500	75	.003L	1/300	75	.002 5*L*	1/400	125	.003 3L	1/300
	b) For load bearing walls												
	1) *L*/H=2*	60	.000 2*L*	1/5 000	60	.000 2*L*	1/5 000	← Not likely to be encountered →					
	2) *L*/H=7*	60	.000 4*L*	1/2 500	60	.000 4*L*	1/2 500						
iv)	For water towers and silos	50	.001 5*L*	1/666	75	.001 5*L*	1/666	100	.002 5*L*	1/400	125	.002 5*L*	1/400

NOTE – The values given in the table may be taken only as a guide and the permissible total settlement/different settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer.

L denotes the length of deflected part of wall/raft or centre-to-centre distance between columns.
H denotes the height of wall from foundation footing.
* For intermediate ratios of *L*/*H*, the values can be interpolated.

7.3.5 *Dimensions of Foundation*

The dimensions of the foundation in plan should be such as to support loads as given in good practice [6-2(10)]. The width of the footings shall be such that maximum stress in the concrete or masonry is within the permissible limits. The width of wall foundation (in mm) shall not be less than that given by:

$$B = W + 300$$

where

B = width at base in mm, and

W = width of supported wall in mm.

7.3.6 In the base of foundations for masonry foundation it is preferable to have the steps in multiples of thickness of masonry unit.

7.3.7 The plan dimensions of excavation for foundations should be wide enough to ensure safe and efficient working in accordance with good practice [6-2(9)].

7.3.8 Unreinforced foundation may be of concrete or masonry (stone or brick) provided that angular spread of load from the base of column/wall or bed plate to the outer edge of the ground bearing is not more than 1 vertical to $1/2$ horizontal for masonry or 1 vertical to 1 horizontal for cement concrete and 1 vertical to $2/3$ horizontal for lime concrete. The minimum thickness of the foundation of the edge should not be less than 150 mm. In case the depth to transfer the load to the ground bearing is less than the permissible angle of spread, the foundations should be reinforced.

7.3.9 If the bottom of a pier is to be belled so as to increase its load carrying capacity such bell should be at least 300 mm thick at its edge. The sides should be sloped at an angle of not less than 45° with the horizontal. The least dimension should be 600 mm (circular, square or rectangular). The design should allow for the vertical tilt of the pier by 1 percent of its height.

7.3.10 If the allowable bearing capacity is available only at a greater depth, the foundation can be rested at a higher level for economic considerations and the difference in level between the base of foundation and the depth at which the allowable bearing capacity occurs can be filled up with either: (a) concrete of allowable compressive strength not less than the allowable bearing pressure, (b) in compressible fill material, for example, sand, gravel, etc. in which case the width of the fill should be more than the width of the foundation by an extent of dispersion of load from the base of the foundation on either side at the rate of 2 vertical to 1 horizontal.

7.3.11 The cement concrete foundation (plain or reinforced) should be designed in accordance with Part 6 ‘Structural Design’, Section 5A 'Plain and Reinforced Concrete' and masonry foundation in accordance with Part 6 ‘Structural Design’, Section 4 'Masonry'.

7.3.12 *Thickness of Footing*

The thickness of different types of footings, if not designed according to **7.2**, should be as given in Table 5.

7.3.13 *Land Slip Area*

On a sloping site, spread foundation shall be on a horizontal bearing and stepped. At all changes of levels, they shall be lapped at the steps for a distance at least equal to the thickness of the foundation or twice the height of the step, whichever is greater. The steps shall not be of greater height than the thickness of the foundation unless special precautions are taken. On sloping ground on clay soils, there is always a tendency for the upper layers of soil to move downhill, depending on type of soil, the angle of slope, climatic conditions, etc. Special precautions are necessary to avoid such a failure.

Table 5 Thickness of Footings
(*Clause* 7.2.12)

Sl No. (1)	**Type of Footings** (2)	**Thickness of Footings, *Min*** (3)	**Remarks** (4)
i)	Masonry	a) 250 mm b) Twice the maximum projection from the face of the wall	Select the greater of the two values
ii)	Plain concrete		
	For normal structures	a) 200 mm	-
		b) Twice the maximum offset in a stepped footing	For footings resting on soil
		c) 300 mm	For footings resting on top of the pile
	For lightly loaded structures	a) 150 mm	Resting on soil
		b) 200 mm	Resting on pile
iii)	Reinforced concrete	a) 150 mm	Resting on soil
		b) 300 mm	Resting on pile

7.3.14 In the foundations, the cover to the reinforcement shall be as prescribed in Part 6 'Structural Design', Section 5 'Concrete' for the applicable environment exposure condition.

7.3.15 For detailed information regarding preparation of ground work, reference may be made to good practice [6-2(16)].

7.4 Raft Foundations

7.4.1 *Design Considerations*

Design provisions given in **7.2** shall generally apply.

7.4.1.1 The structural design of reinforced concrete rafts shall conform to Part 6 'Structural Design', Section 5 'Concrete'.

7.4.1.2 In the case of raft, whether resting on soil directly or on lean concrete, the cover to the reinforcement shall be as prescribed in Part 6 'Structural Design', Section 5 'Concrete' for the applicable environment exposure condition.

7.4.1.3 In case the structure supported by the raft consists of several parts with varying loads and heights, it is advisable to provide separation joints between these parts. Joints shall also be provided wherever there is a change in the direction of the raft.

7.4.1.4 Foundations subject to heavy vibratory loads should preferably be isolated.

7.4.1.5 The minimum depth of foundation shall generally be not less than one metre.

7.4.1.6 *Dimensional parameters*

The size and shape of the foundation adopted affect the magnitude of subgrade modulus and long term deformation of the supporting soil and this, in turn, influences the distribution of contact pressure. This aspect needs to be taken into consideration in the analysis.

7.4.1.7 *Eccentricity of loading*

A raft generally occupies the entire area of the building and often it is not feasible and rather uneconomical to proportion it coinciding the centroid of the raft with the line of action of the resultant force. In such cases, the effect of the eccentricity on contact pressure distribution shall be taken into consideration.

7.4.1.8 *Properties of supporting soil*

Distribution of contact pressure underneath a raft is affected by the physical characteristics of the soil supporting it. Consideration must be given to the increased contact pressure developed along the edges of foundation on cohesive soils and the opposite effect on granular soils. Long term consolidation of deep soil layers shall be taken into account in the analysis. This may necessitate evaluation of contact pressure distribution both immediately after construction and after completion of the consolidation process. The design must be based on the worst conditions.

7.4.1.9 *Rigidity of foundations*

Rigidity of the foundation tends to iron out uneven deformation and thereby modifies the contact pressure distribution. High order of rigidity is characterized by long moments and relatively small, uniform settlements. A rigid foundation may also generate high secondary stresses in structural members. The effect of rigidity shall be taken into account in analysis.

7.4.1.10 *Rigidity of the super structure*

Free response of the foundations to soil deformation is restricted by the rigidity of the superstructure. In the extreme case, a stiff structure may force a flexible foundation to behave as rigid. This aspect shall be considered to evaluate the validity of the contact pressure distribution.

7.4.1.11 *Modulus of elasticity and modulus of subgrade reaction*

Annex B enumerates the methods of determination of modulus of elasticity (E_s). The modulus of subgrade reaction (k) may be determined in accordance with Annex C.

7.4.2 *Necessary Information*

The following information is necessary for a satisfactory design and construction of a raft foundation:

a) Site plan showing the location of the proposed as well as the neighbouring structures;
b) Plan and cross-sections of building showing different floor levels, shafts and openings, etc, layout of loading bearing walls, columns, shear walls, etc;
c) Loading conditions, preferably shown on a schematic plan indicating combination of design loads transmitted to the foundation;
d) Information relating to geological history of the area, seismicity of their area, hydrological information indicating ground water conditions and its seasonal variations, etc;

e) Geotechnical information giving subsurface profile with stratification details, engineering properties of the founding strata (namely, index properties, effective shear parameters determined under appropriate drainage conditions, compressibility characteristics, swelling properties, results of field tests like static and dynamic penetration tests, pressure meter tests etc); and

f) A review of the performance of similar structure, if any, in the locality.

7.4.3 *Choice of Raft Type*

7.4.3.1 For fairly small and uniform column spacing and when the supporting soil is not too compressible a flat concrete slab having uniform thickness throughout (a true mat) is most suitable (*see* Fig. 5A).

7.4.3.2 A slab may be thickened under heavy loaded columns to provide adequate strength for shear and negative moment. Pedestals may also be provided in such cases (*see* Fig. 5B).

7.4.3.3 A slab and beam type of raft is likely to be more economical for large column spacing and unequal column loads particularly when the supporting soil is very compressive (*see* Fig. 5C and 5D).

7.4.3.4 For very heavy structures, provision of cellular raft or rigid frames consisting of slabs and basement walls may be considered.

7.4.4 *Methods of Analysis*

The essential task in the analysis of a raft foundation is the determination of the distribution of contact pressure underneath the raft which is a complex function of the rigidity of the superstructure, the supporting soil and the raft itself, and cannot be determined with exactitude, except in very simple cases. This necessitates a number of simplifying assumptions to make the problem amenable to analysis. Once the distribution of contact pressure is determined, design bending moments and shears can be computed based on statics. The methods of analysis suggested are distinguished by the assumptions involved. Choice of a particular method should be governed by the validity of the assumptions in the particular case.

7.4.4.1 *Rigid foundation (conventional method)*

This method is based on the assumption of linear distribution of contact pressure. The basic assumptions of this method are:

a) the foundations rigid relative to the supporting soil and the compressible soil layer is relatively shallow; and

b) the contact pressure variation is assumed as planar, such that the centroid of the contact pressure coincides with the line of action of the resultant force of all loads acting on the foundation.

This method may be used when either of the following conditions is satisfied:

a) The structure behaves as rigid (due to the combined action of the superstructure and the foundation) with relative stiffness factor $K > 0.5$ (for evaluation of K *see* Annex D); and
b) The column spacing is less than $1.75/\lambda$ (*see* Annex D).

The raft is analyzed as a whole in each of the two perpendicular directions. The contact pressure distribution is determined by the procedure outlined in Annex E. Further analysis is also based on statics.

In the case of uniform conditions when the variations in adjacent column loads and column spacings do not exceed 20 percent of the higher value, the raft may be divided into perpendicular strips of widths equal to the distance between midspans and each strip may be analyzed as an independent beam with known column loads and known contact pressures. Such beams will not normally satisfy statics due to shear transfer between adjacent strips and design may be based on suitable moment coefficients, or by moment distribution.

NOTE - On soft soils, for example, normally consolidated clays, peat, muck, organic silts, etc, the assumptions involved in the conventional method are commonly justified.

7.4.4.2 *Flexible foundations*

a) *Simplified Method* - In this method, it is assumed that the subgrade consists of an infinite array of individual elastic springs each of which is not affected by others. The spring constant is equal to the modulus of subgrade reaction (k). The contact pressure at any point under the raft is, therefore, linearly proportional to the settlement at the point. Contact pressure may be determined as given in Annex F. This method may be used when all the following conditions are satisfied:

   1) The structure (combined action of superstructure and raft) may be considered as flexible (relative stiffness factor $K < 0.5$, *see* Annex D).
   2) Variation in adjacent column load does not exceed 20 percent of the higher value.

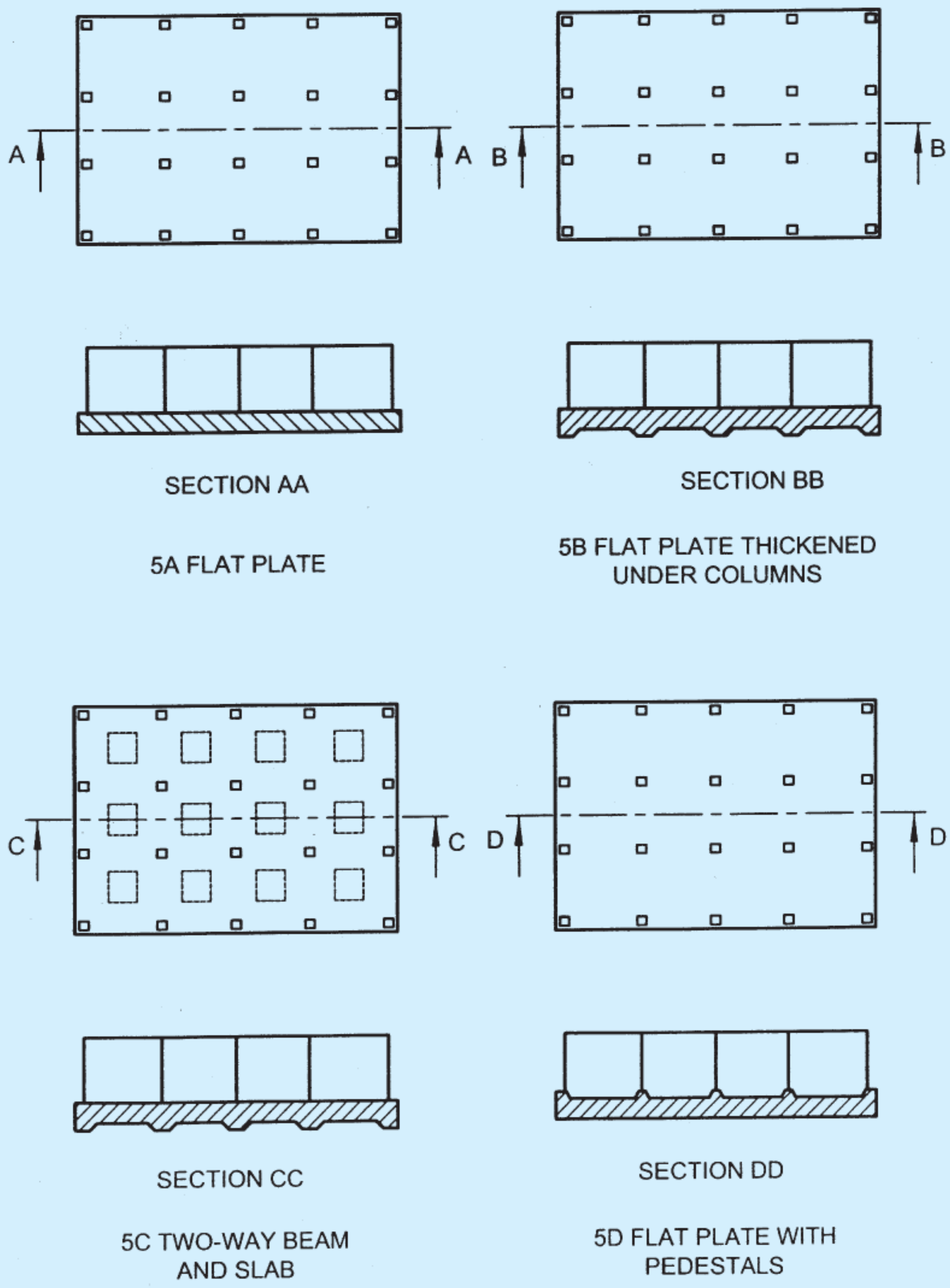


FIG. 5 COMMON TYPES OF RAFT FOUNDATION

b) *General Method* - for the general case of a flexible foundation not satisfying the requirements of (a), the method based on closed form solution of elastic plate theory may be used. This method is based on the theory of plates on winkler foundation which takes into account the restraint on deflection of a point provided by continuity of the foundation in orthogonal foundation. The distribution of deflection and contact pressure on the raft due to a column load is determined by the plate theory. Since the effect of a column load on an elastic foundation is damped out rapidly, it is possible to determine the total effect at a point of all column loads within the zone of influence by the method of superimposition. The computation of effect at any point may be restricted to columns of two adjoining bays in all directions. The procedure is outlined in Annex G.

7.5 Ring Foundations

For provisions regarding ring foundations, good practice [6-2(17)] shall be referred to.

7.6 Shell Foundations

For provisions regarding shell foundations, good practice [6-2(18)] shall be referred to.

8 DRIVEN/BORED CAST IN-SITU CONCRETE PILES

8.0 General

Piles find application in foundations to transfer loads from a structure to competent subsurface strata having adequate load-bearing capacity. The load transfer mechanism from a pile to the surrounding ground is complicated and is not yet fully understood, although application of piled foundations is in practice over many decades. Broadly, piles transfer axial loads either substantially by friction along its shaft and/or by end bearing. Piles are used where either of the above load transfer mechanism is possible depending upon the subsoil stratification at a particular site. Construction of pile foundations require a careful choice of piling system depending upon the subsoil conditions, the load characteristics of a structure and the limitations of total settlement, differential settlement and any other special requirement of a project.

8.1 Materials and Stresses

8.1.1 *Concrete*

Consistency of concrete to be used shall be consistent with the method of installation of piles. Concrete shall be so designed or chosen as to have a homogeneous mix having a slump/workability consistent with the method of concreting under the given conditions of pile installation.

The slump should be 150 to180 mm at the time of pouring.

The minimum grade of concrete to be used for bored piling shall be M 25. For sub aqueous concrete, the requirements specified in Part 6 Structural Design, Section 5A Plain and Reinforced Concrete shall be followed. The minimum cement content shall be 400 kg/m^3. However with proper mix design and use of proper admixture the cement content may be reduced but in no case the cement content shall be less than 350 kg/m^3.

For the concrete, water and aggregates, specifications laid down in Part 6 'Structural Design', Section 5A 'Plain and Reinforced Concrete' shall be followed in general.

The average compressive stress under working load should not exceed 25 percent of the specified works cube strength at 28 days calculated on the total cross-sectional area of the pile. Where the casing of the pile is permanent, of adequate thickness and of suitable shape, the allowable compressive stress may be increased.

8.1.2 *Steel Reinforcement*

Steel reinforcement shall conform to any one of the types of steel specified in Part 6 Structural Design', Section 5A 'Plain and Reinforced Concrete'.

8.2 Design Considerations

8.2.1 *General*

Pile foundations shall be designed in such a way that the load from the structure can be transmitted to the sub-surface with adequate factor of safety against shear failure of sub-surface and without causing such settlement, (differential or total), which may result in structural damage and/or functional distress under permanent/transient loading. The pile shaft should have adequate structural capacity to withstand all loads (vertical, axial or otherwise) and moments which are to be transmitted to the subsoil and shall be designed according to Part 6 Structural Design, Section 5A Plain and Reinforced Concrete.

8.2.2 *Adjacent structures*

8.2.2.1 When working near existing structures, care shall be taken to avoid damage to such structures. The good practice [6-2(19)] may be used as a guide for studying qualitatively the effect of vibration on persons and structures.

8.2.2.2 In case of deep excavations adjacent to piles, proper shoring or other suitable arrangement shall be made to guard against undesired lateral movement of soil.

8.2.3 *Pile capacity*

The load carrying capacity of a pile depends on the properties of the soil in which it is embedded. Axial load from a pile is normally transmitted to the soil through skin friction along the shaft and end bearing at its tip. A horizontal load on a vertical pile is transmitted to the subsoil primarily by horizontal subgrade reaction generated in the upper part of the shaft. Lateral load capacity of a single pile depends on the soil reaction developed and the structural capacity of the shaft under bending. It would be essential to investigate the lateral load capacity of the pile using appropriate values of horizontal subgrade modulus of the soil. Alternatively, piles may be installed in rake.

8.2.3.1 The ultimate load capacity of a pile may be estimated by means of static formula on the basis of soil test results, or by using a dynamic pile formula using data obtained during driving the pile. However, dynamic pile driving formula should be generally used as a measure to control the pile driving at site. Pile capacity should preferably be confirmed by initial load tests [*see* the good practice [6-2(20)]. For rock-socketed piles, reference shall also be made to good practice [6-2(21)] for estimating the load capacity of piles.

The settlement of pile obtained at safe load/working load from load-test results on a single pile shall not be directly used for estimating the settlement of a structure. The settlement may be determined on the basis of subsoil data and loading details of the structure as a whole using the principles of soil mechanics.

8.2.3.1.1 *Static formula*

The ultimate load capacity of a single pile may be obtained by using static analysis, the accuracy being dependent on the reliability of the soil properties for various strata. When computing capacity by static formula, the shear strength parameters obtained from a limited number of borehole data and laboratory tests should be supplemented, wherever possible by *in-situ* shear strength obtained from field tests. The two separate static formulae, commonly applicable for cohesive and non-cohesive soil respectively, are indicated in Annex H. Other formula based on static cone penetration test [*see* the accepted standards {6-2(22)}] and standard penetration test [*see* the accepted standard {6-2(13)}] are given in **Annex H-3** and **Annex H-4**.

8.2.3.1.2 *Dynamic formula*

For driven piles, any established dynamic formula may be used to control the pile driving at site giving due consideration to limitations of various formulae.

Whenever double acting diesel hammers or hydraulic hammers are used for driving of piles, manufacturer's guidelines about energy and set criteria may be referred to. Dynamic formulae are not directly applicable to cohesive soil deposits such as saturated silts and clays as the resistance to impact of the tip of the casing will be exaggerated by their low permeability while the frictional resistance on the sides is reduced by lubrication.

8.2.3.1.3 *Load test results*

The ultimate load capacity of a single pile is determined with reasonable accuracy from test loading as per good practice [6-2(20)]. The load test on a pile shall not be carried out earlier than four weeks from the time of casting the pile.

8.2.3.2 *Uplift capacity*

The uplift capacity of a pile is given by sum of the frictional resistance and the weight of the pile (buoyant or total as relevant). The recommended factor of safety is 3.0 in the absence of any pullout test results and 2.0 with pullout test results. Uplift capacity can be obtained from static formula (*see* Annex H) by ignoring end bearing but adding weight of the pile (buoyant or total as relevant).

8.2.4 *Negative Skin Friction or Dragdown Force*

When a soil stratum, through which a pile shaft has penetrated into a underlying hard stratum, compresses as a result of either it being unconsolidated or it being under a newly placed fill or as a result of remoulding during driving of the pile, a dragdown force is generated along the pile shaft up to a point in depth where the surrounding soil does not move downward relative to the pile shaft. Existence of such a phenomenon shall be assessed and suitable correction shall be made to the allowable load where appropriate.

8.2.5 *Structural Capacity*

The piles shall have necessary structural strength to transmit the loads imposed on it, ultimately to the soil. In case of uplift, the structural capacity of the pile, that is, under tension should also be considered.

8.2.5.1 *Axial capacity*

Where a pile is wholely embedded in the soil (having an undrained shear strength not less than 0.01 N/mm^2), its axial load carrying capacity is not necessarily limited by its strength as a long column. Where piles are installed through very weak soils (having an undrained shear strength less than 0.01 N/mm^2), special considerations shall be made to determine whether the shaft would behave as a long column or not. If necessary, suitable reductions shall be made for its structural strength following the normal structural principles covering the buckling phenomenon.

When the finished pile projects above ground level and is not secured against buckling by adequate bracing, the effective length will be governed by the fixity imposed on it by the structure it supports and by the nature of the soil into which it is installed. The depth below the ground surface to the lower point of contraflexure varies with the type of the soil. In good soil the lower point of contraflexure may be taken at a depth of 1 m below ground surface subject to a minimum of 3 times the diameter of the shaft. In weak soil (undrained shear strength less than 0.01 N/mm^2) such as soft clay or soft silt, this point may be taken at about half the depth of penetration into such stratum but not more than 3 m or 10 times the diameter of the shaft whichever is more. The degree of fixity of the position and inclination of the pile top and the restraint provided by any bracing shall be estimated following accepted structural principles.

The permissible stress shall be reduced in accordance with similar provision for reinforced concrete columns as laid down in Part 6 ‘Structural Design’, Section 5A ‘Plain and Reinforced Concrete’.

8.2.5.2 *Lateral load capacity*

A pile may be subjected to lateral force for a number of causes, such as wind, earthquake, water current, earth pressure, effect of moving vehicles or ships, plant and equipment, etc. The lateral load capacity of a single pile depends not only on the horizontal subgrade modulus of the surrounding soil but also on the structural strength of the pile shaft against bending, consequent upon application of a lateral load. While considering lateral load on piles, effect of other co-existent loads, including the axial load on the pile, should be taken into consideration for checking the structural capacity of the shaft. A recommended method for the pile analysis under lateral load in given in Annex J.

Because of limited information on horizontal subgrade modulus of soil and pending refinements in the theoretical analysis, it is suggested that the adequacy of a design should be checked by an actual field load test. In the zone of soil susceptible to liquefaction, the lateral resistance of the soil shall not be considered.

8.2.5.2.1 *Fixed and free head conditions*

A group of three or more pile connected by a rigid pile cap shall be considered to have fixed head condition. Caps for single piles must be interconnected by grade beams in two directions and for twin piles by grade beams in a line transverse to the common axis of the pair so that the pile head is fixed. In all other conditions the pile shall be taken as free headed.

8.2.5.4 *Raker piles*

Raker piles are normally provided where vertical piles cannot resist the applied horizontal forces. Generally the rake will be limited to 1 horizontal to 6 vertical. In the preliminary design, the load on a raker pile is generally considered to be axial. The distribution of load between raker and vertical piles in a group may be determined by graphical or analytical methods. Where necessary, due consideration should be made for secondary bending induced as a result of the pile cap movement, particularly when the cap is rigid. Free-standing raker piles are subjected to bending moments due to their own weight or external forces from other causes. Raker piles, embedded in fill or consolidating deposits, may become laterally loaded owing to the settlement of the surrounding soil. In consolidating clay, special precautions, like provision of permanent casing should be taken for raker piles.

8.2.6 *Spacing of Piles*

The minimum centre to centre spacing of pile is considered from three aspects, namely,

a) practical aspects of installing the piles;
b) diameter of the pile; and
c) nature of the load transfer to the soil and possible reduction in the load capacity of piles group.

NOTE — In the case of piles of non-circular cross-section, diameter of the circumscribing circle shall be adopted.

8.2.6.1 In case of piles founded on hard stratum and deriving their capacity mainly from end bearing the minimum spacing shall be 2.5 times the diameter of the circumscribing circle corresponding to the cross-section of the pile shaft. In case of piles resting on rock, the spacing of two times the said diameter may be adopted.

8.2.6.2 Piles deriving their load carrying capacity mainly from friction shall be spaced sufficiently apart to ensure that the zones of soils from which the piles derive their support do not overlap to such an extent that their bearing values are reduced. Generally the spacing in such cases shall not be less than 3 times the diameter of the pile shaft.

8.2.7 Pile Groups

8.2.7.1 In order to determine the load carrying capacity of a group of piles a number of efficiency equations are in use. However, it is difficult to establish the accuracy of these efficiency equations as the behaviour of pile group is dependent on many complex factors. It is desirable to consider each case separately on its own merits.

8.2.7.2 The load carrying capacity of a pile group may be equal to or less than the load carrying capacity of individual piles multiplied by the number of piles in the group. The former holds true in case of friction piles, driven into progressively stiffer materials or in end-bearing piles. For driven piles in loose sandy soils, the group capacity may even be higher due to the effect of compaction. In such cases a load test may be carried out on a pile in the group after all the piles in the group have been installed.

8.2.7.3 In case of piles deriving their support mainly from friction and connected by a rigid pile cap, the group may be visualized as a block with the piles embedded within the soil. The ultimate load capacity of the group may then be obtained by taking into account the frictional capacity along the perimeter of the block and end bearing at the bottom of the block using the accepted principles of soil mechanics.

8.2.7.3.1 When the cap of the pile group is cast directly on reasonably firm stratum which supports the piles, it may contribute to the load carrying capacity of the group. This additional capacity along with the individual capacity of the piles multiplied by the number of piles in the group shall not be more than the capacity worked out as per **8.2.7.3**.

8.2.7.4 When a pile group is subjected to moment either from superstructure or as a consequence of inaccuracies of installation, the adequacy of the pile group in resisting the applied moment should be checked. In case of a single pile subjected to moment due to lateral loads or eccentric loading, beams may be provided to restrain the pile cap effectively from lateral or rotational movement.

8.2.7.5 In case of a structure supported on single piles/group of piles resulting in large variation in the number of piles from column to column it may result in excessive differential settlement. Such differential settlement should be either catered for in the structural design or it may be suitably reduced by judicious choice of variations in the actual pile loading. For example, a single pile cap may be loaded to a level higher than that of the pile in a group in order to achieve reduced differential settlement between two adjacent pile caps supported on different number of piles.

8.2.8 Factor of Safety

8.2.8.1 Factor of safety should be chosen after considering,

a) the reliability of the calculated value of ultimate load capacity of a pile;
b) the types of superstructure and the type of loading; and
c) allowable total/differential settlement of the structure.

8.2.8.2 When the ultimate load capacity is determined from either static formula or dynamic formula, the factor of safety would depend on the reliability of the formula and the reliability of the subsoil parameters used in the computation. The minimum factor of safety on static formula shall be 2.5. The final selection of a factor of safety shall take into consideration the load settlement characteristics of the structure as a whole at a given site.

8.2.8.3 Higher value of factor of safety for determining the safe load on piles may be adopted, where,

a) settlement is to be limited or unequal settlement avoided;
b) large impact or vibrating loads are expected; and
c) the properties of the soil may deteriorate with time.

8.2.9 *Transient Loading*

The maximum permissible increase over the safe load of a pile, as arising out of wind loading, is 25 percent. In case of loads and moments arising out of earthquake effects, the increase of safe load shall be as given in **7.2.2.4.2** and Table 3. For transient loading arising out of superimposed loads, no increase is allowed.

8.2.10 *Overloading*

When a pile in a group, designed for a certain safe load is found, during or after execution, to fall just short of the load required to be carried by it, an overload up to 10 percent of the pile capacity may be allowed on each pile. The total overloading on the group should not, however, be more than 10 percent of the capacity of the group subject to the increase of the load on any pile being not more than 25 percent of the allowable load on a single pile.

8.2.11 *Reinforcement*

8.2.11.1 The design of the reinforcing cage varies depending upon the driving and installation conditions, the nature of the subsoil and the nature of load to be transmitted by the shaft - axial, or otherwise. The minimum area of longitudinal reinforcement of any type or grade within the pile shaft shall be 0.4 percent of the cross-sectional area of the pile shaft. The minimum reinforcement shall be provided throughout the length of the shaft.

8.2.11.2 The curtailment of reinforcement along the depth of the pile, in general, depends on the type of loading and subsoil strata. In case of piles subjected to compressive load only, the designed quantity of reinforcement may be curtailed at appropriate level according to the design requirements. For piles subjected to uplift load, lateral load and moments, separately or with compressive loads, it would be necessary to provide reinforcement for the full depth of pile. In soft clays or loose sands, or where there may be danger to green concrete due to driving of adjacent piles, the reinforcement should be provided to the full pile depth, regardless of whether or not it is required from uplift and lateral load considerations. However, in all cases, the minimum reinforcement specified in **8.2.11.1** shall be provided throughout the length of the shaft.

8.2.11.3 Piles shall always be reinforced with a minimum amount of reinforcement as dowels keeping the minimum bond length into the pile shaft below its cut-off level and with adequate projection into the pile cap, irrespective of design requirements.

8.2.11.4 Clear cover to all main reinforcement in pile shaft shall be not less than 50 mm. The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handing and installations. The minimum diameter of the links or spirals shall be 8 mm and the spacing of the links or spirals shall be not less than 150 mm. Stiffner rings preferably of 16 mm diameter at every 1.5 m centre to centre should be provided along the length of the cage for providing rigidity to reinforcement cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars can be bundled to maintain such spacing.

8.2.12 Design of Pile Cap

8.2.12.1 The pile caps may be designed by assuming that the load from column is dispersed at 45º from the top of the cap to the mid-depth of the pile cap from the base of the column or pedestal. The reaction from piles may also be taken to be distributed at 45º from the edge of the pile, up to the mid-depth of the pile cap. On this basis the maximum bending moment and shear forces should be worked out at critical sections. The method of analysis and allowable stresses should be in accordance with Part 6 Structural Design, Section 5A Plain and Reinforced Concrete.

8.2.12.2 Pile cap shall be deep enough to allow for necessary anchorage of the column and pile reinforcement.

8.2.12.3 The pile cap should be rigid enough so that the imposed load could be distributed on the piles in a group equitably.

8.2.12.4 In case of a large cap, where differential settlement may occur between piles under the same cap, due consideration for the consequential moment should be given.

8.2.12.5 The clear overhang of the pile cap beyond the outermost pile in the group shall be a minimum of 150 mm.

8.2.12.6 The cap is generally cast over a 75 mm thick levelling course of concrete. The clear cover for main reinforcement in the cap slab shall not be less than 60 mm.

8.2.12.7 The embedment of pile into cap should be 75 mm.

8.2.13 *Grade Beams*

8.2.13.1 The grade beams supporting the walls shall be designed taking due account of arching effect due to masonry above the beam. The beam with masonry due to composite action behaves as a deep beam.

For the design of beams, a maximum bending moment of $\frac{wl^2}{50}$, where *w* is uniformly distributed load per metre run (worked out by considering a maximum height of two storeys in structures with load bearing walls and one storey in framed structures) and *l* is the effective span in metres, will be taken if the beams are supported during construction till the masonry above it gains strength. The value of bending moment shall be increased to $\frac{wl^2}{30}$, if the beams are not supported. For considering composite action, the minimum height of wall shall be 0.6 times the beam span. The brick strength should not be less than 3 N/mm^2. For concentrated and other loads which come directly over the beam, full bending moment should be considered.

8.2.13.2 The minimum overall depth of grade beams shall be 150 mm. The reinforcement at the bottom should be kept continuous and an equal amount may be provided at top to a distance of a quarter span both ways from pile centers. The longitudinal reinforcement both at top and bottom should not be less than three bars of 10 mm diameter mild steel (or equivalent deformed steel) and stirrups of 6 mm diameter bars should be spaced at a minimum of 300 mm spacing.

8.2.13.3 In expansive soils, the grade beams shall be kept a minimum of 80 mm clear off the ground. In other soils, beams may rest on ground over a leveling concrete course of about 80 mm (*see* Fig. 6).

8.2.13.4 In the case of exterior beams over piles in expensive soils, a ledge projection of 75 mm thickness and extending 80 mm into ground (*see* Fig. 6) shall be provided on the outer side of the beam.

8.3 For detailed information on driven/bored cast *in-situ* concrete piles regarding control of piling, installation, defective pile and recording of data, reference shall be made to good practice [6-2(23)].

8.4 Bored Cast In-Situ Concrete Piles on Rocks

Design and construction of bored cast-in-situ piles founded on rocks shall be carried out in accordance with good practice [6-2(21)].

8.5 Non-destructive testing

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with good practice [6-2(24)].

9 DRIVEN PRECAST CONCRETE PILES

9.1 Provisions of **8** except **8.2.11** shall generally apply.

9.2 Design of Pile Section

9.2.1 Design of pile section shall be such as to ensure the strength and soundness of the pile against lifting from the casting bed, transporting, handling and driving stresses without damage.

9.2.2 Any shape having radial symmetry will be satisfactory for precast piles. The most commonly used cross-sections are square and octagonal.

9.2.3 Where exceptionally long lengths of piles are required, hollow sections can be used. If the final condition requires larger cross-sectional area, the hollow sections can be filled with concrete after driving in position.

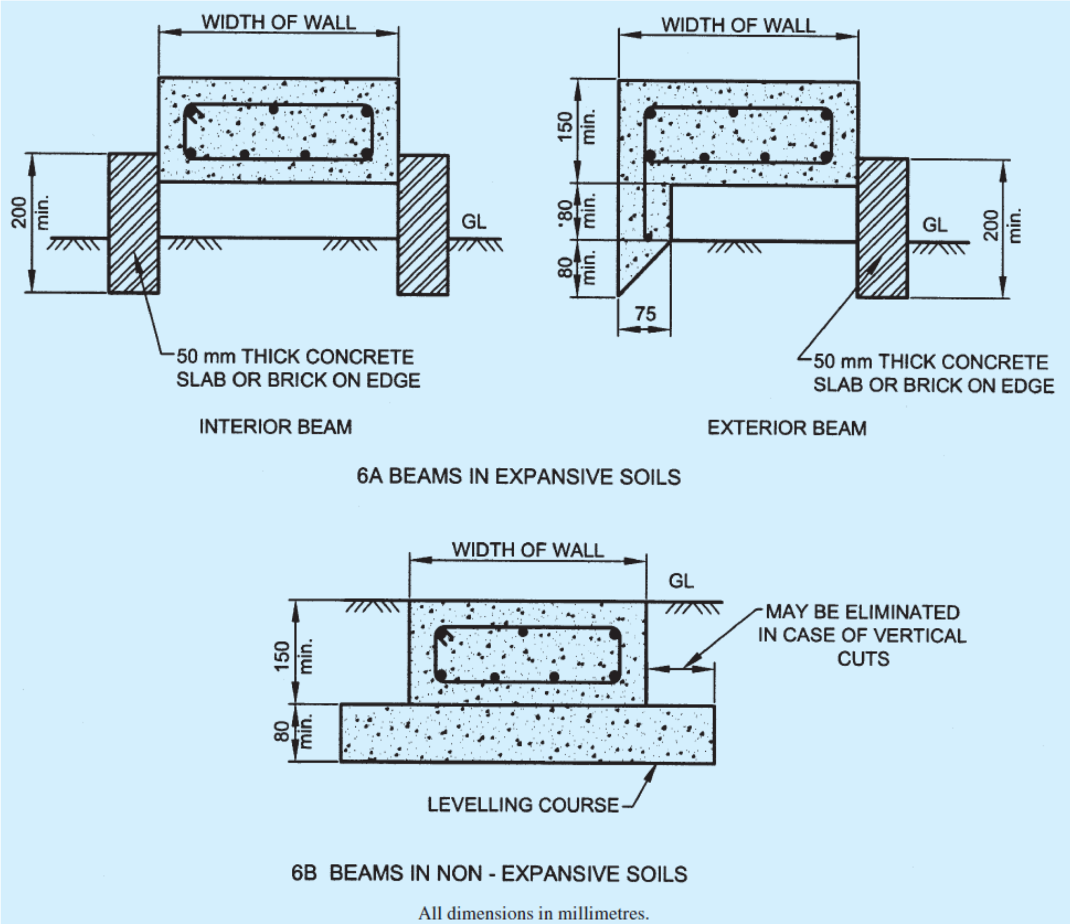


FIG. 6 TYPICAL SECTIONS OF GRADE BEAMS

9.2.4 Wherever final pile length is so large that a single length precast pile unit is either uneconomical or impracticable for installation, the segmental precast RCC piles with a number of segments using efficient mechanical jointing could be adopted.

Excessive whipping during handling precast pile may generally be avoided by limiting the length of pile to a maximum of 50 times the least width. As an alternative, segmental precast piling technique could be used.

The design of joints shall take care of corrosion by providing additional sacrificial thickness for the joint, wherever warranted.

9.2.5 Stresses induced by bending in the cross section of precast pile during lifting and handling may be estimated as for any reinforced concrete section in accordance with relevant provisions of Part 6 'Structural Design', Section 5A 'Plain and Reinforced Concrete'. The calculations for bending moment for different support conditions during handling are given in Table 6.

Table 6 Bending Moment for Different Support Conditions
(*Clause* 9.2.5)

Sl No.	Number of Points of Pick Up	Location of Support from End in Terms of Length of Pile for Minimum Moments	Bending Moment to be Allowed for Design kN-m
(1)	(2)	(3)	(4)
i)	One	0.293 *L*	4.3 *WL*
ii)	Two	0.207 *L*	2.2 *WL*
iii)	Three	0.145 *L*, the middle point will be at the centre	1.05 *WL*

NOTE— *W* = weight of pile, in kN.
L = length of pile, in m.

9.2.6 The driving stresses on a pile may be estimated by the following formula:

$$\frac{\text{Driving resistance}}{\text{Cross-sectional area of the pile}} \times \left[\frac{2}{\sqrt{n}} - 1\right]$$

where

n is the efficiency of the blow (*see* **9.2.6.1** for probable value of *n*).

NOTES- For the purpose of this formula, cross-sectional area of the pile shall be calculated as the overall sectional area of the pile including the equivalent area for reinforcement.

9.2.6.1 The formula for efficiency of the blow, representing the ratio of energy after impact to striking energy of ram, *n*, is:

Where *W* is greater than *P.e* and the pile is driven into penetrable ground,

$$n = \frac{W + \left(P \cdot e^2\right)}{W + P}$$

Where *W* is less than *P.e* and the pile is driven into penetrable ground,

$$n = \left[\frac{W + \left(P \cdot e^2\right)}{W + P}\right] - \left[\frac{W - \left(P \cdot e\right)}{W + P}\right]^2$$

The following are the values of *n* in relation to *e* and to the ratio of *P*/*W*:

Ratio of *P*/*W*	*e* = 0.5	*e* = 0.4	*e* = 0.32	*e* = 0.25	*e* = 0
½	0.75	0.72	0.70	0.69	0.67
1	0.63	0.58	0.55	0.53	0.50
1½	0.55	0.50	0.47	0.44	0.40
2	0.50	0.44	0.40	0.37	0.33
2½	0.45	0.40	0.36	0.33	0.28
3	0.42	0.36	0.33	0.30	0.25
3½	0.39	0.33	0.30	0.27	0.22
4	0.36	0.31	0.28	0.25	0.20
5	0.31	0.27	0.24	0.21	0.16
6	0.27	0.24	0.21	0.19	0.14
7	0.24	0.21	0.19	0.17	0.12
8	0.22	0.20	0.17	0.15	0.11

NOTES

1 *W* = mass of the ram, in tonne and *P* = weight of the pile, anvil, helmet, and follower (if any) in tonne.

2 Where the pile finds refusal in rock, 0.5*P* should be substituted for *P* in the above expressions for *n*.

3 *e* is the coefficient of restitution of the materials under impact as tabulated below:

a) For steel ram of double-acting hammer striking on steel anvil and driving reinforced concrete pile, *e* = 0.5.
b) For cast-iron ram of single-acting or drop hammer striking on head of reinforced concrete pile, *e* = 0.4.
c) Single-acting or drop hammer striking a well-conditioned driving cap and helmet with hard wood dolly in driving reinforced concrete piles or directly on head of timber pile, *e* = 0.25.
d) For a deteriorated condition of the head of pile or of dolly, *e* = 0.

9.3 *Reinforcement*

9.3.1 The longitudinal reinforcement of any type or grade shall be provided in precast reinforced concrete piles for the entire length. All the main longitudinal bars shall be of the same length and should fit tightly into the pile shoe if there is one. Shorter rods to resist local bending moments may be added but the same should be carefully detailed to avoid any sudden discontinuity of the steel which may lead to cracks during heavy driving. The area of main longitudinal reinforcement shall not be less than the following percentages of the cross-sectional area of the piles:

a) For piles with a length less than 30 times the least width — 1.25 percent,
b) For piles with a length 30 to 40 times the least width — 1.5 percent, and
c) For piles with a length greater than 40 times the least width — 1.5 percent.

9.3.2 Piles shall always be reinforced with a minimum amount of reinforcement as dowels keeping the minimum bond length into the pile shaft below its cut-off level and with adequate projection into the pile cap, irrespective of design requirements.

9.3.3 Clear cover to all main reinforcement in pile shaft shall be not less than 50 mm. The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handing and installations. The minimum diameter of the links or spirals shall be 8 mm and the spacing of the links or spirals shall be not less than 150 mm. Stiffner rings preferably of 16 mm diameter at every 1.5 m centre to centre to be provided along the length of the cage for providing rigidity to reinforcement cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars can be bundled to maintain such spacing.

9.4 For detailed information regarding casting and curing, storing and handling, control of pile driving and recording of data, reference may be made to good practice [6-2(25)].

9.5 Non-destructive testing

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with good practice [6-2(24)].

10 PRECAST CONCRETE PILES IN PREBORED HOLES

10.1 Provisions of **9** except **9.3** shall generally apply.

10.2 Handling Equipment for Lowering and Grouting Plant

Handling equipment such as crane, scotch derricks, movable gantry may be used for handling and lowering the precast piles in the bore. The choice of equipment will depend upon length, mass and other practical requirements.

The mixing of the grout shall be carried out in any suitable high speed colloidal mixer. Normally the colloidal mixer is adequate to fill the annular space with grouts. Where this is not possible, a suitable grout pump shall be used.

10.3 Reinforcement

10.3.1 The design of the reinforcing cage varies depending upon the handling and installation conditions, the nature of the subsoil and the nature of load to be transmitted by the shaft - axial, or otherwise. The minimum area of longitudinal reinforcement of any type or grade within the pile shaft shall be 0.4 percent of the cross-sectional area of the pile shaft or as required to cater for handling stresses (*see* **9.2.5**), whichever is greater. The minimum reinforcement shall be provided throughout the length of the shaft.

10.3.2 Piles shall always be reinforced with a minimum amount of reinforcement as dowels keeping the minimum bond length into the pile shaft below its cut-off level and with adequate projection into the pile cap, irrespective of design requirements.

10.3.3 Clear cover to all main reinforcement in pile shaft shall be not less than 50 mm. The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handing and installations. The minimum diameter of the links or spirals shall be 8 mm and the spacing of the links or spirals shall be not less than 150 mm. Stiffner rings preferably of 16 mm diameter at every 1.5 m centre to centre to be provided along length of the cage for providing rigidity to reinforcement cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars can be bundled to maintain such spacing.

10.3.4 A thin gauge sheathing pipe of approximately 40 mm diameter may be attached to the reinforcement cage, in case of solid piles, to form the central duct for pumping grout to the bottom of the bore. The bottom end of the pile shall have proper arrangements for flushing/cleaning for grouting. Air lift technique may be used for cleaning the borehole, however this technique should be used carefully in case of silty and sandy soil.

10.4 For detailed information regarding casting and curing, storing and handling, control of pile installation and recording of data, reference may be made to good practice [6-2(26)].

10.5 Non-destructive testing

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with good practice [6-2(24)].

11 UNDER-REAMED PILES

11.0 General

Under-reamed piles are bored cast-in-situ and bored compaction concrete types having one or more bulbs formed by enlarging the borehole for the pile stem. These piles are suited for expansive soils which are often subjected to considerable ground movements due to seasonal moisture variations. These also find wide application in other soil strata where economics are favorable. When the ground consists of expansive soil, for example black cotton soils, the bulb of under-reamed pile provide anchorage against upift due to swelling pressure, apart from the increased bearing, provided topmost bulb is formed close to or just below the bottom of active zone. Negative slopes may not be stable in certain strata conditions, for example, in pure sands (clean sands with fines less than five per cent) and very soft clayey strata having N of SPT less than 2 (undrained shear strength of less than 12.5 kN/m^2). Hence formation of bulb(s) in such strata is not advisable. In sub-soil strata above water table, the maximum number of bulbs in under-reamed pile should be restricted to four. In the strata such as clay, silty clay and clayey silt with high water table where sides of bore hole stand by itself without needing any stabilization by using drilling mud or otherwise, the maximum number of bulbs in under-reamed piles should be restricted to two. In strata for example clayey sand, silty sand and sandy silt with high water table where bore hole needs stabilization by using drilling mud, under-reamed piles with more than one bulb shall not be used. In loose to medium pervious strata such as clayey sand, silty sand and sandy silt strata, compaction under-reamed piles can be used as the process of compaction, increases the load carrying capacity of piles. From practical considerations, uner-reamed piles of more than 10 m depth shall not be used without ensuring their construction feasibility and load carrying capacity by initial load tests in advance. In view of additional anchorage available with the provision of bulbs, under-reamed piles can be used with advantage to resist uplift loads.

11.1 Materials

11.1.1 Provisions of **8.1** shall generally apply.

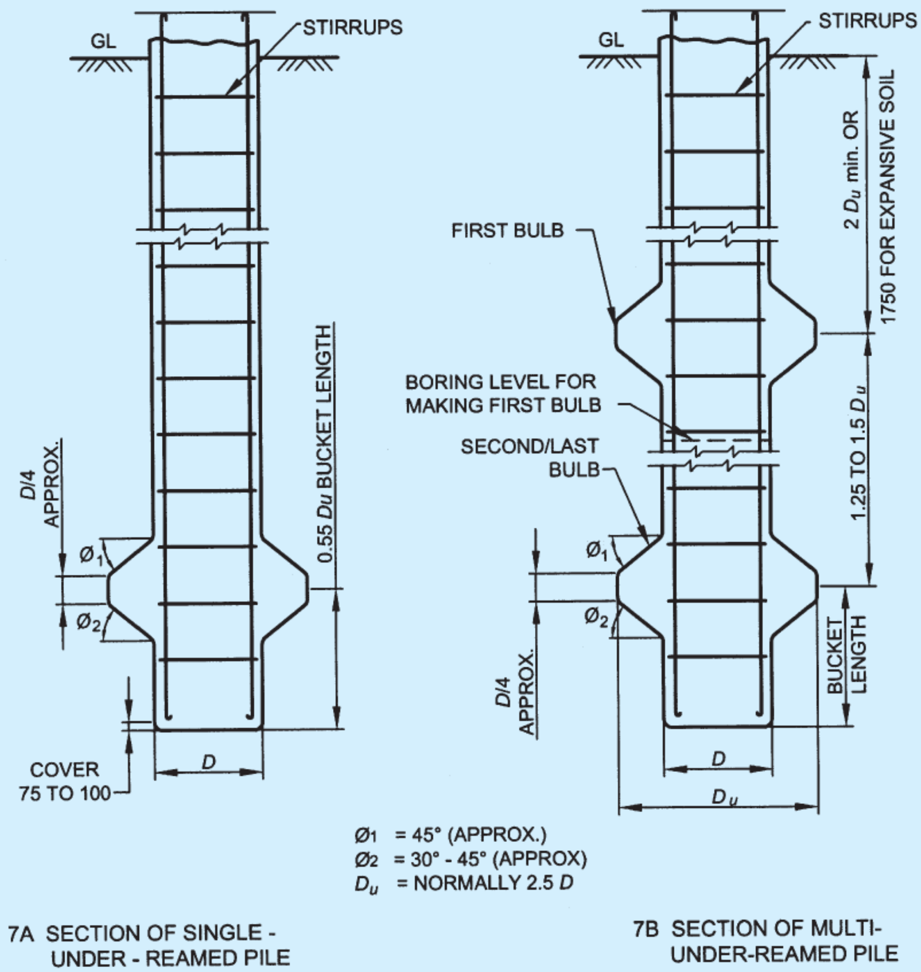
11.2 Design Considerations

11.2.1 *General*

Under-reamed pile foundation shall be designed in such a way that the load from the structure they support can be transmitted to the soil without causing failure of soil or failure of pile material and without causing settlement (differential or total) under permanent transient loading as may result in structural damage and/or functional distress (*see* Fig. 7).

11.2.1.1 In deep deposits of expansive soils the minimum length of piles, irrespective of any other considerations, shall be 3.5 m below ground level. If the expansive soil deposits are of shallow depth and overlying on non-expansive soil strata of good bearing or rock, piles of smaller length can also be provided. In recently filled up grounds or other strata or poor bearing the piles should pass through them and rest in good bearing strata.

11.2.1.2 The minimum stem diameter of under-reamed pile can be 200 mm upto 5 m depth in dry conditions, i.e. strata with low water table. The minimum stem diameter for piles upto 5 m depth in strata with high water table within pile depth, shall be 300 mm for normal under-reamed pile and 250 mm for compaction under-reamed pile. For piles of more than 5 m depth, the minimum diameter in two cases shall be 375 mm and 300 mm respectively. The minimum diameter of stem for strata consisting of harmful constituents, such as sulphates, should also be 375 mm.



All dimensions in millimetres.

FIG. 7 TYPICAL DETAILS OF BORED CAST *IN-SITU* UNDER-REAMED PILE FOUNDATION

11.2.1.3 The diameter of under-reamed bulbs may vary from 2 to 3 times the stem diameter, depending, upon the feasibility of construction and design requirements. In bored cast in-situ under-reamed piles and under-reamed compaction piles, the bulb diameter shall be normally 2.5 and 2 times the stem diameter respectively.

11.2.1.4 For piles of up to 300 mm diameter, the spacing of the bulbs should not exceed 1.5 times the diameter of the bulb. For piles of diameter greater than 300 mm, spacing can be reduced to 1.25 times the bulb diameter.

11.2.1.5 The topmost bulb should be at a minimum depth of two times the bulb diameter. In expansive soils it should also be not less than 2.75 m below ground level.

The minimum clearance below the underside of pile cap embedded in the ground and the bulb should be a minimum of 1.5 times the bulb diameter.

11.2.1.6 Under-reamed piles with more than one bulb are not advisable without ensuring their feasibility in strata needing stabilization of bore holes by drilling mud. The number of bulbs in the case of bored compaction piles should also not exceed one in such strata.

11.2.1.7 Under-reamed batter piles without lining in dry conditions, that is, strata with low water table can be constructed with batter not exceeding 15 degrees.

11.2.2 *Safe Load*

Safe load on a pile can be determined:

a) by calculating the ultimate load from soil properties and applying a suitable factor of safety as given in Annex K;
b) by load test on pile as good practice [6-2(20)]; and
c) from safe load tables.

11.2.2.1 *Load Test*

Provisions of **8.2.3.1.3** shall generally apply.

11.2.2.2 In the absence of detailed sub-soil investigations and pile load tests for minor and less important structures, a rough estimate of safe load on piles may be made from the Safe Load Table in accordance with good practice [6-2(27)].

NOTE- Safe loads as given in the above mentioned Table are symptomatic. Safe load carrying capacity of pile shall be worked out for the actual geotechnical data using **5.2.3.1** of good practice [6-2(27)], subjected to confirmation by initial pile load test in accordance with good practice [6-2(20)] and other provisions in **5.2.3.2** of good practice [6-2(27)].

11.2.3 *Negative Skin Friction or Dragdown Force*

Provisions of **8.2.4** shall generally apply subject to the condition that the under-reamed bulb is provided below the strata susceptible to negative skin friction.

11.2.4 *Structural Capacity*

Provisions of **8.2.5** shall generally apply except that the under-reamed pile stem is designed for axial capacity as a short column. Under-reamed piles under lateral loads and moments tend to behave more as rigid piles due the presence of bulbs and therefore the analysis can be done on rigid pile basis. Nominally reinforced long single bulb piles which are not rigid may be analyzed as per the method given in Annex H or as per other accepted methods.

11.2.5 *Spacing*

11.2.5.1 Generally the centre to centre spacing for bored cast *in-situ* under-reamed piles in a group should be two times the bulb diameter ($2D_u$). It shall not be less than 1.5 D_u. For under-grade beams, the maximum spacing of piles should generally not exceed 3 m. In under-reamed compaction piles, generally the spacing should not be less than 1.5 D_u. If the adjacent piles are of different diameter, an aveage value of bulb diameter should be taken for spacing.

11.2.6 *Group Efficiency*

For bored cast *in-situ* under-reamed piles at a usual spacing of $2D_u$, the group efficiency will be equal to the safe load of an individual pile multiplied by the number of piles in the group. For piles at a spacing of $1.5D_u$, the safe load assigned per pile in a group should be reduced by 10 percent.

In under-reamed compaction piles, at the usual spacing of $1.5D_u$, the group capacity will be equal to the safe load on an individual pile multiplied by the number of piles in the group.

11.2.7 *Transient and Overloading*

Provisions of **8.2.9** and **8.2.10** shall generally apply.

11.2.8 *Reinforcement*

11.2.8.1 The minimum area of longitudinal reinforcement (any type or grade) within the pile shaft shall be 0.4 percent of the sectional area calculate on the basis of outside area of the shaft or casing if used. Reinforcement is to be provided in full length and further a minimum of 3 bars of 10 mm diameter mild steel or three 8 mm diameter high strength steel bars shall be provided. Transverse reinforcement shall not be less than 6 mm diameter at a spacing of not more than the stem diameter or 300 mm, whichever is less.

In under-reamed compaction piles, a minimum number of four 12 mm diameter bars shall be provided. For piles of lengths exceeding 5 m and of 375 mm diameter, a minimum number of six 12 mm diameter bars shall be provided. For piles exceeding 400 mm diameter, a minimum number of six 12 mm diameter bars shall be provided. The circular stirrups for piles of lengths exceeding 5 m and diameter exceeding 375 mm shall be minimum 8 mm diameter bars.

For piles in earthquake prone areas, a minimum number of six bars of 10 mm diameter shall be provided. Also transverse reinforcement in the form of stirrups or helical should be at 150 mm centre-to-centre in top few meter depth.

11.2.8.2 The minimum clear cover over the longitudinal reinforcement shall be 40 mm. In aggressive environment of sulphates etc, it may be increased to 75 mm.

11.2.9 The design of pile cap and grade beams shall conform to the requirements specified in **8.2.12** and **8.2.13** respectively.

11.3 For detailed information on under-reamed piles regarding control of pile, installation, reference may be made to good practice [6-2(26)].

11.4 Non-destructive testing

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with good practice [6-2(24)].

12 TIMBER PILES

12.0 General

Timber piles find extensive use for compaction of soils and also for supporting as well as protecting water-front structures. The choice for using a timber pile shall be mainly governed by the site conditions, particularly the water-table conditions. Use of treated or untreated piles will depend upon the site conditions and upon whether the work is permanent or of temporary nature. The timber pile installed shall have its entire length embedded under water so that the pile may not get deteriorated. They have the advantages of being comparatively light for their strength and are easily handled. However, they will not withstand as hard driving as steel or concrete piles. Timber has to be selected carefully and treated where necessary for use as piles, as the durability and performance would considerably depend upon the quality of the material and relative freedom from natural defects. In coastal areas such as Kochi, Coconut/Palmyra tree trunks have been used effectively as timber piles.

12.1 Materials

12.1.1 *Timber*

The timber shall have the following characteristics:

a) Only structural timber shall be used for piles (*see* Part 6 ‘Structural Design’, Section 3 ‘Timber and Bamboo: 3A Timber’);
b) The length of an individual pile shall be :

    1) the specified length ± 300 mm for piles up to and including 12 m in length, and
    2) the specified length ± 600 mm for piles above 12 m in length.

c) The ratio of heartwood diameter to the pile butt diameter shall be not less than 0.8;
d) Piles to be used untreated shall have as little sapwood as possible.

12.2 Design Considerations

12.2.1 *General*

Timber piles shall be designed in such a way that the load from the structure can be transmitted to the sub-surface with adequate factor of safety against shear failure of sub-surface and without causing such settlement, (differential or total), which may result in structural damage and/or functional distress under permanent/transient loading. The pile shaft should have adequate structural capacity to withstand all loads (vertical, axial or otherwise) and moments which are to be transmitted to the subsoil and shall be designed according to Part 6 Structural Design, Section 3 Timber and Bamboo: 3A Timber.

12.2.2 *Adjacent structures*

When working near existing structures, care shall be taken to avoid damage to such structures. In case of deep excavations adjacent to piles, proper shoring or other suitable arrangement shall be made to guard against undesired lateral movement of soil.

12.2.3 *Pile Capcity – See* **8.2.3**.

12.2.4 *Structural Capacity*

The pile shall have the necessary structural strength to transmit the load imposed on it to the soil. Load tests shall be conducted on a single pile or preferably on a group of piles. For compaction piles, test should be done on a group of piles with their caps resting on the ground as good practice [6-2(20)]. If such test data is not available, the load carried by the pile shall be determined by the Engineering News formula (*see* Note).

NOTE – For timber piles, the load carried shall be determined by the Engineering News formula given below. Care shall be taken that while counting the number of blows, the head of the timber pile is not broomed or brushed and in case of interrupted driving counting shall be done after 300 mm of driving.

For piles driven with drop hammer,

$$P = \frac{160\,WH}{S + 25}$$

For piles driven with single-acting steam hammer,

$$P = \frac{160\,WH}{S + 2.5}$$

where

P = safe load on pile in kN,
W = weight of monkey in kN,
H = free fall of monkey in m, and
S = penetration of pile in mm to be taken as the average of the last three blows.

12.2.5 For detailed information on timber piles regarding spacing, classification, control of pile driving, storing and handling, reference may be made to good practice [6-2(28)].

13 OTHER FOUNDATIONS, SUB-STRUCTURES AND FOUNDATIONS FOR SPECIAL STRUCTURES

13.1 Pier Foundations

13.1.1 *Design Considerations*

13.1.1.1 *General*

The design of concrete piers shall conform to the requirements for columns specified in Part 6 'Structural Design', Section 5 'Concrete'. If the bottom of the pier is to be belled so as to increase its load carrying capacity, such bell shall be at least 300 mm thick at its edge. The sides shall slope at an angle of not less than 60° with the horizontal. The least permissible dimensions shall be 600 mm, irrespective of the pier being circular, square or rectangular. Piers of smaller dimensions if permitted shall be designed as piles (*see* **8** and **9**).

13.1.1.2 *Plain concrete piers*

The height of the pier shall not exceed 6 times the least lateral dimension. When the height exceeds 6 times the least lateral dimension, buckling effect shall be taken into account, but in no case shall the height exceed 12 times the least lateral dimension.

When the height exceeds 6 times the least lateral dimension, the deduction in allowable stress shall be given by the following formula:

$$f_c' = f_c\left(1.3 - \frac{H}{20D}\right)$$

where,

f_c' = reduced allowable stress,
f_c = allowable stress,
H = height of pier, and

D = least lateral dimension.

NOTE – The above provision shall not apply for piers where the least lateral dimension is 1.8 m or greater.

13.1.1.3 *Reinforced concrete piers*

When the height of the pier exceeds 18 times its least dimension, the maximum load shall not exceed :

$$P' = P\left(1.5 - \frac{H}{36D}\right)$$

where,

P' = permissible load ;
P = permissible load when calculated as axially loaded short column,
H = height of the pier measured from top of bell, if any, to the level of cut-off of pier; and
D = least lateral dimension.

13.2 Design and construction of machine foundations, diaphragm walls etc, shall be carried out in accordance with good practice [6-2(29)].

14 GROUND IMPROVEMENT

14.1 In poor and weak subsoils, the design of conventional shallow foundation for structures and equipment may present problems with respect to both sizing of foundation as well as control of foundation settlements. A viable alternative in certain situations, is to improve the subsoil to an extent such that the subsoil would develop an adequate bearing capacity and foundations constructed after subsoil improvement would have resultant settlements within acceptance limits. This method/technique is called ground improvement which is used to improve in-situ soil characteristics by improving its engineering performance as per the project requirement by altering its natural state, instead of having to alter the design in response to the existing ground limitations. The improvement is in terms of increase in bearing capacity, shear strength, reducing settlement and enhancing drainage facility, etc of soil.

14.2 For provisions with regard to necessary data to be collected to establish the need for ground improvement at a site; considerations for establishing need for ground improvement methods; selection of ground improvement techniques; equipment and accessories for ground improvement; control of ground improvement works and recording of data, reference shall be made to good practice [6-2(30)].

14.2.1 For provisions relating to ground improvement by reinforcing the ground using stone columns so as to meet the twin objective of increasing the bearing capacity with simultaneous reduction of settlements, reference shall be made to good practice [6-2(31)].

14.2.2 Whenever soft cohesive soil strata underlying a structure are unable to meet the basic requirements of safe bearing capacity and tolerable settlement, ground improvement is adopted to make it suitable for supporting the proposed structure. Both the design requirements that is shear strength and settlement under loading, can be fulfilled by consolidating the soil by applying a preload, if necessary, before the construction of the foundation. This consolidation of soil is normally accelerated with the use of vertical drains. For provisions relating to ground improvement by preconsolidation using vertical drains, reference shall be made to good practice [6-2(32)].

14.2.3 Use of suitable geo-synthetics/geo-textiles may be made in an approved manner for ground improvement, where applicable; *see also* accepted standards [6-2(33)].

ANNEX A

(*Clause* 3.2.1.1 *and* 7.2.2.4.2)

ASSESSMENT OF LIQUEFACTION POTENTIAL

A-1 LIQUEFACTION

One of the major cause of destruction during an earthquake is the failure of ground structure. The ground may fail due to fissures, abnormal or unequal movements, or loss of strength. The loss of strength may take place in fine cohesionless soils due to an increase in pore pressure, and this phenomena is termed as liquefaction.

Soil liquefaction occurs in loose, saturated cohesionless soil units (sands and silts) and sensitive clays. Saturated loose to medium dense fine cohesionless soils and low plasticity silts tend to densify and consolidate when subjected to cyclic shear deformations inherent with large seismic ground motions. Pore-water pressures within such layers increase as the soils are cyclically loaded, resulting in a decrease in vertical effective stress and shear strength. If the shear strength drops below the applied cyclic shear loadings, the layer is expected to transition to a semi fluid state until the excess pore-water pressure dissipates.

A-2 SUB SURFACE EXPLORATIONS FOR LIQUEFACTION ASSESSMENT

The liquefaction assessments may require following field and laboratory tests

a) Field test

- i) Standard penetration test (SPT);
- ii) Cone Penetration test (CPT);
- iii) Shear wave velocity (cross hole method)/Refraction microtremor (ReMi) survey; and
- iv) Ground water table.

b) Laboratory test

- i) Grain size analysis including hydrometer test;
- ii) Atterberg’s limits;
- iii) Density/Relative density and void ratio;
- iv) Cyclic triaxial tests; and
- v) Natural moisture content.

A-3 FACTORS AFFECTING LIQUEFACTION SUSCEPTIBILITY

Following factors affect liquefaction susceptibility of soil:

a) *Geologic age and origin* — Liquefaction potential decreases with increasing age of a soil deposits. Pre-holocene age soil deposits generally do not liquefy. Though liquefaction has occasionally been observed in Pleistocene-age deposits.

b) *Fine contents and plasticity Index* — Soil having greater than 15 percent (by weight) finer than 0.005 mm, a liquid limit (LL) greater than 35 percent or an in-situ water content less than 0.9 times the liquid limit [that is, w > 0.9 (LL)] do not liquefy.

c) *Saturation* — At least 80 to 85 percent saturation deemed to be necessary condition for soil liquefaction.

d) *Depth below ground surface* — Shallow foundations are generally not affected if liquefaction occurs more than 15 m below the ground surface.

e) *Soil penetration resistance* — A normalized Standard Penetration Test (SPT) value of 30 is considered to be the threshold value above which liquefaction will not occur. Also, it is considered that no liquefaction is possible if normalized Cone Penetration Test (CPT) cone resistance, q_{cl},is larger than 15 MPa.

If three or more of the above criteria indicate that liquefaction is not likely, the potential for liquefaction may be considered to be small enough that a formal liquefaction potential analysis is not required.

A-4 Evaluation of Liquefaction Potential

The most basic procedure used in engineering practice for assessment of site liquefaction potential is that of the “Simplified Procedure”. The procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude).

Simplified procedure comprises the following four steps:

1) Identify the potentially liquefiable layers to be analyzed.

2) Calculate the shear stress required to cause liquefaction (resisting forces).

   Based on the characteristics of the potentially liquefiable layers (for example, fines content, normalized standardized blowcount), the critical (cyclic) stress ratio (CSR_L) can be determined using the graphical as given in specialist literature, such as U.S. EPA, Seismic Design Guidance.

NOTE- This determination is typically based on an earthquake of magnitude 7.5. If the design earthquake is of a different magnitude, or if the site is not level, the CSR_L will need to be corrected as follows:

$$CSR_{L(M-M)} = CSR_{L(M=7.5)} \cdot k_M \cdot k_\sigma \cdot k_\alpha$$

where

$CSR_{L(M-M)}$ = corrected critical stress ratio resisting liquefaction;

$CSR_{L(M=7.5)}$ = critical stress ratio resisting liquefaction for a magnitude 7.5 earthquake;

k_M = magnitude correction factor;

k_σ = correction factor for stress levels exceeding 1 tsf; and

k_α = correction factor for the driving static shear stress of sloping ground conditions exist at the facilities. Special expertise is required for evaluation of liquefaction resistance beneath ground sloping more than six percent.

The k-values are available from tabled or graphical sources in the referenced materials.

3) Calculation of the design earthquake's effect on the critical zone (driving force).

The following equation can be used:

$$CSR_{EQ} = 0.65\left(\frac{a_{max,z}}{g}\right) r_d \left(\frac{\sigma_o}{\sigma_o^{'}}\right)$$

where

CSR_{EQ} = equivalent uniform cyclic stress ratio induced by the earthquake;

σ_o = total vertical overburden stress;

$\sigma_o^{'}$ = effective vertical overburden stress;

$a_{max,z}$ = the maximum horizontal ground acceleration; and

$= (a_{max})(r_d)$

g = the acceleration of gravity.

where

$a_{max,z}$ = the maximum horizontal ground acceleration; and

a_{max} = peak ground surface acceleration; and

r_d = empirical stress reduction factor.

$$r_{\mathrm{d}} = \frac{a_{\mathrm{max@depthD}}}{\sigma_{\mathrm{o@depth\ D}}\left(\dfrac{a_{\mathrm{max@surface}}}{g}\right)}$$

4) Calculate the factor of safety against liquefaction (resisting force divided by driving force).

$$FS_{\mathrm{L}} = \frac{CSR_{\mathrm{L(M-M)}}}{CSR_{\mathrm{EQ}}} \geq 1.00$$

where

FS_{L}	=	factor of safety against liquefaction;
$CSR_{\mathrm{L(M-M)}}$	=	shear stress ratio required to cause liquefaction; and
CSR_{EQ}	=	equivalent uniform cyclic stress ratio.

NOTE — The correction factors can be obtained from different sources, such as the 1995, U.S. EPA, Seismic Design Guidance, or the summary report from 1996 and 1998 NCEER/NSF Liquefaction Workshops. The U.S. EPA document tends to be somewhat more conservative for earthquake with a magnitude less than 6.5. In 1999, I.M. Idriss proposed yet a different method for calculating the empirical stress reducation factor (r_{d}), which was less conservative than the method included in the U.S. EPA guidance, but more conservative than the method included in the NCEER method. Designers should select correction factors based on site-specific circumstances and include documentation explaining their choices in submittals to Ohio EPA.

ANNEX B

(*Clause* 7.4.1.11 *and* C-3.3)

DETERMINATION OF MODULUS OF ELASTICITY (E_s) AND POISSONS'S RATIO (μ)

B-1 DETERMINATION OF MODULUS OF ELASTICITY (E_s)

B-1.1 The modulus of elasticity is a function of composition of the soil, its void ratio, stress history and loading rate. In granular soils it is a function of the depth of the strata, while in cohesive soil it is markedly influenced by the moisture content. Due to its great sensitivity to sampling disturbance, accurate evaluation of the modulus in the laboratory is extremely difficult. For general cases, therefore, determination of the modulus may be based on field tests (**B-2**). Where properly equipped laboratory and sampling facility is available, E_s may be determined in the laboratory (*see* **B-3**).

B-2 FIELD DETERMINATION

B-2.1 The value of E_s shall be determined from plate load test in accordance with good practice [6-2(12)].

$$E_s = qB\frac{\left(1-\mu^2\right)}{s}I_w$$

where

q = intensity of contact pressure,
B = least lateral dimension of test plate,
s = settlement,
μ = Poisson's ratio, and
I_w = influence ratio
= 0.82 for a square plate.

NOTE- While this procedure may be adequate for light or less important structures under normal conditions, relevant laboratory tests or field tests are essential in the case of unusual soil types and for all heavy and important structures. Plate load test, though useful in obtaining the necessary information about the soil with particular reference to design of foundation has some limitations. The test results reflect only the character of the soil located within a depth of less than twice the width of the bearing plate. Since the foundations are generally larger than the test plates, the settlement and shear resistance will depend on the properties of a much thicker stratum. Moreover this method does not give the ultimate settlements particularly in case of cohesive soils. Thus the results of the test are likely to be misleading, if the character of the soil changes at shallow depths, which is not uncommon. A satisfactory load test should, therefore, include adequate soil exploration {*see* good practice [6-2(2)]} with due attention being paid to any weaker stratum below the level of the footing. Another limitation is the concerning of the effect of size of foundation. For clayey soils the bearing capacity (from shear consideration) for a larger foundation is almost the same as that for the smaller test plate. But in dense sandy soils the bearing capacity increases with the size of the foundation. Thus tests with smaller size plate tend to give conservative values in dense sandy soils. It may, therefore, be necessary to test with plates of at least three sizes and the bearing capacity results extrapolated for the size of the actual foundation (minimum dimensions in the case of rectangular footings).

B-2.1.1 The average value of E_s shall be based on a number of plate load tests carried out over the area, the number and location of the tests, depending upon the extent and importance of the structure.

B-2.1.2 *Effect of Size*

In granular soils the value of E_s corresponding to the size of the raft shall be determined as follows :

$$E_s = E_p \frac{B_f}{B_p}\left[\frac{B_f + B_p}{2B_f}\right]^2$$

Where, B_f, B_p represent sizes of foundation and plate and E_p is the modulus determined by the plate load test.

B-2.2 For stratified deposits or deposits with lenses of different materials, results of plate load test will be unreliable and static cone penetration tests may be carried out to determine E_s.

B-2.2.1 Static cone penetration tests shall be carried out in accordance with accepted standard [6-2(22)]. Several tests shall be carried out at regular depth intervals up to a depth equal to the width of the raft and the results plotted to obtain an average value of E_s.

B-2.2.2 The value of E_s may be determined from the following relationship :

$$E_s = 2\ C_{kd}$$

Where,

C_{kd} = cone resistance in kN/m^2.

B-3 LABORATORY DETERMINATION OF E_S

B-3.1 The value of E_s shall be determined by conducting triaxial test in the laboratory in accordance with accepted standard [6-2(34)] on samples collected with least disturbances.

B-3.2 In the first phase of the triaxial test, the specimen shall be allowed to consolidate fully under an all-round confining pressure equal to the vertical effective overburden stress for the specimen in the field. In the second phase, after equilibrium has been reached, further drainage shall be prevented and the deviator stress shall be increased from zero value to the magnitude estimated for the field loading condition. The deviator stress shall then be reduced to zero and the cycle of loading shall be repeated.

B-3.3 The value of E_s shall be taken as the tangent modulus at the stress level equal to one-half the maximum deviator stress applied during the second cycle of loading.

ANNEX C
(*Clause* 7.4.1.11)

DETERMINATION OF MODULUS OF SUBGRADE REACTION

C-1 GENERAL

C-1.1 The modulus of subgrade reaction (*k*) as applicable to the case of load through a plate of size 300 mm x 300 mm or beams 300 mm wide on the soils is given in Table 7 for cohesionless soils and in Table 8 for cohesive soils. Unless more specific determination of k is done (*see* **C-2** and **C-3**) these value may be used for design of raft foundation in cases where the depth of the soil affected by the width of the footing may be considered isotropic and the extra-polation of plate load test results is valid.

Table 7 Modulus of Subgrade Reaction (*k*) for Cohesionless Soils
(*Clause C*-1.1)

Soil Characteristic		[1] Modulus of Subgrade Reaction (*k*) in kN/m^3	
Relative Density	Standard Penetration Test Value (N) (Blows per 300 mm)	For Dry or Moist State	For Submerged State
(1)	(2)	(3)	(4)
Loose	< 10	15 000	9 000
Medium	10 to 30	15 000 to 47 000	9 000 to 29 000
Dense	30 and over	47 000 to180 000	29 000 to 108 000

[1] The above values apply to a square plate 300 mm x 300 mm or beams 300 mm wide.

Table 8 Modulus of Subgrade Reaction (*k*) for Cohesive Soils
(*Clause C*-1.1)

Soil Characteristic		[1] Modulus of Subgrade Reaction (*k*) in kN/m^3
Consistency	Unconfined Compressive Strength, kN/m^2	
(1)	(2)	(3)
Stiff	100 to 200	27 000
Very Stiff	200 to 400	27 000 to 54 000

Hard	400 and over	54 000 to 108 000

[1] The values apply to a square plate 300 mm x 300 mm. The above values are based on the assumption that the average loading intensity does not exceed half the ultimate bearing capacity.

C-2 FIELD DETERMINATION

C-2.1 In cases where the depth of the soil affected by the width of the footing may be considered as isotropic, the value of *k* may be determined in accordance with accepted standard [6-2(35)]. The test shall be carried out with a plate of size not less than 300 mm.

C-2.2 The average value of *k* shall be based on a number of plate load tests carried out over the area, the number and location of the tests depending upon the extent and importance of the structure.

C-3 LABORATORY DETERMINATION

C-3.1 For stratified deposits or deposits with lenses of different materials, evaluation of *k* from plate load test will be unrealistic and its determination shall be based on laboratory tests [*see* {6-2(4)}].

C-3.2 In carrying out the test, the continuing cell pressure may be so selected as to be representative of the depth of the average stress influence zone (about 0.5 B to B).

C-3.3 The value of *k* shall be determined from the following relationship :

$$k = 0.65 \times \sqrt[12]{\left(\frac{E_s B^4}{EI}\right)} \cdot \frac{E_s}{\left(1-\mu^2\right)} \cdot \frac{1}{B}$$

where

E_s = modulus of elasticity of soil (*see* Annex B)
E = Young's modulus of foundation material,
μ = Poisson's ratio of soil,
I = moment of inertia of the foundation, and
B = width of the footing

C-4 CALCULATIONS

When the structure is rigid (*see* Annex D), the average modulus of subgrade reaction may also be determined as follows:

$$k_s = \frac{Average\,contact\ pressure}{Average\,settlement\,of\ the\,raft}$$

ANNEX D

(*Clauses* 7.4.4.1, 7.4.4.2 & 7.5.2.1.1 *and* C-4)

RIGIDITY OF SUPERSTRUCTURE AND FOUNDATION

D-1 DETERMINATION OF THE RIGIDITY OF THE STRUCTURE

D-1.1 The flexural rigidity EI of the structure of any section may be estimated according to the relation given below (*see also* Fig. 8)

$$EI = \frac{E_1 I_i b^2}{2H^2} + \sum E_2 I_b \left[1 + \frac{\left(I_u' + I_l'\right) b^2}{\left(I_b' + I_u' + I_f'\right) l^2}\right]$$

where

E_1 = modulus of elasticity of the infilling material (wall material) in kN/m^2,
I_i = moment of inertia of the infilling in m^4,
b = length or breadth of the structure in the direction of bending in m,
H = total height of the infilling in m,
E_2 = modulus of elasticity of the frame material in kN/m^2 ,
I_b = moment of inertia of the beam in m^4,

$$I_u' = \frac{I_u}{h_u}$$

$$I_l' = \frac{I_l}{h_i}$$

$$I_b' = \frac{I_b}{l}$$

l = spacing of the columns in m,
h_u = length of the upper column in m,
h_i = length of the lower column in m,

$$I_f' = \frac{I_f}{l}$$

I_u = moment of inertia of the upper column in m^4,
I_l = moment of inertia of the lower column in m^4, and

I_f = moment of inertia of the foundation beam or raft in m^4 .

NOTE – The summation is to be done over all the storeys including the foundation beam or raft. In the case of the foundation, I'_f replaces I'_b and I_l becomes zero, whereas for the topmost beam I'_u becomes zero.

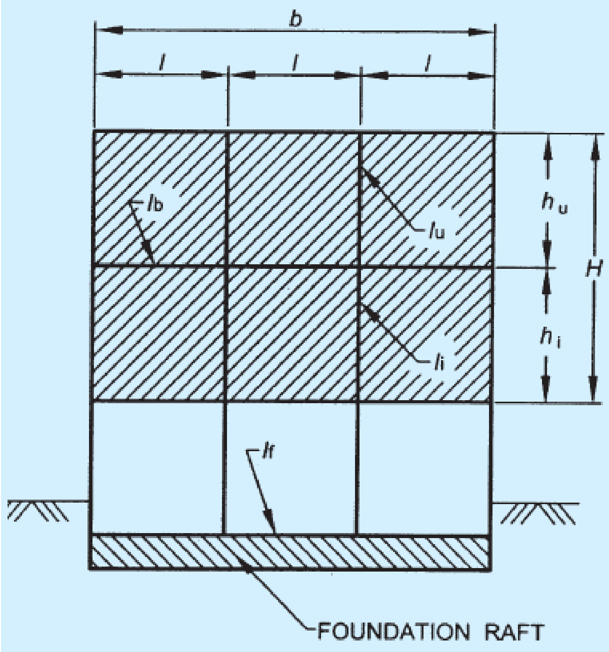


FIG. 8 DETERMINATION OF RIGIDITY OF STRUCTURE

D-2 RELATIVE STIFFNESS FACTOR, *K*

D-2.1 Whether a structure behaves as rigid or flexible depends on the relative stiffness of the structure and the foundation soil. This relation is expressed by the relative stiffness factor *K* given below :

a) For the whole structure, $K = \dfrac{EI}{E_s b^3 a}$

b) For rectangular rafts, $K = \dfrac{E}{12E_s}\left(\dfrac{d}{b}\right)^3$

c) For circular rafts, $K = \dfrac{E}{12E_s}\left(\dfrac{d}{2R}\right)^3$

where

EI = flexural rigidity of the structure over the length (a) in kN/m^2,
E_s = modulus of compressibility of the foundation soil in kN/m^2,
b = length of the section in the bending axis in m,
a = length perpendicular to the section under investigation in m,
d = thickness of the raft or beam in m, and
R = radius of the raft in m.

D-2.1.1 For *K* > 0.5, the foundation may be considered as rigid [*see* **7.4.4.1** (a)].

D-3 DETERMINATION OF CRITICAL COLUMN SPACING

D-3.1 Evaluation of the characteristics λ is made as follows :

$$\lambda = \sqrt[4]{\left(\frac{kB}{4E_c I}\right)}$$

where

k = modulus of subgrade reaction in kN/m^3 for footing of width B in m (*see* Annex B)
B = width of raft B in m,
E_c = modulus of elasticity of concrete in kN/m^2, and
I = moment of inertia of raft in m^4.

ANNEX E

(*Clause* 7.4.4.1)

CALCULATION OF PRESSURE DISTRIBUTION BY CONVENTIONAL METHOD

E-1 DETERMINATION OF PRESSURE DISTRIBUTION

E-1.1 The pressure distribution (q) under the raft shall be determined by the following formula :

$$q = \frac{Q}{A} \pm \frac{Qe'_{\mathrm{y}}}{I'_{\mathrm{x}}} y \pm \frac{Qe'_{\mathrm{x}}}{I'_{\mathrm{y}}} x$$

where

Q = total vertical load on the raft,
A = total area of the raft,
$e'_{\mathrm{x}}, e'_{\mathrm{y}}$, $I'_{\mathrm{x}}, I'_{\mathrm{y}}$ = eccentricities and moments of inertia about the principal axes through the centroid of the section, and
x, y = co-ordinates of any given point on the raft with respect to the *x* and *y* axes passing through the centroid of the area of the raft.

$I'_{\mathrm{x}}, I'_{\mathrm{y}}, e'_{\mathrm{x}}, e'_{\mathrm{y}}$ may be calculated from the following equations:

$$l'_{\mathrm{x}} = l_{\mathrm{x}} - \frac{l_{\mathrm{xy}}^2}{l_{\mathrm{y}}}$$

$$l'_{\mathrm{y}} = l_{\mathrm{y}} - \frac{l_{\mathrm{xy}}^2}{l_{\mathrm{x}}}$$

$$e'_{\mathrm{x}} = e_{\mathrm{x}} - \frac{l_{\mathrm{xy}}}{l_{\mathrm{x}}} e_{\mathrm{y}}$$

$$e'_{\mathrm{y}} = e_{\mathrm{y}} - \frac{l_{\mathrm{xy}}}{l_{\mathrm{y}}} e_{\mathrm{x}}$$

where

$I_{\mathrm{x}}, I_{\mathrm{y}}$ = moment of inertia of the area of the raft respectively about the *x* and *y* axes through the centroid,
$I_{\mathrm{xy}} = \int xy \cdot dA$ for the whole area about *x* and *y* axes through the centroid,
and

e_x, e_y = eccentricities in the x and y directions of the load from the centroid.

For a rectangular raft, the equation simplifies to :

$$q = \frac{Q}{A}\left(1 \pm \frac{12e_y y}{b^2} \pm \frac{12e_x x}{a^2}\right)$$

where

a and *b* = the dimensions of the raft in the x and y directions respectively.

NOTE – If one or more of the values of (*q*) are negative as calculated by the above formula, it indicates that the whole area of foundation is not subject to pressure and only a part of the area is in contact with the soil, and the above formula will still hold good, provided the appropriate values of I_x, I_y, I_{xy}, e_x and e_y, are used with respect to the area in contact with the soil instead of the whole area.

ANNEX F
(*Clause* 7.4.4.2)

CONTACT PRESSURE DISTRIBUTION AND MOMENTS BELOW FLEXIBLE FOUNDATION

F-1 CONTACT PRESSURE DISTRIBUTION

F-1.1 The distribution of contact pressure is assumed to be linear with the maximum value attained under the columns and the minimum value at mid span.

F-1.2 The contact pressure for the full width of the strip under an interior column load located at a point *i* can be determined as (*see* Fig. 9 A) :

$$p_\mathrm{i} = \frac{5P_\mathrm{i}}{\bar{l}} + \frac{48M_\mathrm{i}}{\left(\bar{l}\right)^2}$$

where

$\bar{l}$ = average length of adjacent span (m),
P_i = column load in t at point *i*, and
M_i = moment under an interior columns loaded at *i*.

F-1.3 The minimum contact pressure for the full width of the strip at the middle of the adjacent spans can be determined as (*see* Fig. 9A and 9B).

$$p_\mathrm{ml} = 2P_\mathrm{i}\left(\frac{l_\mathrm{r}}{l_\mathrm{l}\bar{l}}\right) - p_i\left(\frac{\bar{l}}{l_\mathrm{l}}\right)$$

$$p_\mathrm{mr} = 2P_\mathrm{i}\left(\frac{l_\mathrm{l}}{l_\mathrm{r}\bar{l}}\right) - p_i\left(\frac{\bar{l}}{l_\mathrm{r}}\right)$$

$$p_\mathrm{m} = \frac{p_\mathrm{ml} + p_\mathrm{mr}}{2}$$

where, l_r, l_l as shown in Fig. 9A.

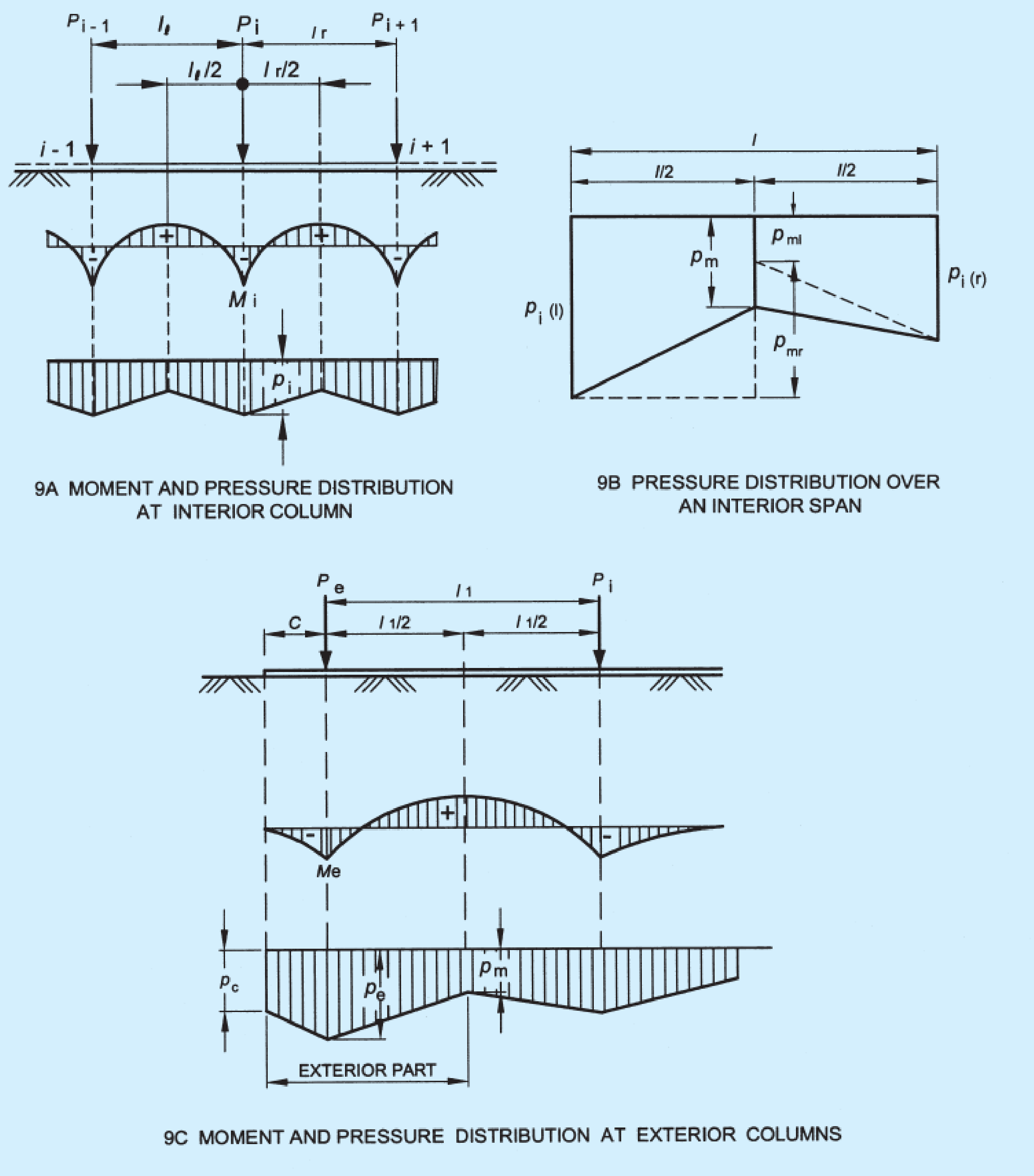


FIG. 9 MOMENT AND PRESSURE DISTRIBUTION AT COLUMNS

F-1.4 If **F-2.3**(a) governs the moment under the exterior columns, contact pressures under the exterior columns and at end of strip can be determined as (*see* Fig. 9C) :

$$p_{\mathrm{e}} = \frac{4P_{\mathrm{e}} + \dfrac{6M_{\mathrm{e}}}{C} - p_{\mathrm{m}} l_1}{C + l_1}$$

$$p_{\mathrm{c}} = -\frac{3M_{\mathrm{e}}}{C^2} - \frac{p_{\mathrm{e}}}{2}$$

where P_e, p_m, M_e, l_1, C are as shown in Fig. 9C.

F-1.5 If **F-2.3**(b) governs the moment under the exterior columns, the contact pressures are determined as (*see* Fig. 9C) :

$$p_{\mathrm{e}} = p_{\mathrm{c}} \frac{4P_{\mathrm{e}} - p_{\mathrm{m}} l_1}{4C + l_1}$$

F-2 BENDING MOMENT DIAGRAM

F-2.1 The bending moment under an interior column located at *i* (*see* Fig. 9A) can be determined as :

$$M_{\mathrm{i}} = -\frac{P_{\mathrm{i}}}{4\lambda}\left(0.24\lambda\bar{l} + 0.16\right)$$

F-2.2 The bending moment at mid span is obtained as (*see* Fig. 9A) :

$$M_{\mathrm{m}} = M_{\mathrm{o}} + M_{\mathrm{i}}$$

M_o = moment of simply supported beam

$$= \frac{l^2}{48}\left[p_{\mathrm{i}}(l) + 4\overline{p_{\mathrm{m}}} + p_{\mathrm{i}}(r)\right]$$

M_i = average of negative moments M_i at each end of the bay.

where l, $p_i(l)$, $p_i(r)$, $\overline{p_{\mathrm{m}}}$ are as shown in Fig. 9B.

F-2.3 The bending moment M_e under exterior columns can be determined as the least of (*see* Fig. 9C) :

a) $M_{e1} = -\frac{P_e}{4\lambda}\left(0.13\lambda l_1 + 1.06\lambda C - 0.50\right)$

b) $M_{e2} = -\frac{\left(4P_e - p_m l_1\right)}{\left(4C + l_1\right)}\frac{C^2}{2}$

ANNEX G
(*Clause* 7.3.4.2)

FLEXIBLE FOUNDATION – GENERAL CONDITION

G-1 CLOSED FORM SOLUTION OF ELASTIC PLATE THEORY

G-1.1 For a flexible raft foundation with non-uniform column spacing and load intensity, solution of the differential equation governing the behaviour of plates on elastic foundation (Winkler Type) gives radial moment (M_r) tangential moment (M_t) and deflection (w) at any point by the following expressions :

$$M_r = -\frac{P}{4}\left[Z_4\left(\frac{r}{L}\right) - (1-\mu)\frac{Z_3'\left(\frac{r}{L}\right)}{\left(\frac{r}{L}\right)} \right]$$

$$M_t = -\frac{P}{4}\left[\mu Z_4\left(\frac{r}{L}\right) + (1-\mu)\frac{Z_3'\left(\frac{r}{L}\right)}{\left(\frac{r}{L}\right)} \right]$$

$$w = \frac{PL^2}{4D} \cdot Z_3\left(\frac{r}{l}\right)$$

where

P = column load,
r = distance of the point under investigation from column load along radius, and
L = radius of effective stiffness

$$= \left(\frac{D}{k}\right)^{1/4}$$

where

k = Modulus of subgrade reaction for footing of width B,
D = flexural rigidity of the foundation,

$$P = \frac{Et^2}{12(1-\mu)^2}$$

t = raft thickness,
E = Modulus of elasticity of the foundation material,
μ = Possion's ratio of the foundation material, and

$$\left.\begin{array}{l} Z_3\left(\dfrac{r}{L}\right), \\ Z_3'\left(\dfrac{r}{L}\right), \\ Z_4\left(\dfrac{r}{L}\right) \end{array}\right\}$$ = functions of shear, moment and deflection (*see* Fig. 10)

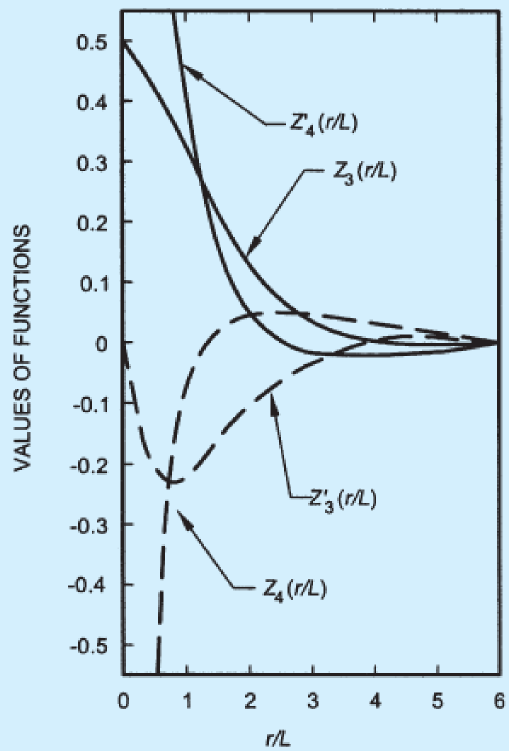


FIG. 10 FUNCTIONS FOR SHEAR MOMENT AND DEFLECTION

G-1.2 The radial and tangential moments can be converted to rectangular co-ordinates:

$$M_x = M_r \cos^2\phi + M_t \sin^2\phi$$
$$M_y = M_r \sin^2\phi + M_t \cos^2\phi$$

where

Φ = angle with x-axis to the line jointing origin to the point under consideration.

G-1.3 The shear Q per unit width of raft can be determined by :

$$Q = -\frac{P}{4L} Z_4^{'}\left(\frac{r}{L}\right)$$

$Z_4^{'}\left(\frac{r}{L}\right)$ = function for shear (*see* Fig. 10).

G-1.4 When the edge of the raft is located within the radius of influence, the following corrections are to be applied. Calculate moments and shears perpendicular to the edge of the raft within the radius of influence, assuming the raft to be infinitely large. Then apply opposite and equal moments and shears on the edge of the mat. The method for beams on elastic foundation may be used.

G-1.5 Finally, all moments and shears calculated for each individual column and wall are superimposed to obtain the total moment and shear values.

ANNEX H
(*Clauses* 8.2.4.1.1)

LOAD CARRYING CAPACITY OF PILES - STATIC ANALYSIS

H-1 PILES IN GRANULAR SOILS

H-1.1 The ultimate load capacity (Q_u) of piles, in kN, in granular soils is given by the following formula:

$$Q_u = A_p(\tfrac{1}{2} D \gamma N_\gamma + P_D N_q) + \sum_{i=1}^{n} K_i P_{Di} \tan \delta_i A_{si} \qquad \ldots\ldots\ldots(1)$$

The first term gives end bearing resistance and the second term gives skin friction resistance.

where

A_p = cross-sectional area of pile tip, in m^2;
D = diameter of pile shaft, in m;
γ = effective unit weight of the soil at pile tip, in kN/m^3;
N_γ and N_q = bearing capacity factors depending upon the angle of internal friction, ϕ at pile tip;
P_D = effective overburden pressure at pile tip, in kN/m^2 (*see* Note 5);
$\sum_{i=1}^{n}$ = summation for layers 1 to *n* in which pile is installed and which contribute to positive skin friction;
K_i = coefficient of earth pressure applicable for the *i*th layer (*see* Note 3);
P_{Di} = effective overburden pressure for the *i*th layer, in kN/m^2;
δ_i = angle of wall friction between pile and soil for the *i*th layer; and
A_{si} = surface area of pile shaft in the *i*th layer, in m^2.

NOTES

1 N_γ factor can be taken for general shear failure according to IS 6403.

2 N_q factor will depend on the nature of soil, type of pile, the *L*/*B* ratio and its method of construction. The values applicable for bored piles are given in Fig 11.

3 K_i, the earth pressure coefficient depends on the nature of soil strata, type of pile, spacing of piles and its method of construction. For bored piles in loose to dense sand with ϕ varying between 30° and 40°, K_i values in the range of 1 to 1.5 may be used.

4 δ, the angle of wall friction may be taken equal to the friction angle of the soil around the pile shaft.

5 In working out pile capacity by static formula, the maximum effective overburden at the pile tip should correspond to the critical depth, which may be taken as 15 times the diameter of the pile shaft for $\phi \leq 30^\circ$ and increasing to 20 times for $\phi \geq 40^\circ$.

6 For piles passing through cohesive strata and terminating in a granular stratum, a penetration of at least twice the diameter of the pile shaft should be given into the granular stratum.

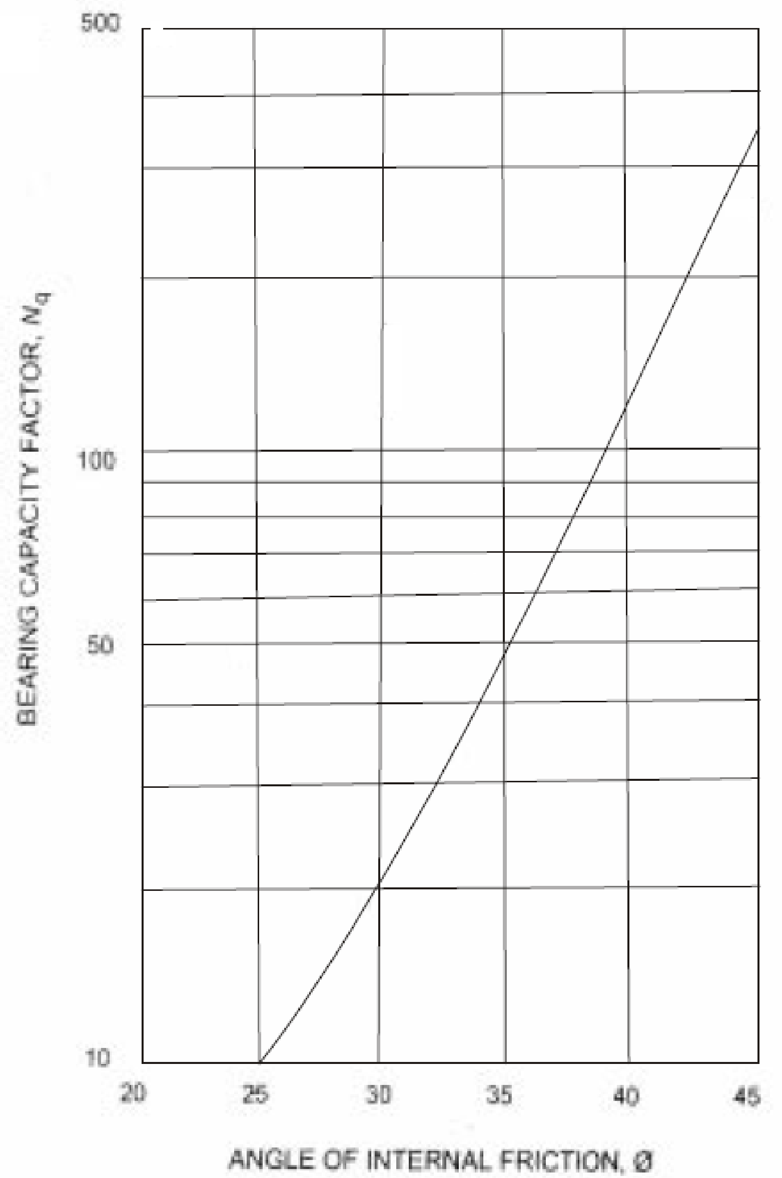


Fig. 11 BEARING CAPACITY FACTOR, N_q FOR BORED PILES

H-2 PILES IN COHESIVE SOILS

The ultimate load capacity (Q_u) of piles, in kN, in cohesive soils is given by the following formula:

$$Q_u = A_p N_c c_p + \sum_{i=1}^{n} \alpha_i \ c_i \ A_{si} \qquad \ldots\ldots\ldots\ldots\ldots(2)$$

The first term gives end bearing resistance and the second term gives the skin friction resistance.

where

A_p	=	cross-sectional area of pile tip, in m^2;
N_c	=	bearing capacity factor, may be taken as 9;
c_p	=	average cohesion at pile tip, in kN/m^2;
$\sum_{i=1}^{n}$	=	summation for layers 1 to *n* in which pile is installed and which contribute to positive skin friction;
α_i	=	adhesion factor for the *i*th layer depending on the consistency of soil, (*see* Note);
c_i	=	average cohesion for the *i*th layer, in kN/m^2; and
A_{si}	=	surface area of pile shaft in the *i*th layer, in m^2.

NOTE— The value of adhesion factor, α_i depends on the undrained shear strength of the clay and may be obtained from Fig. 12.

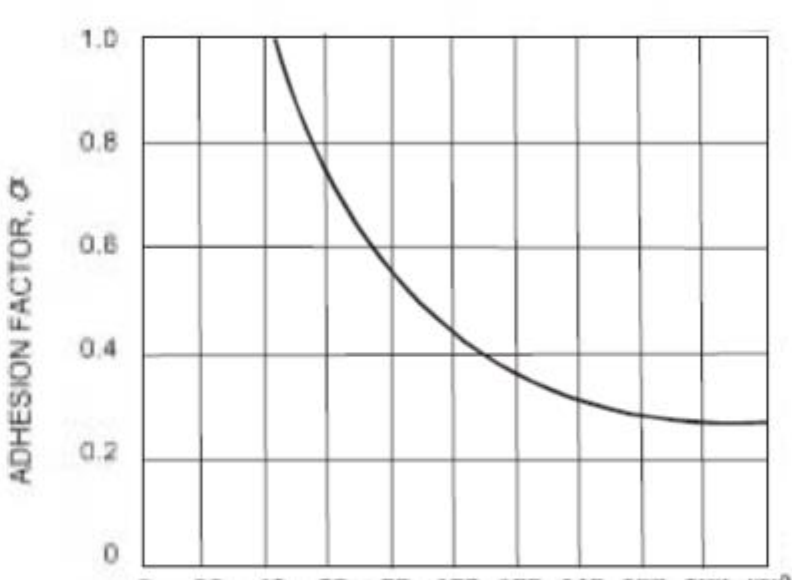


FIG. 12 VARIATION OF α WITH C_u

H-3 USE OF STATIC CONE PENETRATION DATA

H-3.1 When static cone penetration data are available for the entire depth, the following correlation may be used as a guide for the determination of ultimate load capacity of a pile.

H-3.2 Ultimate end bearing resistance (q_u), in kN/m^2, may be obtained as:

$$q_u = \frac{\frac{q_{c0} + q_{c1}}{2} + q_{c2}}{2}$$

where

q_{c0} = average static cone resistance over a depth of 2*D* below the pile tip, in kN/m^2;
q_{c1} = minimum static cone resistance over the same 2*D* below the pile tip, in kN/m^2;
q_{c2} = average of the envelope of minimum static cone resistance values over the length of pile of 8*B* above the pile tip, in kN/m^2; and
D = diameter of pile shaft.

H-3.3 Ultimate skin friction resistance can be approximated to local side friction (f_s), in kN/m^2, obtained from static cone resistance as given in Table 9 below:

Table 9 Side Friction for Different Soil Types
(*Clause* H-3.3)

Sl. No.	**Type of Soil**	**Local Side Friction, f_s** kN/m^2
(1)	(2)	(3)
i)	q_c less than 1 000 kN/m^2	$q_c/30 < f_s < q_c/10$
ii)	Clay	$q_c/25 < f_s < 2q_c/25$
iii)	Silty clay and silty sand	$q_c/100 < f_s < q_c/25$
iv)	Sand	$q_c/100 < f_s < q_c/50$
v)	Coarse sand and gravel	$q_c/100 < f_s < q_c/150$

where q_c = cone resistance, in kN/m^2.

H-3.4 The correlation between standard penetration resistance, *N* (blows/30 cm) and static cone resistance, q_c, in kN/m^2 as given in Table 10 may be used for working out the end bearing resistance and skin friction resistance of piles. This correlation should only be taken as a guide and should preferably be established for a given site as they can vary substantially with the grain size, Atterberg limits, water table, etc.

Table 10 Co-Relation between *N* and q_c for Different Types of Soil
(*Clause* H-3.4)

Sl. No.	Type of Soil	q_c/N
(1)	(2)	(3)
i)	Clay	150 – 200
ii)	Silts, sandy silts and slightly cohesive silt-sand mixtures	200 – 250
iii)	Clean fine to medium sand and slightly silty sand	300 – 400
iv)	Coarse sand and sands with little gravel	500 – 600
v)	Sandy gravel and gravel	800 – 1 000

H-4 USE OF STANDARD PENETRATION TEST DATA FOR COHESIONLESS SOIL

H-4.1 The correlation suggested by Meyerhof using standard penetration resistance, *N* in saturated cohesionless soil to estimate the ultimate load capacity of bored pile is given below. The ultimate capacity of pile (Q_u), in kN, is given as:

$$Q_u = 13\, N \frac{L}{B} A_p + \frac{\overline{N} A_s}{0.50} \quad \text{.............(3)}$$

The first term gives end bearing resistance and the second term gives frictional resistance.

where

N = average *N* values at pile tip;
L = length of penetration of pile in the bearing strata, in m;
B = diameter or minimum width of pile, in m;
A_p = cross-sectional area of pile tip, in m^2;
$\overline{N}$ = average *N* along the pile shaft; and
A_s = surface area of pile shaft, in m^2.

NOTE —The end bearing resistance should not exceed 130 NA_p

H-4.2 For non-plastic silt or very fine sand the equation has been modified as:

$$Q_u = 10\ N\ \frac{L}{B} A_p + \frac{\overline{N}\, A_s}{0.60} \qquad \text{…………..(4)}$$

The meaning of all terms is same as for equation 3.

H-5 FACTOR OF SAFETY

The minimum factor of safety for arriving at the safe pile capacity from the ultimate capacity obtained by using static formulae shall be 2.5.

H-6 PILES IN STRATIFIED SOIL

In stratified soil / C-ϕ soil, the ultimate load capacity of piles should be determined by calculating the skin friction and end bearing in different strata by using appropriate expressions given in **H-1** and **H-2**.

H-7 PILES IN HARD ROCK

When the crushing strength of the rock is more than characteristic strength of pile concrete, the rock should be deemed as hard rock. Piles resting directly on hard rock may be loaded to their safe structural capacity.

H-8 PILES IN WEATHERED / SOFT ROCK

For pile founded in weathered/soft rock different empirical approaches are used to arrive at the socket length necessary for utilizing the full structural capacity of the pile.

Since it is difficult to collect cores in weathered/soft rocks, the method suggested by Cole and Stroud using '*N*' values is more widely used. The allowable load on the pile, Q_a, in kN, by this approach, is given by:

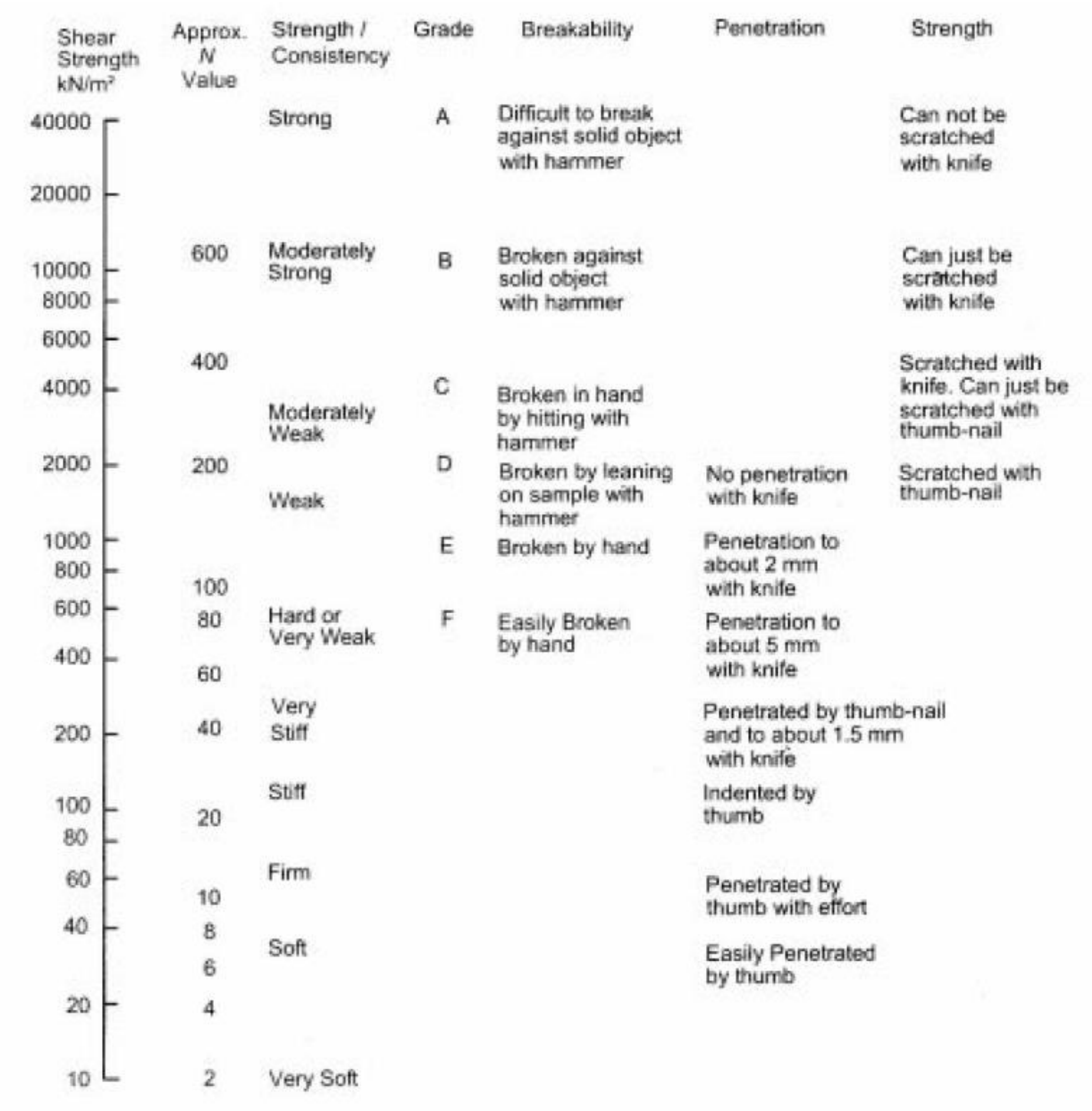
$$Q_a = c_{u1} N_c \cdot \frac{\pi B^2}{4F_s} + \alpha\, c_{u2} \cdot \frac{\pi B L}{F_s}$$

where

c_{u1} = shear strength of rock below the base of the pile, in kN/m^2 (*see* Fig 13);
N_c = bearing capacity factor taken as 9;
F_s = factor of safety usually taken as 3;
α = 0.9 (recommended value);
c_{u2} = average shear strength of rock in the socketed length of pile, in kN/m^2 (*see* Fig 13);
B = minimum width of pile shaft (diameter in case of circular piles), in m; and

L = socket length of pile, in m.

NOTE — For $N \geq 60$, the stratum is to be treated as weathered rock rather than soil.



NOTE — Standard penetration test may not be practicable for N values greater than 200. In such cases, design may be done on the basis of shear strength of rock.

Fig. 13 CONSISTENCY AND SHEAR STRENGTH OF WEATHERED ROCK

ANNEX J

(*Clause* 8.2.5.2)

ANALYSIS OF LATERALLY LOADED PILES

J-1 GENERAL

J-1.1 The ultimate resistance of a vertical pile to a lateral load and the deflection of the pile as the load builds up to its ultimate value are complex matters involving the interaction between a semi-rigid structural element and soil which deforms partly elastically and partly plastically. The failure mechanisms of an infinitely long pile and that of a short rigid pile are different. The failure mechanisms also differ for a restrained and unrestrained pile head conditions.

Because of the complexity of the problem only a procedure for an approximate solution that is adequate in most of the cases is presented here. Situations that need a rigorous analysis shall be dealt with accordingly.

J-1.2 The first step is to determine, if the pile will behave as a short rigid unit or as an infinitely long flexible member. This is done by calculating the stiffness factor *R* or *T* for the particular combination of pile and soil.

Having calculated the stiffness factor, the criteria for behaviour as a short rigid pile or as a long elastic pile are related to the embedded length *L* of the pile. The depth from the ground surface to the point of virtual fixity is then calculated and used in the conventional elastic analysis for estimating the lateral deflection and bending moment.

J-2 STIFFNESS FACTORS

J-2.1 The lateral soil resistance for granular soils and normally consolidated clays which have varying soil modulus is modeled according to the equation:

$$\frac{p}{y} = \eta_h z$$

where

p = Lateral soil reaction per unit length of pile at depth z below ground level;
y = Lateral pile deflection; and
η_h = Modulus of subgrade reaction for which the recommended values are given in Table 11.

J-2.2 The lateral soil resistance for preloaded clays with constant soil modulus is modeled according to the equation:

$$\frac{p}{y} = K$$

where

$$K = \frac{k_1}{1.5} \times \frac{0.3}{B}$$

where k_1 is Terzaghi's modulus of subgrade reaction as determined from load deflection measurements on a 30 cm square plate and B is the width of the pile (diameter in case of circular piles). The recommended values of k_1 are given in Table 12.

Table 11 Modulus of Subgrade Reaction for Granular Soils, η_h, in kN/m^3
(*Clause* J-2.1)

Sl. No.	Soil Type	*N* (Blows/ 30 cm)	Range of η_h kN/m^3 x 10^3 Dry	Submerged
(1)	(2)	(3)	(4)	(5)
i)	Very loose sand	0 – 4	< 0.4	< 0.2
ii)	Loose sand	4 –10	0.4 – 2.5	0.2 – 1.4
iii)	Medium sand	10 – 35	2.5 – 7.5	1.4 – 5.0
iv)	Dense sand	> 35	7.5 – 20.0	5.0 – 12.0

NOTE— The η_h values may be interpolated for intermediate standard penetration values, *N*

Table 12 Modulus of Subgrade Reaction for Cohesive Soil, k_1, in KN/m^3
(*Clause* J-2.2)

Sl. No.	Soil consistency	Unconfined compression strength, q_u kN/m^2	Range of k_1 kN/m^3 x 10^3
(1)	(2)	(3)	(4)
i)	Soft	25 – 50	4.5 – 9.0
ii)	Medium stiff	50 – 100	9.0 – 18.0

iii)	Stiff	100 – 200	18.0 – 36.0
iv)	Very stiff	200 – 400	36.0 – 72.0
v)	Hard	> 400	>72.0

NOTE— For q_u less than 25, k_1 may be taken as zero, which implies that there is no lateral resistance

J-2.3 Stiffness Factors

J-2.4.1 *For Piles in Sand and Normally Loaded Clays*

Stiffness factor *T*, in m = $\sqrt[5]{\frac{EI}{\eta_h}}$

where

E = Young's modulus of pile material, in MN/m^2;
I = Moment of inertia of the pile cross-section, in m^4; and
η_h = Modulus of subgrade reaction variation, in MN/m^3 (*see* Table 12).

J-2.4.2 *For Piles in Preloaded Clays*

Stiffness factor *R*, in m = $\sqrt[4]{\frac{EI}{KB}}$

where

E = Young's modulus of pile material, in MN/m^2;
I = Moment of inertia of the pile cross-section, in m^4;
K = $\frac{k_1}{1.5} \times \frac{0.3}{B}$, (see Table 4 for values of k_1, in MN/m^3); and
B = Width of pile shaft (diameter in case of circular piles), in m.

J-3 CRITERIA FOR SHORT RIGID PILES AND LONG ELASTIC PILES

Having calculated the stiffness factor *T* or *R*, the criteria for behaviour as a short rigid pile or as a long elastic pile are related to the embedded length *L* as given in Table 13.

Table 13 Criteria for Behaviour of Pile Based on its Embedded Length
(*Clause* J-3)

Sl. No.	Type of Pile Behaviour	Relation of Embedded Length with Stiffness Factor	
		Linearly increasing	Constant
(1)	(2)	(3)	(4)
i)	Short (Rigid) Pile	$L \leq 2T$	$L \leq 2R$
ii)	Long (Elastic) Pile	$L \geq 4T$	$L \geq 3.5R$

NOTE— The intermediate *L* shall indicate a case between rigid pile behaviour and elastic pile behaviour

J-4 DEFLECTION AND MOMENTS IN LONG ELASTIC PILES

J-4.1 Equivalent cantilever approach gives a simple procedure for obtaining the deflections and moments due to relatively small lateral loads. This requires the determination of depth of virtual fixity, z_f.

The depth to the point of fixity may be read from the plots given in Fig. 14. *e* is the effective eccentricity of the point of load application obtained either by converting the moment to an equivalent horizontal load or by actual position of the horizontal load application. *R* and *T* are the stiffness factors described earlier.

J-4.2 The pile head deflection, *y* shall be computed using the following equations:

$$\text{Deflection, } y = \frac{H(e+z_f)^3}{3EI} \times 10^3 \qquad \text{.......for free head pile}$$

$$\text{Deflection, } y = \frac{H(e+z_f)^3}{12EI} \times 10^3 \qquad \text{.......for fixed head pile}$$

where

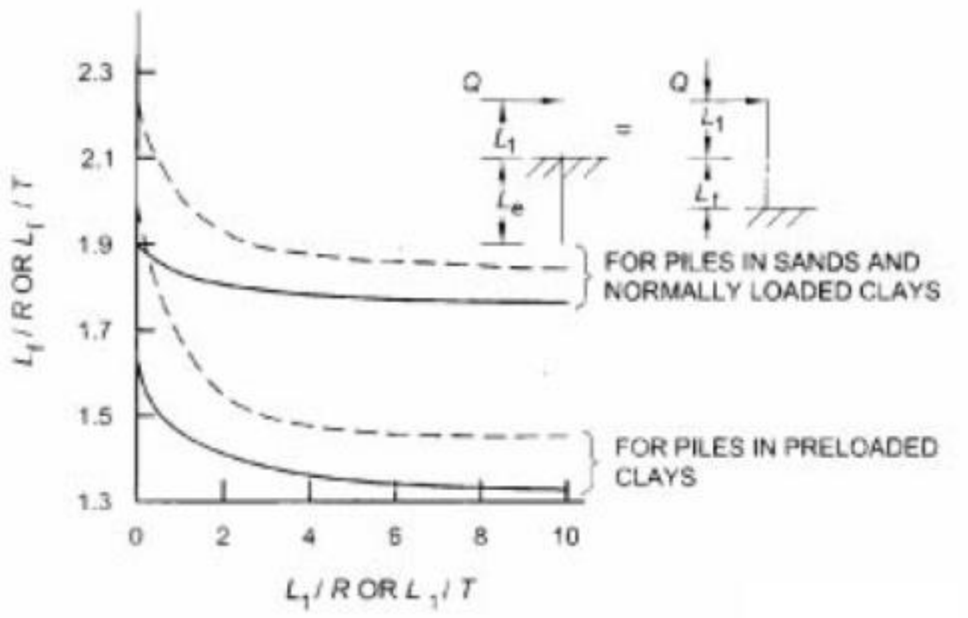
- H = Lateral load, in kN;
- y = Deflection of pile head, in mm;
- E = Young's modulus of pile material, in kN/m^2;
- I = Moment of inertia of the pile cross section, in m^4;
- z_f = Depth to point of fixity, in m; and
- e = Cantilever length above ground/bed to the point of load application, in m.

J-4.3 The fixed end moment of the pile for the equivalent cantilever may be determined from the following expressions.

Fixed end moment, $M_F = H(e + z_f)$ ……….for free head pile

Fixed end moment, $M_F = \dfrac{H(e + z_f)}{2}$ ……….for fixed head pile

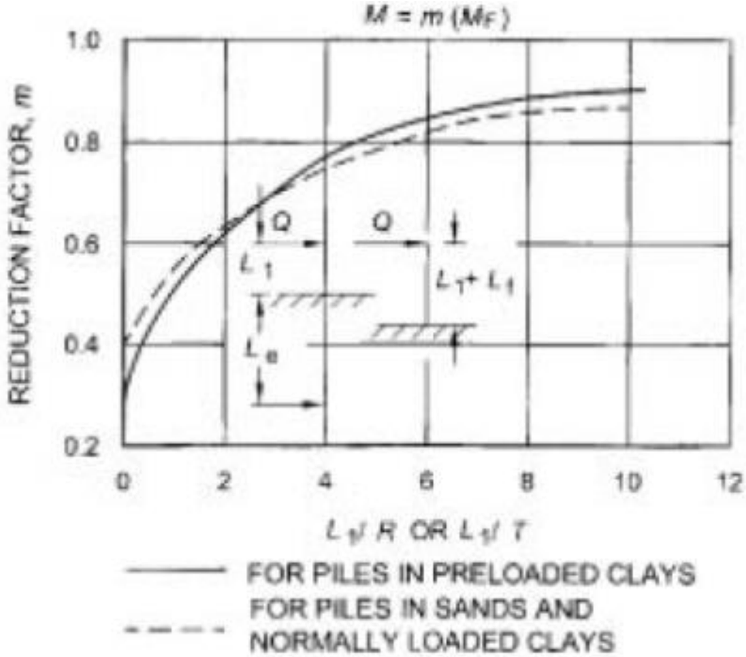
The fixed head moment, M_F of the equivalent cantilever is higher than the actual maximum moment M in the pile. The actual maximum moment may be obtained by multiplying the fixed end moment of the equivalent cantilever by a reduction factor, m, given in Fig. 15.



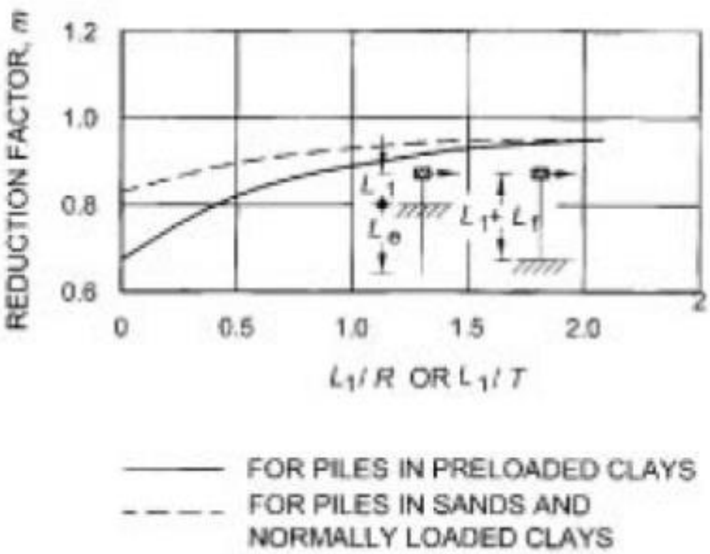
WHERE,
$L_1 = e$ and $L_f = Z_f$

——— FREE HEAD PILES
– – – – FIXED HEAD PILES

FIG **14** DEPTH OF FIXITY



15A For Free Head Pile



15B For Fixed Head Pile

FIG. 15 DETERMINATION OF REDUCTION FACTORS FOR COMPUTATION OF MAXIMUM MOMENT IN PILE

ANNEX K
(*Clause* 11.2.2)

LOAD CARRYING CAPACITY OF UNDERREAMED PILES FROM SOIL PROPERTIES

K-1 ULTIMATE LOAD CAPACITY

The ultimate load capacity of a pile can be calculated from soil properties. The soil properties required are strength parameters, cohesion, angle of internal friction and soil density.

a) *Clayey Soils* – For clayey soils, the ultimate load carrying capacity of an under-reamed pile may be worked out from the following expression :

$$Q_u = A_p N_c C_p + A_a N_c C'_a + \alpha C_a A_s$$

where

Q_u = ultimate bearing capacity of pile in kN;

A_p = cross-sectional area of the pile stem at the toe level in m^2 ;

N_c = bearing capacity factor, usually taken as 9;

C_p = cohesion of the soil around toe in kN/m^2 ;

A_a = $(\pi/4)(D_u^2 - D^2)$, where D_u and D are the under-reamed and stem diameter, respectively in m;

C_a = average cohesion of the soil along the pile stem in kN/m^2 ;
A_s = surface area of the stem in m^2 ;

A'_s = surface area of the cylinder circumscribing the under-reamed bulbs in m^2 ;

C'_a = average cohesion of the soil around the under-reamed bulbs; and

α = reduction factor (usually taken 0.5 for clays).

NOTES

1 The above expression holds for the usual spacing of under-reamed bulbs spaced at not more than one and a half times their diameter.
2 If the pile is with one bulb only, the third term will not occur. For calculating uplift load, the first term will not occur in the formula.
3 In case of expansive soil top 2 m strata should be neglected for computing skin friction.

b) *Sandy Soils*

$$Q_{\mathrm{u}} = A_{\mathrm{p}}\left(\frac{1}{2}D_{\gamma}N_{\gamma} + \gamma d_{\mathrm{f}}N_{\mathrm{q}}\right) + A_{\mathrm{a}}\left(\frac{1}{2}D_{\mathrm{u}}n\gamma' N_{\gamma}\right) + \gamma N_{\mathrm{q}}\sum_{\mathrm{r}=1}^{\mathrm{r}=\mathrm{n}} d_{\mathrm{r}} + \frac{1}{2}\Pi D_{\gamma}K\tan\delta\left(d_1^{\;2} + d_{\mathrm{f}}^{\;2} - d_{\mathrm{n}}^{\;2}\right)$$

where

A_{p} = π $D^2/4$, where D is stem diameter in m;

A_{a} = π/4 ($D_{\mathrm{u}}^2 - D^2$) where D_{u} is the under-reamed bulb diameter in m;

n = number of under-reamed bulbs;

ϒ = average unit weight of soil (submerged unit weight in strata below water table) in kN/m^3 ;

N_{Υ}, N_{q} = bearing capacity factors, depending upon the angle of internal friction;

d_{r} = depth of the centre of different under-reamed bulbs below ground level in m;

d_{f} = total depth of pile below ground level in m;

K = earth pressure coefficient (usually taken as 1.75 for sandy soils);

δ = angle of wall friction (may be taken as equal to the angle of internal friction $\varnothing$);

d_1 = depth of the centre of the first under-reamed bulb in m; and

d_{n} = depth of the centre of the last under-reamed bulb in m.

NOTES

1 For uplift bearing on pile tip, A_{p} will not occur.
2 N_{Υ} will be as specified in good practice [6-2(11)] and N_{q} will be taken from Fig. 11.

c) *Soil Strata having both Cohesion and Friction* – In soil strata having both cohesion and friction or in layered strata having two types of soil, the bearing capacity may be estimated using both the formulae. However, in such cases load test will be a better guide.

d) *Compaction Piles in Sandy Strata* – For bored compaction piles in sandy strata, the formula in (b) shall be applied but with the modified value of $\varnothing_1$ as given below:

$$\Phi_1 = (\Phi + 40)/2$$

where

Φ = angle of internal friction of virgin soil.

The values of N_{γ}, N_q and δ are taken corresponding to $\varnothing_1$. The value of the earth pressure coefficient K will be 3.

e) *Piles Resting on Rock* – For piles resting on rock, the bearing component will be obtained by multiplying the safe bearing capacity of rock with bearing area of the pile stem plus the bearing provided by the bulb portion.

NOTE – To obtain safe load in compression and uplift from ultimate load capacity generally the factors of safety will be 2.5 and 3 respectively.

ANNEX M
(Clause 5.1.6.1)

SOIL IMPROVEMENT METHODS

	Summary of Soil Improvement Methods								
	Method	**Principle**	**Most Suitable Soil Conditions/ Types**	**Maximum Effective Treatment Depth**	**Special Materials Required**	**Special Equipment Required**	**Properties of Treated Material**	**Special Advantages and Limitation**	**Relative Cost**
In-Situ Deep compaction of Cohesionless Soils	Blasting	Shock waves and vibrations cause liquefaction and displacement with settlement to higher density	Saturated, clean sands: partly saturated sands and silts (collapsible loess) after flooding	>30 m	Explosives, backfill to plug drill holes, hole casings	Jetting or drilling machine	Can obtain relative densities to 70-80, may get variable density strength gain	Rapid, inexpensive, can treat any size areas: variable properties, no improvement near surface, dangerous	Low
	Vibratory Probe	Densification by vibration; liquefaction induced settlement under overburden	Saturated or dry clean sand	20 m (Ineffective above 3 – 4m depth)	None	Vibratory pile driver and 750 mm dia open steel pipe	Can obtain relative densities of up to 80. Ineffective in some sands	Rapid, simple, good underwater, soft under layers may damp vibrations, difficult to penetrate, stiff over layers, not good in partly saturated soils	Moderate
	Vibro-compaction	Densification by vibration and compaction of backfill material	Cohesionless soils with less than 20 fines	30 m	Granular backfill, water supply	Vibroflot, crane, pumps	Can obtain high relative densities, good uniformity	Useful in saturated and partly saturated soils, uniformity	Moderate

In-Situ Deep compaction of Cohesionless Soils	Compaction Piles	Densification by displacement of pile volume and by vibration during driving	Loose sandy soils: partly saturated clayey soils, loess	> 20 m	Pile material (often sand or soil plus cement mixture)	Pile driver, special sand pile equipment	Can obtain high densities, good uniformity	Useful in soils with fines, uniform compaction, easy to check results, slow, limited improvement in upper 1-2m	Moderate to high
	Heavy tamping (Dynamic Consolidation)	Repeated application of high intensity impacts at surface	Cohesionless soils, waste fills, partly saturated soils	30 m	None	Tampers of up to 200 tonnes, high capacity crane	Can obtain good improvement and reasonable uniformity	Simple, rapid, suitable for some soils with fines; usable above and below water, requires control, must be away from existing structures	Low
Injection and Grouting	Particulate Grouting	Penetration grouting-fill soil pores with soil, cement, and/or clay	Medium to coarse sand and gravel	Unlimited	Grout, water	Mixers, tanks, pumps, hoses	Impervious, high strength with cement grout, eliminate liquefaction danger	Low cost grouts, high strength; limited to coarse-grained soils, hard to evaluate	Lowest of the grout systems
	Chemical Grouting	Solution of two or more chemicals react in soil pores to form a gel or a solid precipitate	Medium silts and coarser	Unlimited	Grout, water	Mixers, tanks, pumps, hoses	Impervious, high strength with cement grout, eliminate liquefaction danger	Low viscosity controllable gel time, good water shut-off; high cost, hard to evaluate	High to very high

Injection and Grouting	Pressure injected Lime	Lime slurry injected to shallow depths under high pressure	Expansive clays	Unlimited, but 2-3 m usual	Lime, water surfactant	Slurry tanks, agitators, pumps, hoses	Lime in capsulated zones formed by channels resulting from cracks, root holes, hydraulic fracture	Only effective in narrow range of soil conditions	Competitive with other solutions to expansive soil problems
	Displacements Grout	Highly viscous grout acts as radical hydraulic jack when pumped in under high pressure	Soft, fine-grained soils; foundation soils with large voids or cavities	Unlimited, but a few metre usual	Soil, cement water	Batching equipment, high pressure pumps, hoses	Grout bulbs within compressed soil matrix	Good for correction of differential settlements, filling large voids; careful control required	Low material high injection
	Electro-kinetic injection	Stabilization chemicals moved into soil by electro-osmosis or colloids into pores by electrophoresis	Saturated silts; silty clays (clean sands in case of colloid injection)	Unknown	Chemicals stabilizer colloidal void fillers	DC power supply, anodes, cathodes	Increased strength, reduced compressibility reduced liquefaction potential	Existing soil and structures not subjected to high pressures; not good in soils with high conductivity	Expansive
	Jet Grouting	High speed jets at depth excavate, inject, and mix stabilizer with soil to form columns or panels	Sands, silts, clays	--	Water, stabilizing chemicals	Special jet nozzle, pumps, pipes and hoses	Solidified columns and walls	Useful in soils that can't be permeation grouted, precision in locating treated zones	--

Precompression	Preloading with/without Drain	Load is applied sufficiently in advance of construction so that compression of soft soils is completed prior to development of the site	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	--	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment, large water tanks or vacuum drainage systems sometimes used; settlement markers, piezometers	Reduced water content and void ratio, increased strength	Easy, theory well developed, uniformity; requires long time (vertical drains can be used to reduce consolidation time)	Low (moderate if vertical drains are required)
	Surcharge Fills	Fill in excess of that required permanently is applied to achieve a given amount of settlement in a shorter time; excess fill then removed	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	--	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment; settlement markers, piezometers	Reduced water content, void ratio and compressibility increased strength	Faster than preloading without surcharge, theory well developed, extra material handling; can use vertical drains to reduce consolidation time	Moderate
	Electro-osmosis	DC current causes water flow from anode towards cathode where it is removed	Normally consolidated silts and silty clays	--	Anodes (usually rebars or aluminium) cathodes (well points or rebars)	DC power supply, wiring, metering systems	Reduced water content and compressibility, increased strength, electrochemical hardening	No fill loading required, can be used in confined area, relatively fast; non-uniform properties between electrodes; not good in highly conductive soils	High

Admixtures	Remove and Replace	Foundation soil excavated, improved by drying or admixture, and re-compacted	Inorganic soils	10 m	Admixture stabilizers	Excavating, mixing and compaction equipment, dewatering system	Increased strength and stiffness, reduced compressibility	Uniform, controlled foundation soil when replaced; may require large area dewatering	High
	Structural Fills	Structural fill distributes loads to underlying soft soils	Use over soft clays or organic soils, marsh deposits	--	Sand, gravel fly ash, bottom ash, slag, expanded aggregate, clam shell or oyster shell, incinerator ash	Mixing and compaction equipment	Soft sub-grade protected by structural load-bearing fill	High strength, good load distribution to underlying soft soils	Low to high
	Mix-in-place Piles and Walls	Lime cement or asphalt introduced through rotating auger or special in-place mixer	All soft or loose inorganic soils	> 20m	Cement lime asphalt, or chemical stabilizer	Drill rig, rotary cutting and mixing head, additive pro-portioning equipment	Solidified soil piles for walls of relatively high strength	Use native soil, reduced lateral support requirements during excavation; difficult quality control	Moderate to high
Thermal	Heating	Drying at low temperatures; alteration of clays at intermediate temperatures (400-600 °C); fusion at high temperatures (>1 000 °C)	Fine- grained soils, especially partly saturated clays and silts, loess	15 m	Fuel	Fuel tanks, burners, blowers	Reduced water content, plasticity, water sensitivity; increased strength	Can obtain irreversible improvements in properties; can introduce stabilizers with hot gases	High

	Freezing	Freeze soft, wet ground to increase its strength stiffness	All soils	Several m	Refrigerant	Refrigeration system	Increased strength and stiffness, reduced permeability	No good in flowing ground water, temporary	High
Reinforcement	Vibro replacement stone and sand columns	Hole jetted into soft, fine-grained soil and backfilled with densely compacted gravel or sand	Soft clays and alluvial deposits	20 m	Gravel or crushed rock backfill	Vibroflot, crane or vibrocat, water	Increased bearing capacity, reduced settlement	Faster than precompression, avoids dewatering required for remove and replace; limited bearing capacity	Moderate to high
	Root Piles, Soils Nailing	Inclusions used to carry tension, shear, compression	All soils	--	Reinforcing bars, cement grout	Drilling and grouting equipment	Reinforced zone behaves as a coherent mass	*In-situ* reinforcement for soils that can't be grouted or mixed in-place with admixtures	Moderate to high
	Strips and membranes	Horizontal tensile strips, membranes buried in soil under embankments, gravel base courses and footings	Cohesionless soils	Can construct earth structures to heights of several metres	Metal or plastic strips, geotextiles	Excavating, earth handling, and compaction equipment	Self-supporting earth structures, increased bearing capacity, reduced deformations	Economical, earth structures coherent, can tolerate deformations; increased allowable bearing pressure	Low to moderate

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.

(1)	IS 1892:1979	Code of practice for subsurface investigation for foundation (*first revision*)
	IS 2131:1981	Method of standard penetration test for soils (*first revision*)
	IS 2132:1986	Code of practice for thin walled tube sampling of soils (*second revision*)
	IS 4434:1978	Code of practice for *in-situ* vane shear test for soils (*first revision*)
	IS 4968	Method for subsurface sounding for soils
	(Part 1):1976	Part 1 Dynamic method using 50 mm cone without bentonite slurry (*first revision*)
	(Part 2):1976	Part 2 Dynamic method using cone and bentonite slurry (*first revision*)
	(Part 3):1976	Part 3 Static cone penetration test (*first revision*)
	IS 8763:1978	Guide for undisturbed sampling of sands and sandy soils
	IS 9214:1979	Method for determination of modulus of subgrade reaction (*k*-value) of soils in the field
(2)	IS 1892:1979	Code of practice for subsurface investigation for foundation (*first revision*)
(3)	IS 10042:1981	Code of practice for site-investigations for foundation in gravel boulder deposits
(4)	IS 13365 (Part 1):1998	Guidelines for quantitative classification systems of rock mass Part 1 RMR for predictin of engineering properties
(5)	IS 2720	Methods of tests for soils
	(Part 1):1983	Part 1 Preparation of dry soil samples for various tests (*second revision*)
	(Part 2):1973	Part 2 Determination of water content (*second revision*)
	(Part 3):1980	Part 3 Determination of specific gravity
	(Part 3/Sec 1):1980	Part 3 Determination of specific gravity : Section 1 Fine grained soils (*first revision*)
	(Part 3/Sec 2):1980	Part 3 Determination of specific gravity : Section 2 Fine, medium and coarse grained soils (*first revision*)

	(Part 4):1985	Part 4 Grain size analysis (*second revision*)
	(Part 5):1985	Part 5 Determination of liquid and plastic limits (*second revision*)
	(Part 10):1991	Part 10 Determination of unconfined compressive strength (*second revision*)
	(Part 11):1993	Part 11 Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure (*first revision*)
	(Part 12):1981	Part 12 Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure (*first revision*)
	(Part 13):1986	Part 13 Direct shear test (*second revision*)
	(Part 15):1986	Part 15 Determination of consolidation properties (*first revision*)
	(Part 28):1974	Part 28 Determination of dry density of soils in place, by the sand replacement method (*first revision*)
	(Part 29):1975	Part 29 Determination of dry density of soils in place, by the core cutter method (*first revision*)
	(Part 33):1971	Part 33 Determination of the density in-place by the ring and water replacement method
	(Part 34):1972	Part 34 Determination of density of soils in-place by rubber-balloon method
	(Part 39/Sec 1):1977	Part 39 Direct shear test for soils containing gravel : Section 1 Laboratory test
(6)	IS 1498:1970	Classification and identification of soils for general engineering purposes (*first revision*)
(7)	IS 401:2001	Code of practice for preservation of timber (*fourth revision*)
(8)	IS 15180:2002	Guidelines for use in prediction of subsidence and associated parameters in coal mines having nearly horizontal single seam workings
(9)	IS 3764:1992	Code of safety for excavation work (*first revision*)
(10)	IS 1904:1986	Code of practice for design and construction of foundations in soils : General requirements (*third revision*)
(11)	IS 6403:1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
(12)	IS 1888:1982	Method of load tests on soils (*second revision*)
(13)	IS 2131:1981	Method for standard penetration test for soils (*first revision*)
(14)	IS 8009(Part 1):1976	Code of practice for calculation of settlement of foundations : Part 1 Shallow foundations subjected to symmetrical static vertical loads

(15)	IS 12070:1987	Code of practice for design and construction of shallow foundations on rocks
(16)	IS 1080:1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
(17)	IS 11089:1984	Code of practice for design and construction of ring foundations
(18)	IS 9456:1980	Code of practice for design and construction of conical and hyperbolic paraboloidal types of shell foundations
(19)	IS 2974 (Part 1):1982	Code of practice for design and construction of machine foundations: Part 1 Foundations for reciprocating type machines (*second revision*)
(20)	IS 2911 (Part 4):2013	Design and construction of pile foundations - Code of practice: Part 4 Load test on piles (*second revision*)
(21)	IS 14593:1998	Design and construction of bored cast *in-situ* piles founded on rocks - Guidelines
(22)	IS 4968	Method for subsurface sounding for soils:
	(Part 1):1976	Dynamic method using 50 mm cone without bentonite slurry (*first revision*)
	(Part 2):1976	Dynamic method using cone and bentonite slurry (*first revision*)
	(Part 3):1976	Static cone penetration test (*first revision*)
(23)	IS 2911	Code of practice for design and construction of pile foundations
	(Part 1/Sec 1):2010	Part 1 Concrete piles, Section 1 Driven cast in-situ concrete piles (*second revision*)
	(Part 1/Sec 2):2010	Part 1 Concrete piles, Section 2 Bored cast in-situ concrete piles (*second revision*)
(24)	IS 14893:2001	Guidelines for non-destructive integrity testing of piles
(25)	IS 2911(Part 1/Sec 3): 2010	Code of practice for design and construction of pile foundations : Part 1 Concrete piles, Section 3 Precast driven concrete piles (*second revision*)
(26)	IS 2911(Part 1/Sec 4): 2010	Code of practice for design and construction of pile foundations : Part 1 Concrete piles, Section 4 Precast concrete piles in prebored holes (*first revision*)
(27)	IS 2911 (Part 3):1980	Code of practice for design and construction of pile foundations: Part 3 Under-reamed pile foundation (*first revision*)
(28)	IS 2911(Part 2):1980	Code of practice for design and construction of pile foundations : Part 2 Timber piles (*first revision*)

(29)	IS 2974	Code of practice for design and construction of machine foundation
	(Part 1):1982	Part 1 Foundations for reciprocating type machine (*second revision*)
	(Part 2):1980	Part 2 Foundations for impact type machines (hammer foundations) (*first revision*)
	(Part 3):1992	Part 3 Foundations for rotary type machines (medium and high frequency) (*second revision*)
	(Part 4):1979	Part 4 Foundations for rotary type machines of low frequency (*first revision*)
	(Part 5):1987	Part 5 Foundations for impact machines other than hammers (forging and stamping press; pig breakers, drop crusher and jetter) (*first revision*)
	IS 13301:1992	Guidelines for vibration isolation for machine foundations
	IS 9556:1980	Code of practice for design and construction of diaphragm walls
(30)	IS 13094:1992	Guidelines for selection of ground improvement techniques for foundation in weak soils
(31)	IS 15284 (Part 1):2003	Design and construction for ground improvement — Guidelines: Part 1 Stone columns
(32)	IS 15284 (Part 2):2004	Design and construction for ground improvement — Guidelines: Part 2 Preconsolidation using vertical drains
(33)	IS 13162 (Part 2):1991	Geotextiles – Methods of test: Part 2 Determination of resistance to exposure of ultra-violet light and water (Xenon arc type apparatus)
	IS 13321(Part 1):1992	Glossary of terms for geo-synthetics : Part 1 Terms used in materials and properties
	IS 13325:1992	Method of test for the determination of tensile properties of extruded polymer geogrids using the wide strip
	IS 13326 (Part 1):1992	Method of test for the evaluation of interface friction between geosynthetics and soil : Part 1 Modified direct shear technique
	IS 14293:1995	Geotextiles – Method of test for trapezoid tearing strength
	IS 14294:1995	Geotextiles - Method for determination of apparent opening size by dry sieving technique
	IS 14324:1995	Geotextiles – Methods of test for determination of water permeability-permittivity
	IS 14706:1999	Geotextiles – Sampling and preparation of test specimens
	IS 14714:1999	Geotextiles – Determination of abrasion resistance
	IS 14715 :2000	Woven jute geotextiles – Specification

	IS 14716 :1999	Geotextiles – Determination of mass per unit area
	IS 14739:1999	Geotextiles – Methods for determination of creep
	IS 14986:2001	Guidelines for application of jute geo-grid for rain water erosion control in road and railway embankments and hill slopes
	IS 15060:2001	Geotextiles – Tensile test for joints/seams by wide width method
(34)	IS 2720	Methods of tests for soils
	(Part 11):1993	Part 11 Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure (*first revision*)
	(Part 12):1981	Part 12 Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure (*first revision*)
(35)	IS 9214:1979	Method of determination of subgrade reaction (*K* value) of soils in the field

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