(Provisional)

MYANMAR

NATIONAL

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CODE

2012

PART4

SOILS AND FOUNDATIONS

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MYANMAR NATIONAL BUILDING CODE – 2012

PART 4 SOILS AND FOUNDATIONS

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ABBREVIATION AND SYMBOLS

A_g	Pile cross-sectional area, square inches
A_{ch}	Core area defined by spiral outside diameter
a_{max}	Peak ground acceleration of the site
A_{sh}	Cross-sectional area of transverse reinforcement
CLSM	Controlled low- strength material
CPT	Cone Penetration Test
CQHP	Committee for Quality control of High-rise building construction Project
CRR	Cyclic Resistance Ratio of the in situ soil
CSR	Cyclic Stress Ratio of the in situ soil
D	Depth of Soil
E	Modulus of Elasticity
F	Factor of Safety
FAS	Fourier amplitude spectrum
FI	No. of fracture
FS	Factor of Safety
$f'c$	Specified compressive strength of concrete
f_{yh}	Yield strength of spiral reinforcement
f_{pc}	The effective stress on the gross section
Fy	Minimum specified yield strength
F_b	Bending at fiber stress
F_v	Longitudinal shear
F_c	Axial compression
F_{cb}	Axial compression when combined with bending
$F_{c(per)}$	Compression perpendicular to grain
$F_{t(par)}$	Tension parallel to grain
$Ft_{(per)}$	Tension perpendicular to grain
fy	Yield strength of the steel
g	Acceleration due to gravity (32.2 ft/s2 or 9.81 m/s2)
GPR	Ground-Probing Rader
h_c	Cross-sectional dimension of pile core measured center to center of hoop reinforcement
IP	Induce polarization survey

k_s	Modulus of Sub-grade Reaction
LL	Liquid Limit
P	Axial load on pile, pounds
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PGD	Peak Ground Displacement
RQD	Rock Quality Designation
SASW	Spectral Analysis of Surface Wave
S.C.R	Solid Core Recovery
SF	Safety Factor
SP	Self – potential survey
SPT	Standard Penetration Test
T.C.R	Total Core Recovery
TDEM	Electromagnetic Survey
TEM	Transient Electromagnetic
v_s^{30}	Average shear wave velocity of upper 30 m depth
VLF	Very low frequency
q	Bearing Pressure
q_a	Bearing Pressure, Allowable
q_s	Bearing Capacity, Safe
q_{ns}	Bearing Capacity, Net Safe
q_{nu}	Bearing Capacity, Net Ultimate
q_u	Bearing Capacity, Ultimate
w	Water content
σ_{vo}	Total vertical stress at a particular depth
σ'_{vo}	Vertical effective stress at a particular depth
r_d	Depth reduction factor or stress reduction coefficient
s	Spacing of transverse reinforcement measured along length of pile
c	Cohesion
w	Water Content
γ	Unit Weight of Soil
δ	Settlement
σ_{vo}	Total vertical stress at a particular depth where the liquefaction analysis is being performed.

σ'_{vo}	Vertical effective stress at a particular depth where the liquefaction analysis is being
r_d	Depth reduction factor or stress reduction coefficient
ρ_s	Spiral reinforcement index (vol. spiral/vol. core).

4.1 GENERAL

4.1.1 Scope

This section covers soil and foundation design for all buildings such as individual footings, combined footings, strip footings, rafts, piles and other foundation systems to ensure safety and serviceability without exceeding the permissible stresses of foundation material and the bearing capacity of the supporting soil. Some parameters related to seismic – resistant designs are also included.

This section is formulated with a view to implement in national and economical policies in soils and foundations, such that the design of buildings can be accomplished with safety and usability, using advanced technology, with economy and rationality, assuring the quality and protection of the environment.

Design of soil and foundation must be carried out based on the principles of suiting measures to local conditions, using local materials, protecting the environment and economizing on resources. The design shall be painstakingly performed with comprehensive consideration given to the type of structures, availability of materials and geotechnical survey data of soil and rock.

4.1.2 Design

Allowable bearing pressures, allowable stresses and design formulae provided in this section shall be used with the allowable stress design load combinations specified in Structural Design Section **3.2.1**. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in this code (see Section on Structural Design, Concrete, Masonry and Steel). Safety during construction and the protection of adjacent public and private properties shall govern the design and construction of excavations and fills.

4.1.2.1 Foundation design for seismic overturning

Where the foundation is proportioned using the load combinations specified in Structural Design **Section 3.4.2**, and the computation of the seismic overturning moment is by the equivalent lateral-force method or the model analysis, the proportioning shall be in accordance with Section **3.4.2.**

4.1.2.1.1 Reduction of Foundation Overturning

Overturning effects at the soil foundation interface are permitted to be reduced by 25 percent for foundations of structures that satisfy both of the following conditions:

a) The structure is designed in accordance with the Equivalent Lateral Force Analysis as set forth in Structural Design **Section 3.4.2.**

b) The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil-foundation interface are permitted to be reduced by 10 percent for foundations of structures designed in accordance with the modal analysis requirements of Structural Design **Section 3.4.2.**

4.1.3 Definitions

For the purpose of this Section, the following definitions shall apply.

4.1.3.1 Soil

Clay. Very fine – grained soil (the particles are less than 0.002 mm in size), consisting mainly of hydrate silicate of aluminum. Clay is a plastic cohesive soils which shrinks on drying, expands on wetting and when compressed it gives up water. It comes from the chemical decomposition and disintegration of rock constituents.

Clay (Firm) or (Medium Stiff). A clay which at its natural water content can be moulded by substantial pressure with the fingers and can be excavated with a spade.

Clay (Very soft). (Soft), A clay which at its natural water content can be easily moulded with the fingers and readily excavated.

Clay (Stiff), (Very Stiff), (Hard). A clay which at its natural water content cannot be moulded with fingers and require a pick or pneumatic spade for its removal.

Gravel. Cohesionless aggregates of angular or rounded or semi – rounded fragments of more or less unaltered rocks or minerals. The size is larger than 2.0 mm and less than 60 mm.

Hard Rock. A fresh rock which is normally required blasting or chiseling for excavation.

Laterite and Lateritic soils. Laterite which possess reddish colour should be regarded as a highly weathered material resulted from the concentration of hydrate oxides of iron and aluminum. In the laterite, the ratio of silica oxide (SiO_2) to sesquioxides (Fe_2O_3, Al_2O_3) is usually less than 1.33. Laterite are good foundation soils and it can be used as subbase material for road and small airfield construction. Lateritic soil has the reddish colour and the ratio of silica oxide (SiO_2) to sesquioxides (Fe_2O_3, Al_2O_3) is generally from 1.33 to 2. They are fair to good foundation materials for buildings. Lateritic soils can be divided into three groups; ferruginous soil, ferrallitic soils and ferrisols soils. Ferruginous soil can have better strength than others. The clay minerals of lateritic soils are mostly kaolinite in nature.

Liquefaction. The phenomenon of liquefaction is generally associated with cohesion-less soils. It results from seismic shaking that is of a sufficient intensity and duration. It occurs most commonly in loose, saturated, granular soils that are uniformly graded and that contain few fines. Although sands are especially susceptible, liquefaction is also known to develop in some silts and gravels. The generation of excess pore pressure due to rapid loading under un-drained condition is hallmark of all liquefaction phenomena.

Predominant Period of Soil. It is a parameter that provides a useful tool, although somewhat crude representation of the frequency content of a ground motion. The predominant period is defined as the period of vibration corresponding to the maximum value of the Fourier amplitude spectrum (FAS). During an earthquake, the buildings which have the natural periods of as same as the predominant period of underlying soil deposits will be felt strong shaking and are liable to severe damage.

Problematic Soil

(a) **Expansive Soil.** Foundation materials that exhibit volume change when there are changes in their moisture content are referred to as expansive or swelling clay soils. More detailed is shown in Appendix A.

(b) **Dispersive Soil**. The soil which disperse in the presence of water and can therefore be easily scoured. The most major soil type is CLAY and SILT combination with some amount of sand. The index properties give no indication about this treacherous soil. Detail criteria and suggested tests are shown in Appendix A.

(c) **Peat.** Peat is a fibrous mass of organic matter in various stages of decomposition and dark brown and black in color and of spongy consistency.

(d) **Black Cotton Soil**. It is the inorganic clay of medium to high compressibility. They form a major soil group in middle parts of Myanmar. They are predominantly montmorillonitic in structure and Black or Blackish Grey or Greenish brown in color. They are characterized by high shrinkage and swelling properties.

Sand. Sand is cohesionless soils, the soil particles do not tend to stick together. The particle size ranges from 0.06 mm to 2 mm.

Sand (Fine). Sand which contains particles of size greater than 0.06 mm and less than 0.02 mm

Sand (Medium). Sand which contains particles of size greater than 0.02 mm and less than 0.6 mm

Sand (Coarse). Sand which contains particles of size greater than 0.6 mm and less than 2 mm

Silt. A fine grained soil with little or no plasticity, the size of particles ranges from 0.002 mm to 0.06 mm.

Soft Rock. A rocky cemented material which offers a high resistance to picking up with pick axes and sharp fools but which does not normally required blasting or chiseling for excavation.

Soil. Sediments or other unconsolidated accumulations of soil particles produced by the physical and chemical decomposition of rock and which may or may not contain organic matter, soil is not solid matter but contains air and water between the soil particles.

Soil (Coarse Grained). Soil which includes the coarse and large siliceous and unaltered products of weathered rock is regarded as coarse grained soil. They possess no plasticity and tend to lock cohesion when in dry state.

Soil (Fine Grained). Soil where more than 50% of the material less than 60mm is smaller than 0.06mm.Soil consisting of fine and altered products of weathered rocks, possessing cohesion and plasticity in their natural state is regarded as fine grained soil.

Soil Amplification. Soil amplification is the ratio of amplitude of displacement of the objective layer (surface) to that of the reference layer (engineering bedrock). It is a function with respect to frequency and is equivalent with the ratio of acceleration. The areas covered with thick, soft soil generally show higher amplification. Basin effects also have a great control on amplification characteristics of soil deposits. Some severe damages during an earthquake are mainly related to soil amplification.

4.1.3.2 Shallow Foundation

Back Fill. Material used to raise the ground level to fill a depression, or for construction of an embankment.

Bearing Capacity Safe *(q_s)*. [≈ gross allowable bearing capacity ($q'_{all} = q_u/F$)

The maximum pressure which the soil can carry safety without risk of shear failure. It is equal to the net safe bearing capacity plus original overburden pressure. It is also referred to as the ultimate bearing capacity divided by the factor of safety.

$q_s = q_{ns}+\gamma D = q_{nu}/F+\gamma D= q_u/F$

Bearing Capacity. The supporting power of a soil or rock is referred to as its bearing capacity.

Bearing Capacity, Ultimate *(q_u)*. It is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear. (or) The intensity of loading on the foundation which would cause shear failure of the soil

Bearing Pressure, Allowable (q_a).It is the net loading intensity at which neither the soil fails in shear nor there excessive settlement detrimental to the structure.

Continuous Spread Footing. These are also known as wall footings or strip footings and are used to support bearing walls.

Combined Footing. It supports more than one column. It is useful when columns are located too close together for each to have its own footing.

Factor of Safety. It is applied to the ultimate bearing capacity (net) to arrive at the value of the safe bearing capacity (net).

Footing. It is a portion of the foundation of a structure that transmits loads directly to the soil.

Foundation. It is the part of the structure which is in direct contact with and transmitting loads to the ground.

Ring Spread Footing. These are continuous footings that have been wrapped into a circle. It is commonly used to support the walls of above ground circular storage tanks.

Shallow Foundation. Foundations that have a depth of embedment to width ration of approximately less than four.

Spread Footing. It is an enlargement at the bottom of a column or bearing wall that spreads the applied structural loads over a sufficiently large soil area.

Strip Footing. A footing providing a continuous longitudinal ground bearing.

Mat Foundation (Raft Foundation). A mat is essentially a very large spread footing that usually encompasses the entire footprint of the structure. They are also known as raft foundation.

4.1.3.3 Deep Foundation

Augered uncased piles. Augered uncased piles are constructed by depositing concrete into an uncased auger hole, either during or after the withdrawal of the auger.

Bearing Pile. The pile which transfers the load to a stronger stratum underlying the weak zone.

Batter Pile (Raker Pile). The pile which is installed at an angle to the vertical.

Bored Pile. A pile formed with or without a casing by drilling a hole and subsequently filling it with plain or reinforced concrete.

Belled piers. Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing.

Caisson piles. Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.

Concrete-filled steel pipe and tube piles. Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.

Cut -off Level. It is the level where the installed pile is cut-off to connect the pile cap or beams or any other structural components at that level. (or)The prescribed elevation at which the top of a pile is cut. This may be above or below ground level.

Driven uncased piles. Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.

Driven Precast Pile. The precast piles are constructed in concrete (reinforced or pre-stressed) which is cast and cured in a casting yard and subsequently driven into the ground until it has attained sufficient strength.

Driven Cast-in Situ Pile. A pile installed by driving a permanent or temporary casing, and filling the hole so formed with plain or reinforced concrete.

Enlarged base piles. Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.

Flexural Length. Flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

Friction Pile. The pile which carries the load by mobilizing the friction along its sides.

Factor of Safety. It is the ratio of the ultimate load capacity of a pile to the safe load (working load) of a pile.

Jacked Pile. A pile, usually in short section, which is forced into ground by jacking it against a reaction from the kentledge.

Kentledge. Material used to add temporary loading to a structure or as a dead weight in a loading test.

Micropiles. Micropiles are 16-inch-diameter (406 mm) or less bored, grouted-in-place piles incorporating steel pipe (casing) and/or steel reinforcement.

Negative Skin Friction. A downward frictional force acting to the shaft of a pile caused by the consolidation of compressible strata. It has the effect to increase the loading on the pile and reducing the factor of safety.

Pile Foundations. Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, end bearing or a combination of both.

Pier Foundations. Pier foundations consist of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Pile Raft. A foundation formed of piles and a raft acting together.

Pile Cap. A concrete block cast on the head of a pile or a group of piles to transmit the load from the structure to the pile or group of piles.

Steel-cased piles. Steel-cased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole. The steel casing is left permanently in place and filled with concrete.

Test Pile. A pile installed before the commencement of the main piling works, to which a load is applied to determine the load/settlement characteristics of the pile and the surrounding soil.

Tension Pile. A pile that is designed to resist a tensile force.

Timber piles. Timber piles are round, tapered timbers with the small (tip) end embedded into the soil.

Ultimate Load. The maximum load which a pile can carry before failure of ground or failure of pile materials.

Working Load (allowable load). The load which the pile is designed to carry.

4.2 SITE INVESTIGATION

4.2.1 Geotechnical Site Investigation

Geotechnical site investigation is the process of evaluating the geotechnical character of a site. It may include one or more of the followings:

- Evaluation of the geology and hydrogeology of the site.
- Examination of existing geotechnical information pertaining to the site.
- Excavating or boring in soil or rock.
- In-situ assessment of geotechnical properties of materials.
- Recovery of samples of soil or rock for examination, identification, recording, testing or display.
- Testing of soil or rock samples to quantify properties relevant to the purpose of the investigation.
- Reporting of results.

The specific procedure for geotechnical investigation of a particular site will depend on the geographical and geological conditions and nature of the proposed construction. A timely and intelligently planned site exploration should be considered a pre – requisite for efficient, safe, economical design and construction.

The following general procedure should conduct for site investigation.

a) Desk study

b) Site reconnaissance

c) Subsurface investigation

4.2.1.1 Desk study

In general, any investigation should start with the collection and examination of the existing data on soil and rock and any available geological information relating to the site. Terrain conditions on the proposed site must also be studied. A desk study should typically include collection of geological and engineering geological data through geologic maps, structural geology maps as shown in Appendix B, previous reports, study of aerial photographs, satellite images, and topographic maps.

4.2.1.2 Site reconnaissance

This should involve a walk-over or drive-over of the site area in order to study the character and variability of the ground, select appropriate site investigation methods and visually classify any existing soil or rock exposures.

On going over the site, the study of the following features may be useful: local topography, excavations, cuttings, quarries, evidences of erosion, landslides, fills, water levels in wells and streams, flood marks, and drainage patterns, etc. If there has been an earlier use of the site, information should be gathered in particular about the location of fills and excavations. The present land use should be noted along with any constraints on access for exploration equipment.

4.2.1.3 Subsurface Investigation

The methods of site investigation are largely dependent upon the nature of the ground to be investigated and engineering practices. Adequate subsurface investigation should be carried out prior to the design and construction of the proposed development. In some instances it may be appropriate that this is done before acquiring a building site or making other investments dependent upon a particular site. The procedures for investigation, sampling, and testing shall be in accordance with the appropriate ASTM or AASHTO standards.

4.2.1.3.1 Purpose of Subsurface Investigation

The purpose of subsurface investigation is to assess the nature and sequence of the subsurface soils and rocks; groundwater conditions and the physical and mechanical properties of the subsurface materials.

Some typical examples of the purpose of subsurface investigations are as follows:-

a) To establish suitable horizontal and vertical location of a proposed structure on the site.

b) To locate and evaluate borrow materials for construction of earth embankments for highways or an earth dam.

c) To locate and evaluate sands and gravels suitable for highway aggregate, concrete aggregate, filter material or slope.

d) To determine the need for subgrade or foundation treatment to support loads or to control water movement.

e) To estimate foundation settlement or evaluate the stability of slopes or foundation.

4.2.1.3.2 Methods of Subsurface Investigation.

Some of the following methods may be selected and applied during the site investigation depending on the requirements of the proposed structures. Detailed procedures and the applicability of each method are described in Appendix C.

A summary of their applications is shown in Table 4.2.1.3.2.1.

a) Open Trial Pits (Test Pits) Method

b) Auger Boring (Hand Auger Method)

c) Shell and Auger Boring

d) Wash Boring

e) Standard Penetration Test

f) Cone Penetration Test

g) Rotary Boring

h) Percussion (or) Churn Boring

Table 4.2.1.3.2.1 Exploration objectives and suggested applicable methods

Sr. No.	Purpose of Exploration	Open Trial Pit/ Test pit	Hand Auger Boring	Shell & Auger boring	Wash Boring	SPT	CPT	Rotary Boring	Percussion Boring
1.	Faults	√						√	
2.	Deep – land							√	√
3.	Shallow – land	√	√	√	√	√	√		√
4.	Subaqueous							√	
5.	Soft-soil depth		√	√	√	√	√	√	√
6.	Sliding masses	√	√	√	√		√		
7.	Rock depth							√	√
8.	Rock-mass conditions							√	
9.	Disturbed Soil samples	√	√	√	√	√		√	
10.	Representative Soil samples	√	√	√	√	√		√	
11.	Undisturbed Soil samples	√	√	√	√			√	
12.	Rock cores							√	
13	1 to 2 - Storeyed Buildings (3 - 9 m Depth)	√	√	√	√	√	√	√	√
14.	2 to 6 - Storeyed Buildings (9 - 20 m Depth)	√	√	√	√	√	√	√	√
15.	(7 to 8 - Storeyed Buildings (20 - 30 m Depth)				√	√	√	√	√
16.	High – rise Buildings (9 – Storeyed and above) (30 - 80 m Depth)				√	√	√	√	√

Note: The depths mentioned in Table 4.2.1.3.2.1 are only the minimum requirements. Site investigation should be carried out to sufficient extent and depth to establish the significant soil strata and ground variation. Boreholes should go more than 5 meters into hard stratum with SPT blow counts of 100 or more than 3 times pile diameters beyond the intended founding level.)

4.2.1.3.2.2 Hydrogeological Investigation

Groundwater is generally collected and moves in interconnected voids, pore spaces, cracks, fissures, joints, bedding planes and other openings in soil and rock formations beneath the ground surface. The level of the water table is not stationary. It fluctuates according to the rainfall or seasons. The hydrogeological condition of a proposed site potentially has a great effect on foundation design consideration.

For investigation of the groundwater table, the following methods are suggested to be applied and types of water table should be described according to Appendix D.

4.2.1.3.2.2.1 Method of Groundwater Table Investigation

4.2.1.3.2.2.1.1 Logging at Test Borehole

One day after drilling, the standing groundwater level is usually measured by a level indicator or measuring tape. If drilling of the test borehole is continued on another day, the water table must be measured before and after drilling.

4.2.1.3.2.2.1.2 Installation of Piezometer and Logging

After the completion of drilling the test borehole to the required depth, a piezometer can be installed inside a PVC slotted screen and can be used to measure the water table for an extended period.

4.2.1.3.2.2.1.3 Permeability Tests and Filtration of Groundwater

Some of the following tests may be applied according to the client's requirements.

1) **Permeability**

   These are primarily seepage tests:

   a) Variable head test

   b) Constant head test

The variable head test is typically carried out in cased boreholes below the ground water table or in slow draining soils. The borehole is filled with water and rate of fall is measured against time. An alternative approach is to bail water from the hole and record the rate of rise in the water level until the rise becomes negligible.

The constant head test is typically carried out in unsaturated granular soils. The ground around the hole is saturated by adding metered quantities of water to the hole until the quantity decreases to a steady value. Water continues to be added to maintain a constant level recording the quantity of water added at regular intervals.

2) **Packer Test**

   The general test involves installing packers in boreholes and expanding them with air pressure to seal off sections of the borehole. Water under pressure is introduced between the packers and the elapsed time and volume of water pumped into the rock mass is noted. Curves of flow versus pressure are plotted and approximate values for the rock mass permeability can be estimated.

   One of two procedures is used depending on rock quality. The common procedure, used in poor to moderately poor rock with hole collapse problems, involves drilling the hole to some depth and performing the test with a single packer. Casing is installed if necessary and the hole is advanced to the next test depth. In good quality rock where the hole remains open, the hole is drilled to the final depth and testing proceeds in sections from the bottom up with two packers. Packer spacing depends on rock quality.

3) **Pumping Test**

Pumping tests are usually carried out in gravity wells or artesian wells in soil and rock. The well is pumped a a constant rate until a cone of drawdown measured in observation wells has stabilized (recharge equals the pumping rate). Values of the soil or rock mass permeability can be obtained from the test results.

4.2.1.3.2.3 Geotechnical Instrumentation

The primary requirement of any instrument is that it should be capable of determining a required parameter, such as water pressure, or displacement, without leading to a change in that parameter as a result of the presence of the instrument in the soil. Instrumentation for displacement measurement and pore water pressure and groundwater level measurements are presented in Appendix E.

4.2.1.4 Number and Position of Borehole locations.

The number of boreholes and their locations depend on the following:

1) type and size of the project
2) budget for site investigation
3) soil variability

Factors to be considered in test site selection include the proposed development layout, the site geology and access constraints. The extent to which these factors influence, test site selection will depend on the specific details of each project. A number of examples are described as follows:

1) For large buildings, boreholes should be located close to the proposed foundations;
2) For an industrial plant with a few items of heavy plant and numerous items of light plant, a grid pattern for most of the site would be appropriate with additional holes located at the proposed heavy plant sites;
3) Test pits should not be located in the intended footing positions for houses because of the weakening of the ground by the excavation. Where the layout of a proposed housing estate is known, future road alignments, property boundaries or service line easements (prior to installation of services) are preferred locations to building envelopes for test pits.
4) For bridges, boreholes should be located at the proposed abutment and pier locations.
5) For railways and highways in steep terrain, target boreholes locations should include proposed cuts;

Access is a factor which can influence test site location. Access onto peat swamps, tidal flats and mangrove areas can be very difficult. These are also areas which have a high probability of containing soft unstable soils, which will create problems for future development. Consideration as to how to gain access to such areas should be given at the proposal stage of a project. Existing buildings spoil piles or drainage channels may also prevent drill rig or excavator access.

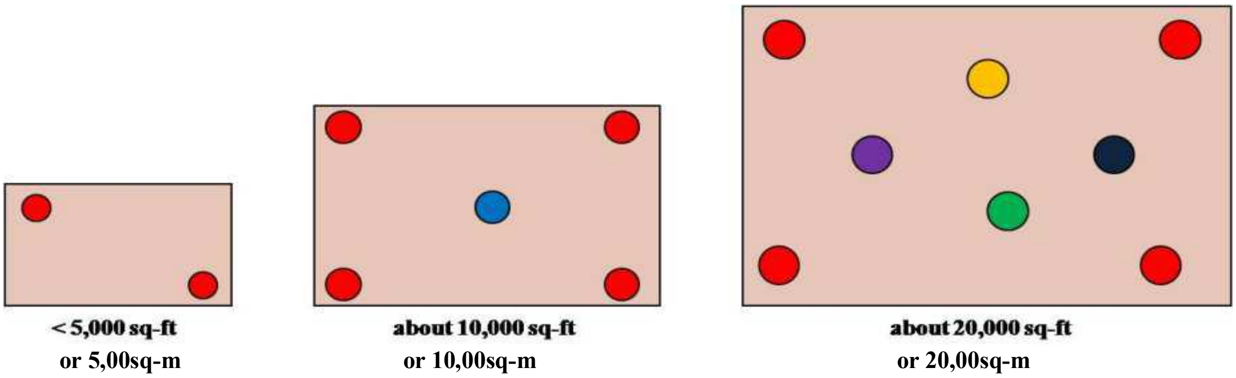

Figure 4.2.1.4 Suggested numbers, position and locations of Minimum Number of Boreholes

Suggested number and locations of boreholes are shown in Table 4.2.1.4 (1) and 4.2.1.4 (2).

Table 4.2.1.4 (1) Suggested number and locations of boreholes

Project	Distance between borings (meter) Horizontal Stratification of Soil and Rock			Minimum number of Boreholes for each structures
	Uniform	Average	Erratic	
3- 8 story buildings	50	30	15	4
1 -2-storey buildings	60	30	15	3

The borehole spacing suggested by the Committee for Quality control of High-rise building construction Projects (C.Q.H.P) (minimum number of borings = 2 boreholes) is as follows:

1) One boring for every 2500 sq-ft (or) 250 sq-m of built-over area < 10,000 sq-ft (or) 1,000 sq-m.
2) One boring for every extra 5,000 sq-ft (or) 500 sq-m for large area projects>10,000 sq-ft (or) 1,000 sq-m.
3) Additional borings for irregular soil conditions.

Table 4.2.1.4 (2) Suggested number and locations of boreholes

Area for Investigation	Boring Spacing (meter) / Boring number
New site of wide extent	borings 60 to 150 m apart
site on soft compressible strata	30 to 60 m at building locations.
Large structure (closely spaced footings)	Space borings 15 m in both directions , foundation walls at machinery or elevator
Low-load warehouse (large area)	Minimum of 4 borings at the corners
Isolated foundations 250 to 10,00sq-m	Minimum of 3 borings around perimeter
Isolated rigid foundation, < 250 sq-m	Min: of 2 borings at opposite corners
Major waterfront structures (drydocks)	space borings generally < 15 m. Add critical locations, deep pumped well, gate seat, tunnel, or culverts.
Long bulkhead or wharf wall.	borings on line of wall at 60 m spacing.
Slope stability, deep cuts, embankments	3 to 5 borings on line in critical direction
Water retaining structures	Space preliminary borings 60 m, cutoff, critical abutment

4.2.1.4.1 Depth of Boring.

Sowers (1979) suggested specified depth criteria as follows:

Minimum depth of borings $= 10\ S^{0.7}$ (ft) or $3\ S^{0.7}$ (m) (for narrow and light buildings)

Minimum depth of borings $= 20\ S^{0.7}$ (ft) or $6\ S^{0.7}$ (m) (for wide and heavy buildings)

where, S = the number of stories in the building

The Committee for Quality control of High-rise building construction Project (CQHP) suggested the following depth specifications.

1) Shallow foundations specified depth as 1.5 times lesser dimension ($B < L$) (Limit to 30ft (or) 10 m minimum)
2) Deep foundations: Minimum depth of borings = $15\ S^{0.7}$ (ft) or $5\ S^{0.7}$ (m) (Limit to 3 consecutive SPT values ≥ 50)

(**Note:** As stated in Clause 3.4.2 of ACI 336.3R-93 (reapproved in 1998) for Design and construction of Drilled Piers, Boring depth should be adequate to investigate settlement of the bearing stratum below the pier. Where practical, at least one boring should go into bedrock.)

(Site investigation should be carried out to sufficient extent and depth to establish the significant soil strata and ground variation. (a) The number of boreholes should be the greater of (i) one borehole per 300sq m or (ii) one borehole at every interval between 10m to 30m, but not less than 3 boreholes in a project site. (b) Boreholes should go more than

5 meters into hard stratum with SPT blow counts of 100 or more than 3 times pile diameters beyond the intended founding level.)

4.2.1.5 Geophysical Methods

The existing methods and techniques of geophysical exploration can be adapted with some modifications to most targets of environmental and engineering interest (shallow depth). In principle, all the geophysical techniques that have been advised for subsurface investigations essentially detect a discontinuity; that is, one underground region differs sufficiently from another in some physical properties such as density, magnetic susceptibility, elasticity, spontaneous polarization, electric resistivity and conductivity, dielectric permittivity, radioactivity and thermal conductivity and so on. The common geophysical methods using in engineering practice are as follows and their summary of application is shown in Table 4.2.1.5.

1) Seismic Survey (Hammering, Blasting)
2) Electrical Methods (Resistivity, IP, SP, VES)
3) Magnetic Survey
4) Electromagnetic Survey (TDEM)
5) Gravity Survey
6) Radioactivity Survey

A detailed explanation of each method is presented in Appendix F.

Table 4.2.1.5 Common geophysical methods and its application

Sr. No.	Purpose of Investigation	Magnetic	Gravity	Electrical				Electromagnetic			Seismic	
		Normal	Normal	SP	VES	ER	IP	VLF	GPR	TEM	Reft	Refr
1.	Subsurface Cavities					√						
2.	Clays, Peat and Soil			√		√			√			
3.	Construction Sites								√			√
4.	Fissures zones in rock					√		√	√			
5.	Fracture zones in rock	√	√			√			√			√
6.	Geological Mapping	√	√									
7.	Groundwater in crystalline rock					√		√				
8.	Groundwater in sedimentary areas				√							
9.	Groundwater flow			√								
10.	Overburden thickness				√							√
11.	Pollution of soil and groundwater					√						
12.	Saltwater invasion					√	√					
13.	Sand deposits								√			√

4.2.1.6 Sampling and Testing

The frequency of sampling and testing in an investigation depends on the information that is already available about the ground conditions and the technical objectives of the investigation.

4.2.1.6.1 Methods of Sampling.

1) In Standard Penetration Test (SPT) (ASTM D1586-99), use the split-spoon sampler for disturbed samples and use the piston tube steel sampler for undisturbed samples.
2) Hand augering method for undisturbed samples.
3) Test pitting for disturbed and undisturbed samples
4) Rotary drilling (manual) for s for disturbed and undisturbed samples and disturbed samples.

Some special techniques of sampling and testing are shown in the following Table 2.3.12 (1).

Table 4.2.1.6.1 (1) Some special techniques of sampling and insitu testing

Type of Ground	Special Techniques
Sand	SPT, CPT, sand samplers
Soft Sensitive Clay	CPT, Flat Plate Dilatometer, borehole or penetration shear vane, thin wall open tube or piston sampler, continuous soil sampler, large diameter samplers
Hard Stony Clay	Plate bearing test, pressure meter, rotary core sampling
Rock	Pressure meter, rotary core drilling using larger core sizes than 70 mm diameter

4.2.1.6.1.1 Soil samples

The selection of a sampling technique depends on the quality of the sample that is required and the character the ground, particularly the extent of disturbance by the sampling process. In choosing a sampling method, it should be made clear whether the mass properties or the intact materials properties of the ground are to be determined.

Sampling methods will differ according to the types of building for which the investigation is being carried out. The following procedure should be followed during sampling.

1) For low rise buildings, houses and sites where the soil profile is expansive, the upper about 1 – 3 m of the profile is critical and sufficient samples should be recovered from this depth interval to characterize the founding conditions.
2) Samples, both disturbed and undisturbed, should be taken in every 1m (more samples will be required if there is a lithologic change within 1m).

3) The collected samples must be sent urgently to the laboratory within two days, or the samples must be stored well enough to maintain their natural conditions.

The weight of sample must be as shown in Table 4.2.1.6.1 (2)

Table 4.2.1.6.1 (2) The recommended weight of soil samples for different tests (BS 5930-99)

Purpose of Sample	Soil	Mass of Sample Required (kg)
Soil identification, including Atterberg's Limits, Sieve analysis, Moisture content test	Clay, Silt, Sand	1
	Fine and Medium Gravel	5
	Coarse Gravel	30
Compaction Tests	All	25 – 60
Comprehensive examination of construction materials, including soil stabilization	Clay, Silt, Sand	100
	Fine and Medium Gravel	130
	Coarse Gravel	160

Complete detailed descriptions must be included on each sample bag as follows:

1) Location (GPS), Project name
2) Sample No.
3) Depth from where to where
4) Collector's name
5) Date of sampling
6) Investigation and Sampling Method

4.2.1.6.1.1.1 Disturbed and Undisturbed Samples.

1) The Standard Penetration Test (SPT) splitspoon sampler may be used for disturbed samples but the piston tube steel sampler or thin wall tube sampler must be used for undisturbed samples.
2) The hand augering method is suitable for obtaining disturbed samples. Undisturbed samples require the use of a thin wall tube sampler.
3) Test pitting is suitable for disturbed samples and for obtaining undisturbed samples either by cutting blocks of soil or using a thin wall tube sampler.
4) Rotary drilling (manual) is suitable for obtaining both disturbed and undisturbed core samples

4.2.1.6.1.1.2 Representative samples

The samples collected at a proposed site should be representative of the natural conditions of the soil such as natural moisture content and density and should be free from remolding effects.

4.2.1.6.1.2 Rock Samples

It is well known that rocks masses are non-homogeneous and the properties of samples taken from one portion of the rock may be different from those taken from another location. Therefore sampling should be properly done to represent the rock mass.

Samples can usually be collected from the field in the form of large blocks in the case of surface and near surface deposits by breaking from the parent body manually using steel hammers. When the deposit is deep under the ground, the samples can be obtained in the form of cores obtained from diamond drilling and large blocks may be available from blasting.

The following procedure should be followed during sampling.

1) Store the samples in the Core-box.
2) On the core-box, a detailed description must be included as follows:

(i) Borehole No., Location (GPS), Project Name.

(ii) Depth from where to where

(iii) Logger's name

(iv) Rock Quality Designation (RQD)

(v) Total Core Recovery (T.C.R)

(vi) Solid Core Recovery (S.C.R)

(vii) Core recovery (%)

(viii) No. of fractures (FI)

3) One core-box must have at least 3m of core run.
4) Cores must be placed in the core-box as soon as the drilling is finished and core losses must be shown systematically as well.
5) Photographs of core samples must be taken after placing the core in the core-box.
6) The core samples in the box must be covered with plastic sheets.
7) Core-boxes with core samples must be locked and stored in cool place.
8) The core samples for laboratory tests must be handled well, placed carefully onto PVC tubing, sealed with wax and covered with plastic sheets.

4.2.1.6.2 Protection, Handling, Labeling of Samples

4.2.1.6.2.1 Protection and Handling.

Samples may cost a considerable sum of money to obtain and should be treated with great care. Ideally, samples should be moisture-proofed immediately after collection either by waxing, spraying, or packing in polythene bags or sheets. They should be transported and stored under cover, and generally protected from excessive changes in humidity and temperature. **Temperature must be between 20º C and 45º C.** The usefulness of laboratory test results depends on the quality of samples at the time they are tested.

4.2.1.6.2.2 Labeling

All samples should be labeled with a unique reference number immediately after being collected. The label should show all necessary information about the sample (site name, borehole number, depth, top and bottom of a core, etc) and should also be recorded on the daily field report. The label should be marked with indelible ink and be sufficiently robust to withstand the effects of the environment and transportation.

4.2.1.6.3 Visual Examination and Description of Laboratory Samples

Information about the grading and plasticity of soils can be estimated from visual inspection of bulk samples obtained during drilling and from tube samples. Information about the structures and fabric of soils cracks in rocks can be visually examined from high quality samples.

The description of samples of soil and rock tested in the laboratory forms an important part of the record of the test results. Such descriptions should be included on the laboratory work sheet. Descriptions of samples noted in the laboratory should be compared with the equivalent field descriptions and any anomalies should be resolved.

4.2.2.2 Various Tests

1) Grain Size analysis

i) Dry – sieve analysis to 75 microns

ii) Wet – sieve analysis for soil less than 75 microns

   - Pipette method
   - Hydrometer analysis

2) Water content and dry unit weight of soil
3) Determination of specific gravity
4) Determination of consistency of soil
5) Shear strength tests
6) Compaction test
7) California Bearing Ratio (CBR) Test
8) Permeability tests
9) Consolidation tests
10) Dispersibility tests
11) Other tests

i) Vane shear test

ii) Swelling pressure test

iii) Free swell test

iv) Linear shrinkage test

4.2.2.3 Result Presentation.

A common sample form for presentation of test results is shown in Appendix G.

4.2.3 Soil and Rock Classification

4.2.3.1 Classification of soil according to ASTM D-2487-00.

A soil classification system divides soil into groups and sub-groups based on common engineering properties such as the grain-size distribution, liquid limit and plastic limit. The major classification system presently in use is the American Society for Testing Materials (ASTM). The ASTM system was originally proposed by A. Casagrande in 1942 and was later revised and adopted by the United States Bureau of Reclamation and U.S Army Corps of Engineers in 1969. The system is used extensively in geotechnical works. The symbols and terminology shown in Table 4.2.3.1 (1) and Table 4.2.3.1 (2) are used for identification. Classification of non – plastic and plastic soils and sample forms of classification are shown in Tables 4.2.3.1 (3), 4.2.3.1 (4) and 4.2.3.1 (5).

Table 4.2.3.1 (1) Terminology used to denote percentage by weight of each component

Descriptive Term	Range of Proportion
Trace (eg. trace sand, trace clay)	1 – 9 %
Some (eg. Some sand, some clay)	10 – 19 %
Adjective (eg. Sandy, silty)	20 – 34 %
Major soil (eg. SAND, CLAY, SILT)	≥ 35 %

Table 4.2.3.1 (2) Symbols for Soil Identification in ASTM System

Symbol	Description
G	Gravel
S	Sand
M	Silt
C	Clay
O	Organic Silts and Clay
Pt	Peat and highly organic soils
H	High plasticity
L	Low Plasticity
W	Well graded
P	Poorly graded

Terminologies used to indicate the compactness and consistency of disturbed materials are described in the following tables.

Table 4.2.3.1 (3) Compactness of non – plastic soil based on SPT values

Compactness of non – plastic soil	SPT values Blows / foot (or) Blows / 0.305 m	Relative Density (%)
Very Loose	0 – 4	0 – 20
Loose	4 – 10	20 – 40
Medium Dense	10 – 30	40 – 70
Dense	30 – 50	70 – 90
Very Dense	> 50	> 90

Table 4.2.3.1 (4) Consistency of plastic soil based on UCS values and SPT values

Consistency of plastic soil	Range of Unconfined Compressive Strength			SPT values Blows /foot (or) Blows / 0.305 m
	(psf)	(KN/Sq.meter)	(Ton/Sq.ft)	
Very soft	0 – 500	< 25	< 0.25	0 – 2
Soft	500 – 1000	25 – 50	0.25 – 0.5	2 – 4
Medium Stiff (firm)	1000 – 2000	50 – 100	0.5 – 1.0	5 – 12
Stiff	2000 – 4000	100 – 200	1.0 – 2.0	12 – 25
Very Stiff	4000 – 6000	200 – 300	2.0 – 3.0	25 – 40
Hard	6000 – 8000	300 – 400	3.0 – 4.0	40 – 50
Very Hard	> 8000	> 400	> 4.0	> 50

The classification of soil at the proposed site should be described as shown in the following sample form.

Table 4.2.3.1 (5) Sample form of soil classification

Compactness or Consistency of soil	Colour	Soil Type	Others (If have)
eg. Very Dense	Reddish Brown	SAND and SILT	With lime powder
eg. Stiff	Yellowish Grey	Silty CLAY	With broken bricks

Unified Soil Classification System (USCS) which is adopted as ASTM D-2487-00 is also application for classification of soil. The plasticity chart and characteristics of soil groups with the group symbols for various types of soil in USCS are shown in Appendix H.

4.2.3.2 Classification of Soil According to $\bar{v}_s^{30}$ (Seismic Site Classification)

For seismic design consideration, the soil is generally classified based on their average shear wave velocity of upper 30 m depth, $\bar{v}_s^{30}$, of soil layers as shown in Table 4.2.3.2.

$$\bar{v}_s^{30} = \frac{\sum_{i=1}^{n} t_i}{\sum_{i=1}^{n} t_i / v_{si}}$$

where,

$\bar{v}_s^{30}$ = Average S-wave velocity in upper 30 m depth

n = Number of soil layers

t_i = Thickness of i^{th} soil layer

v_{si} = S-wave velocity of i^{th} soil layer

Table 4.2.3.2 Seismic Site Classification

Site Class	$\bar{v}_s^{30}$	$\bar{N}$
E (Soft Soil)	< 600 ft/s (< 175 m/s)	< 15
D (Medium Dense Soil)	600 – 1,200 ft/s (175 – 350 m/s)	15 – 50
C (Dense Soil)	1,200 – 2,500 ft/s (120- 250 m/s)	> 50

4.2.3.3 Classification of Construction Materials

4.2.3.3.1 Materials for Concrete Aggregate

Material suitable for use as concrete aggregate, shall comply with the requirements of ASTM C-33-03.

4.2.3.3.2 Materials for backfills.

Materials which are classified within Group (a) in Table 4.2.3.3.2 are suitable for use as backfill materials. Materials which are classified as Group (b) in Table 4.2.3.3.2 are unsuitable for use as backfill but may be designated for other uses.

Various types of backfills are shown in Table 4.2.3.3.2.

Table 4.2.3.3.2 Various Types of Backfills

DESCRIPTION OF BACKFILL MATERIAL	UNIFIED SOIL CLASSIFICATION
Group (a)	
Well-graded, clean gravels; gravel-sand mixes	GW
Poorly graded clean gravels; gravel-sand mixes	GP
Silty gravels, poorly graded gravel-sand mixes	GM
Clayey gravel, poorly graded gravel-and-clay mixes	GC
Well-graded, clean sands; gravelly sand mixes	SW
Poorly graded clean sands; sand-gravel mixes	SP
Silty sands, poorly graded sand-silt mixes	SM
Sand-silt clay mix with plastic fines	SM-SC
Clayey sands, poorly graded sand-clay mixes	SC
Inorganic silts and clayey silts	ML
Mixture of inorganic silt and clay	ML-CL
Inorganic clays of low to medium plasticity	CL
Group (b)	
Organic silts and silt clays, low plasticity	OL
Inorganic clayey silts, elastic silts	MH
Inorganic clays of high plasticity	CH
Organic clays and silty clays	OH

4.2.4 Seismic Design Category

4.2.4.1 Site class definitions

The design categories A, B, C, D, E, F shall be classified based on the average shear wave velocity at upper 30 m (100 ft) depth, standard penetration resistance, and un-drained shear strength of soil in accordance with Section **3.4.1** of Structural Design.

When the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines that Site Class E or F soil is likely to be present at the site.

4.2.4.2 Spectral Response Acceleration Parameters

4.2.4.2.1 Preparation of Maps of Response Acceleration of 0.2s and 1s

The parameters S_s and S_1 shall be determined from the prepared 0.2 and 1-second spectral response acceleration maps or shall be determined based on seismic source to site distance, magnitude of designed earthquake, focal depth and v_s^{30} for a particular site of interest. Where S_1 is less than or equal to 0.04 and S_s is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A.

4.2.4.2.2 Values of Site Coefficient F_a

According to Section **3.4.1** of Structural Design.

4.2.4.2.3 Values of Site Coefficient F_v

According to Section **3.4.1** of Structural Design.

4.2.4.3 Soil Amplification

1D seismic response analysis by using equivalent linear method should be used to obtain the amplification factor of underlying soil of a proposed site. Amplification is the ratio of amplitude of the objective layer to those of the reference layer, and is a function with respect to frequency.

For high – rise buildings (generally starting from 9 – storeys), the calculated amplification based on soil data from the proposed site should be used for design consideration.

4.2.4.4 Fundamental Frequency and Predominant Period

Fundamental Frequency or Predominant Period of underlying soil is one of the important parameters in seismic resistant design consideration. Buildings with similar natural period of resonance and resonance frequency to those of the supporting soil can be expected to suffer severe damage during an earthquake.

The frequency or period that is corresponding to peak soil amplification is usually regarded as the Fundamental Frequency or Predominant Period of that soil for that site. These parameters can be obtained through seismic response analysis.

4.2.4.5 Seismic Response Analysis

This is the most important and most reliable approach that can be conducted for seismic resistant design of a structure. It is a simulation based on the specific soil parameters of the site, the shear wave velocity structure and generated or recorded bedrock motion.

Seismic Response Analysis will give all the required parameters for the design of various structures, including high – rise buildings, such as peak ground acceleration, peak ground velocity, peak ground displacement, amplification factor, fundamental frequency and predominant period. The general procedure for 1D seismic response analysis is shown in Appendix I.

4.2.5 Report Preparation and Geotechnical Criteria.

Where buildings in Myanmar have 9 storeys are greater they are regarded as high – rise buildings. For such buildings, the geotechnical investigation report should be prepared according to the instructions of the Committee for Quality Control of High – Rise Buildings. These instructions are summarized as follows.

The site investigation, laboratory testing and report shall include the following components:

a) Environmental study of the site and surrounding area.

b) A site location plan and a site area plan with the number of boreholes, depths, elevation and their locations marked on the plan.

c) A subsurface investigation and sampling of the foundation soils shall be carried out in accordance with the standard methods adopted in Myanmar for high-rise buildings.

d) The following tests shall be carried out as appropriate for the site conditions.

   1) Visual soil classification

   2) Tests for Moisture content and density of all soil samples

   3) Grain size analysis for selected samples

   4) Atterberg’s Limits Tests (liquid limit, plastic limit, and plasticity index) for semi-plastic and plastic soil.

5) Unconfined compressive strength test for semi-plastic soils and plastic soils.

6) Direct shear test for selected soil samples or triaxial compression test for selected soil samples.

7) Specific gravity test for selected soil samples.

8) Consolidation test for semi-plastic soils or plastic soil samples for shallow foundation.

e) Seismicity of the area, liquefaction, predominant period, fundamental frequency, and amplification (site effects) of soils at a proposed site should be included.

f) For high – rise buildings, seismic response analysis has to be included for seismic-resistant design practices.

g) Soil profiles with standard soil descriptions, depths, elevation, groundwater levels and the results of in-situ testing such as the standard penetration tests (SPT) and/or vane shear tests shall be provided for each borehole. If rock was encountered, the report should include a description of the rock including the degree of weathering and fracturing, compressive strength and rock quality designation (RQD) if core was recovered.

h) A geological description of the site shall be provided.

i) Recommendations shall be presented for alternative types of foundations along with founding depths and any precautions that are relevant.

j) For shallow foundations, recommended allowable bearing capacity values at various depths should be provided for each borehole profile..

k) The results of any other appropriate tests and other general recommendations for foundation design and construction should also be presented in the report.

l) Photographs of the site, and site investigation should also be included.

m) The report should include a discussion of the advantages and disadvantages of the proposed site with respect to the proposed high-rise development.

For other geotechnical investigation reports (9 – storeyed and above), the sampling method mentioned in above will keep on applying. For lower buildings (less than 9 – storeyed), proper types of sampling methods (e.g, standard drilling method) may apply and soil sampling methods must be included in report. In soil reports, standards of sampling and standard for tests must be clearly described.

4.3 EXCAVATION, GRADING AND FILL

4.3.1 Excavations near footing or foundations

Excavations for any purpose shall not remove lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation.

4.3.2 Placement of backfill

Excavations outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or shall be backfilled with a controlled low-strength material (CLSM).The backfill shall be placed in lifts and compacted in a manner that does not damage the foundation, the waterproofing or the damp-proofing material.

4.3.3 Site grading

The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5 percent slope)for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstruction or allotment boundaries prohibit 10 feet (3048mm) of horizontal distance, a 5 percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building.

4.3.4 Grading and Filling in Flood Hazard Areas

1) Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.
2) In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses, performed by a registered design professional in accordance with standard engineering practice, that the proposed grading or fill, or both, will not result in increased flood levels during the occurrence of the design flood.
3) In flood hazard areas subject to high velocity wave action, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.
4) Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed flood hazard area encroachment, when combined with all other existing and anticipated flood hazard area encroachments, will not increase the design flood elevation more than 1 foot (305 mm) at any point.

4.3.5 Compacted Fill Material

Where footings bear onto compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain the following.

1) Specifications for the preparation of the site prior to placement of compacted fill material.
2) Specifications for material to be used as compacted fill.
3) Test methods to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4) Maximum allowable thickness of each lift of compacted fill material.
5) Field test methods for determining the in – place dry density of the compacted fill.

6) The minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.

7) The number and frequency of field tests required to determine compliance with Item 6.

4.3.6 Controlled Low-Strength Material

Where 1) Specifications for the preparation of the site prior to placement of the CLSM.

2) Specifications for the CLSM.

3) Laboratory or field test method (s) to be used to determine the compressive strength or bearing capacity of the CLSM.

4) Test methods for determining the acceptance of the CLSM in the field.

5) Number and frequency of field tests required to determine compliance with Item 4.

footings will bear on controlled low- strength material (**CLSM**), the **CLSM** shall comply with the provisions of an approved report, which shall contain the following:

4.3.7 Soil improvement

Suitable improvement methods should be applied according to the requirements of proposed sites.

4.4 DESIGN RECOMMENDATIONS FOR SOILS AND ROCKS

4.4.1 Basic Design Concepts for Expansive Soil and Black Cotton Soil

Most of the expansive soils likely to be encountered in Myanmar have formed residually over rocks, particularly basalts. These soils generally contain a large percentage of active clay content. Montmorillonite is the predominant clay mineral in expansive soils. Tropical expansive soils, often called Black Cotton Soil (in Myanmar), are major foundation problems in America, Africa, and Asia. The term "Black Cotton Soil" is believed to have originated in India where the locations of these soils are favorable for growing cotton. All expansive soils have a high water holding capacity and it is not possible to completely dry the soil in oven at a temperature of 110º C. The high percentage of clay is responsible for high volumetric changes during wetting and drying. The soil in the dry state can often have high strength, but the soils become soft when in the wet state. During summer and dry seasons, the soil shrinks volumetrically (in three dimensions).

The thickness of black cotton soils are typically in the range of about 4meter depth from ground surface.

The depth at which structures are founded in black cotton soils depends on such factors as the structural design, amount of permissible movement, soil characteristics and profile, and expected moisture variations over the life of the structure.

The use of mats, stiffened rafts, strip footings or individual pad footings will require special evaluation of site specific soil-structure interaction, design of structural elements accordingly and/or special soil treatment or replacement methods.

Where the thickness of expansive soil is less than 2 meter, the observed expansive soil layer could be removed and replaced with suitable non-reactive material. . However, if the thickness of the expansive soil layer is more than 2meter., it may be appropriate to treat the layer by stabilization using cement or lime or some other special treatment.

Moisture variation in expansive soils should be minimized by implementing the following essential points:

a) Providing a good drainage system around the outside of buildings.

b) Avoiding sewage pipes passing near the buildings either surface or subsurface and leading such pipes directly away from the structure at right angles to exterior walls. Such pipes and connections should be well designed to avoid danger of leakage.

c) Keeping taps and other water connections in gardens and walls away from the structure.

d) Planting trees at a distance equal to the mature height of the trees. They should not be planted closer to the structures. Many shrubs also absorb large quantities of moisture from a soil and can cause volume changes of expansive soils.

e) Providing good ventilation and drainage below a suspended floor - this can be helpful in maintaining moisture equilibrium.

f) Paving areas around the structures – this is often advantageous in maintaining a uniform moisture content beneath the structure. Adequate insulation by membranes, such as asphalt or asphalted fiber glass, is helpful in protecting the moisture losses of soil and penetration of surface water.

g) Protecting and backfilling of all foundation excavations in expansive soil areas without delay in order to minimize changes in the natural moisture regime.

h) Providing an appropriate layout and type of ground beams for such soil.

i) Some engineers suggested that safe soil bearing capacity of expansive soils should be taken just over swelling pressure of that soil.

4.4.2 Basic Design Concepts for Potential Landslide Areas

Slope stability analysis should first be performed t in such areas to establish whether various stability measures have to be performed to ensure the stability of buildings.

The main causes that influence landslide potential in Myanmar are: (i) gravity and the gradient of the slope, (ii) hydrogeological characteristics of the slope, (iii) presence of troublesome earth material, (iv). processes of erosion, (v) man-made causes, (vi) geological conditions, and (vii) occurrence of a triggering event.

The general procedure for slope stability analysis is presented in Appendix J.

4.4.3 Strength Parameters of Soils and Rocks

To assist with footing design, the strength parameters for soils and rocks which are encountered at the proposed sites have to be determined. The angle of internal friction, 'ϕ' and cohesion, 'c' are two main shear strength parameters of soils and rocks and are dependent on many factors such as; the types of soil and rock, moisture content, the presence of micro fractures, rate of loading, permeability, stress history and so on. The shear strength, compressive strength and tensile strength of soil and rock have to be calculated by using any available methods and approaches. Both results from in-situ tests in the field and results from laboratory tests have to be considered.

4.4.4 Lateral Earth Pressure *(Both Static and Dynamic)*

The seismic behavior of earth retaining structures depends on the total lateral earth pressure that develops during an earthquake. These total pressures include both static gravitational pressure that exists before an earthquake occurs and the dynamic pressure induced by the earthquake. The lateral earth pressure for retaining structures needs to be calculated for both static and dynamic loading conditions.

For static earth pressure (both active and passive pressures), Rankine Theory (1857) or Coulomb Theory (1776) or Logarithmic spiral method or Stress – Deformation Analysis such as finite element analysis should be conducted.

Calculation of seismic lateral earth pressure on a retaining structure is one of the important applications of the pseudo-static (quasi-static) seismic inertial force. For calculation of dynamic earth pressure (both active and passive pressures), Mononobe – Okabe method (1929) or Steedman – Zeng method (1990) or finite element analysis should be performed.

4.4.5 Design Parameters (Static Load)

The following parameters have to be determined and submitted to the design engineers for consideration in the design for static loading conditions.

a) Natural water content in soil

b) Gravity and Density

c) Dry Unit Weight

d) Specific Gravity of soil particles

e) Natural Void Ratio

f) Saturation

g) Water Ratio

h) Liquid limit

i) Plastic Limit

j) Plastic Ratio

k) Liquid Ratio

l) Coefficient of Compression

m) Compression Modulus

n) Angle of Internal Friction

o) Cohesion

p) Coefficient of Collapsible (Angle of repose)

q) Initial Pressure of Collapsible

(To evaluate stability, informality, bearing capacity of foundation)

4.4.6 Seismic Design Parameters (Seismic Load)

For seismic – resistant design, considerations of high – rise buildings and infrastructures, the following parameters are recommended.

4.4.6.1 Average shear wave velocity of upper 30 m depth, $\bar{v}_s^{30}$

The evaluation of strong motions and site effects of local soil conditions requires information on shear wave velocity especially for areas where thick sediment layers are overlying bedrock.

a) The average shear wave velocity at upper 30 m depth, ($\bar{v}_s^{30}$) will be used for classification of seismic design categories as in Structural Design Section 4.2.3.2.

b) $\bar{v}_s^{30}$ will be used for determination of soil amplification factor during an earthquake.

c) $\bar{v}_s^{30}$ will also be used seismic response analysis for determination of soil amplification factor during an earthquake.

The average shear wave velocity of the top 30 m, v_s^{30} can be evaluated by some geophysical methods or by SPT results.

4.4.6.2 Spectral Response acceleration

The response acceleration will be determined from maps or from empirical calculation by using some attenuation relationships or from response analysis for high – rise buildings and important public buildings.

4.4.6.3 PGA, PGV, PGD

The peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGD) of soil will be calculated by response analysis and should be used in seismic resistant design considerations.

4.4.6.4 Amplification of Soil

The amplification of soil should be calculated by response analysis for high – rise buildings and where thick sediment layers are observed.

4.4.6.5 Fundamental Frequency and Predominant Period of Soil.

The fundamental frequency and predominant period of soil will be calculated by response analysis and should be used for seismic resistant designs.

4.5 FOOTINGS AND FOUNDATIONS

4.5.1 General

4.5.1.1 Allowable Load Bearing Values of Soils

4.5.1.1.1 Presumptive load-bearing values.

The presumptive load-bearing values provided in Table 4.5.1.1.1 shall be used with the allowable stress design load combinations specified in Structural Design Section 2.1.3. The maximum allowable foundation pressure, lateral pressure or lateral sliding-resistance values for supporting soils near the surface shall not exceed the values specified in Table 4.5.1.1.1 unless data to substantiate the use of a higher value are submitted and approved. Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions. Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

Exception: A presumptive load-bearing capacity is permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight and temporary structures.

4.5.1.1.2 Allowable load-bearing pressure by calculation

An assessment of the allowable load-bearing pressure shall be based on the engineering properties of the soil, that is, cohesion, angle of internal friction, density, etc. The bearing capacity shall be calculated from stability considerations of shear and a factor of safety of 2.5 shall be adopted for safe bearing capacity. The potential effects of interference of adjacent foundations should be taken into account.

The procedure for determining the ultimate bearing capacity and allowable bearing pressure of shallow foundations based on shear and allowable settlement criteria shall be calculated by approved analytical, numerical or empirical methods. The bearing pressure beneath a stiff foundation may be assumed to be distributed linearly. The distribution of bearing pressure beneath a flexible foundation may be derived by modeling the foundation as a beam or slab resting on a deforming continuum or series of springs, with appropriate stiffness and strength.

Table 4.5.1.1.1 Allowable Foundation and lateral pressure

CLASS OF MATERIALS	ALLOWABLE FOUNDATION PRESSURE (psf) [d]	LATERAL BEARING (psf/below natural grade) [d]	LATERAL SLIDING	
			Coefficient of friction[a]	Resistance (psf) [b]
1. Crystalline bedrock	12,000	1,200	0.7	-
2. Sedimentary and foliated rock	4,000	400	0.35	-
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	-
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel	2,000	150	0.25	-
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	500 [c]	30	-	40

For SI: 1 pound per square foot = 0.0479 kPa.
1 pound per square foot per foot= 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.
b. Lateral sliding resistance value to be multiplied by the contact area.
c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 500 psf, the allowable bearing capacity shall be determined by a soils investigation.
d. An increase of 1/3 is permitted when using the alternate load combinations in structural design section that include wind or earthquake loads.

4.5.1.1.3 Lateral sliding resistance

Where the loading is not normal to the foundation base, foundations shall be checked against failure by sliding on the base. The resistance of structural walls to lateral sliding shall be calculated by combining the values derived from the lateral bearing and the lateral sliding resistance shown in Table 4.5.1.1.1 unless data to substantiate the use of higher values are submitted for approval. For clay, sandy clay, silty clay and clayey silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

4.5.1.1.3.1 Increases in allowable lateral sliding resistance

The resistance values derived from the table are permitted to be increased by the tabular value for each additional foot (305 mm) of depth to a maximum of 15 times the tabular value.

4.5.1.2 Settlement

4.5.1.2.1 Design consideration for settlement calculation

Calculations of settlements shall include both immediate and long-term settlement. The following three components of settlement should be considered for partially or fully saturated soils:

1) immediate settlement; for fully-saturated soil due to shear deformation at constant volume, and for partially-saturated soil due to both shear deformation and volume reduction;
2) settlement caused by consolidation;
3) settlement caused by creep.

Special consideration should be given to soils such as organic soils and sensitive clays, in which settlement may be prolonged almost indefinitely due to creep. For estimating settlement in soils, the depth to be considered will depend on the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements. For individual pad footings this depth may be roughly estimated as 2 times the foundation width and may be up to 4 times the foundation width for strip footings. The depth may be reduced for lightly-loaded, wider foundation such as rafts. This approach is not valid for very soft soils. Any possible settlement caused by self-weight compaction of the soil, flooding and vibration in fill and collapsible soils shall be assessed.
The permissible values of total and differential settlement for a given type of structure may be taken as given in Table 4.5.1.2.1.

4.5.1.3 Modulus of Sub-grade Reaction, k_s.

It is defined as the pressure applied by the footing divided by the resulting settlement.

$$k_s = \frac{q}{\delta}$$

Where test data are not available, the modulus of subgrade reaction $k_s$shall be reasonably estimated by following formula.

k_s = 12 (SF) q_a (SI)

Values of $k_s$may also be estimated using Table 4.5.1.3.

Table 4.5.1.3Range of Values of Modulus of Subgrade Reaction,*ks*

Soil	K_s,kcf	K_s, KN/m^3
Loose sand	30-100	4800-16000
Medium dense sand	60-500	9600-80000
Dense sand	400-800	64000-128000
Clayey medium dense sand	200-500	12000-80000
Silty medium dense sand	150-100	24000-48000
Clayey soil:		
qa≤ 200 kPa (4 ksf)	75-150	12000-24000
200≤qa≤400 kPa	150-300	24000-48000
qa≥800 kPa	>300	>48000

Note: Use values as guide and for comparison when using appropriate equation

4.5.1.4 Liquefaction

Liquefaction is the sudden and large decrease of shear strength of a submerged cohesionless soil caused by contraction of the soil structure, produced by shock or earthquake-induced shear strains, associated with a sudden but temporary increase of pore water pressures. Liquefaction occurs when the increase in pore water pressures causes the effective stress to become equal to zero and the soil behaves as liquid. Liquefaction potential should be evaluated if site parameters meet the following criteria.

1) Soil having less than 15% of the particles, based on dry weight, that are finer than 0.005 mm (% finer than 0.005 mm < 15%)
2) Soil having a liquid limit (LL) less than 35. (LL < 35)
3) Soil having water content *w* greater than 0.9 of the liquid limit. (*w*> 0.9 LL)
4) Soil below the groundwater table
5) Site having potential of a peak ground acceleration a_{max} greater than 0.10g or local magnitude 5 or larger

The Factor of Safety (FS) against liquefaction shall be defined as

FS = CRR / CSR

where

CRR = Cyclic Resistance Ratio of the in situ soil

CSR = Cyclic Stress Ratio of the in situ soil

$$\text{CSR} = 0.65 r_d \left(\frac{\sigma_{vo}}{\sigma_{vo}}\right)\left(\frac{a_{max}}{g}\right)$$

where

a_{max} = peak ground acceleration of the site

g = acceleration of gravity (32.2 ft/s2 or 9.81 m/s2)

σ_{vo} = total vertical stress at a particular depth where the liquefaction analysis is being performed.

σ'_{vo} = vertical effective stress at a particular depth where the liquefaction analysis is being performed.

r_d = depth reduction factor or stress reduction coefficient

r_d = $1 - 0.00366\, z$ (z in feet)

r_d = $1 - 0.012\, z$ (z in meter)

CRR shall be determined by Figure 4.5.1.4 which has been developed for an earthquake magnitude of 7.5 and for other different magnitudes, the CRR values shall be multiplied by the magnitude scaling factor indicated in Table 4.5.1.4.

Table 4.5.1.2.1 Permissible Differential Settlement and Tilt

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 mm	Maximum settlement mm	Differential settlement mm	Angular distortion mm	Maximum settlement mm	Differential settlement mm	Angular distortion mm	Maximum settlement mm	Differential settlement mm	Angular distortion mm
(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	50	.001 5*L*	1/666	75	.001 5*L*	1/666	75	.002 1*L*	1/500	100	.002 0*L*	1/500
iii)	For multistoreyed buildings												
	a) RC or steel framed buildings with panel walls	60	.002*L*	1/500	75	.002*L*	1/500	75	.002 5*L*	1/400	125	.003 3*L*	1/300
	b) For load bearing walls												
	1) *L*/H = 2*	60	.000 2*L*	1/5 000	60	.000 2*L*	1/5000	← Not likely to be encountered →					
	2) *L*/H = 7*	60	.000 4*L*	1/2 500	60	.000 4*L*	1/2500						
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.

Table 4.5.1.4 Magnitude Scaling Factor

Anticipated earthquake magnitude	Magnitude Scaling Factor (MSF)
8 ½	0.89
7 ½	1.00
6 ¾	1.13
6	1.32
5 ¼	1.50

Note: To determine the Cyclic Resistance Ratio of the in situ soil, multiply the magnitude scaling factor indicated above by the Cyclic Resistance Ratio determined from Fig. 4.5.1.4.

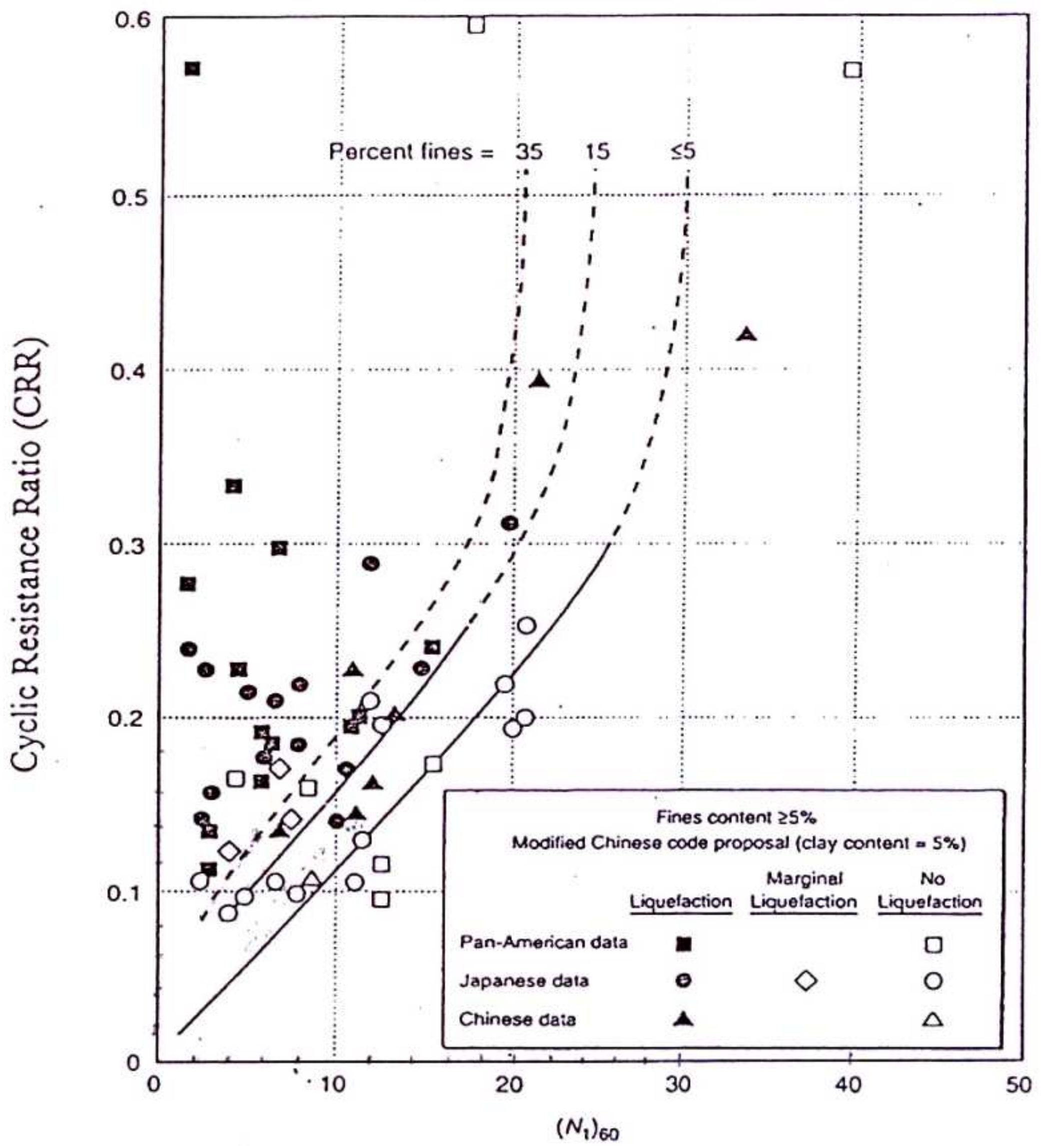

Figure 4.5.1.4 Cyclic Resistance Ratio of the in Situ Soil

4.5.1.5 General Construction Requirements

4.5.1.5.1 Concrete strength

Concrete in footings shall have a specified compressive strength (f'c) of not less than 2,500 pounds per square inch (psi) at 28 days.

4.5.1.5.2 Footing seismic ties

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, individual spread footings founded on soil defined in Section 1613.5.2 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient, SDS, divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

4.5.1.5.3 Plain concrete footings

The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil.

Exception: For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

4.5.1.5.4 Placement of concrete

Concrete footings shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water.

4.5.1.5.5 Protection of concrete

Concrete footings shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

4.5.1.5.6 Forming of concrete

Concrete footings are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require forming. Where forming is required, it shall be in accordance with Chapter 6 of ACI 318.

4.5.1.6 Damp Proofing and Waterproofing

4.5.1.6.1 General

Walls or portions that retain earth and enclose interior spaces and floors below grade shall be damp proofed and water proofed in accordance with this section. Groups other than residential or institutional, omission of damp proofed and water proofed for those spaces are not detrimental effect to the building or occupancy.

4.5.1.6.1.1 Story above grade plane

Where a basement is considered a story above graded plane and the basement floor and wall is partially below the finished ground level for 25 percent or more of the perimeter, the floor and walls shall be damp proofed and a foundation drain shall be installed. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level.

4.5.1.6.1.2 Under floor space

Unless an approved drainage system is provided ,the ground level of the under-floor space shall be as high as the outside finished ground level where the ground water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site.

4.5.1.6.1.2.1 Flood hazard areas

For buildings and structures in flood hazard areas, the finished ground level of an under-floor space such as crawl space shall be equal to or higher than the outside finished ground level.

4.5.1.6.1.3 Ground-water control

The floor and walls shall be damp proofed, where the ground water table is lowered and maintained at an elevation not less than 6 inches (152 mm) below the bottom of the lowest floor. The design of the system to lower the ground-water table shall be based on accepted principles of engineering.

4.5.1.6.2 Damp proofing

Floors and walls shall be damp proofed where the ground-water investigation indicates that a hydrostatic pressure will not occur.

4.5.1.6.2.1 Floors

Damp proofing materials for floors shall be installed between the floor and the base course, except where a separate floor is provided above a concrete slab. Damp proofing materials shall be used locally available materials or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

4.5.1.6.2.2 Walls

Damp proofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level. Damp proofing materials for walls shall be used locally available materials or other approved materials.

4.5.1.6.2.2.1 Surface preparation of walls

All the holes and recesses on the concrete walls shall be sealed by bituminous material or other approved methods or materials prior to the application of damp proofing materials. Unit masonry walls shall be parged on the exterior surface below ground level with not less than 0.375 inches (10 mm) of Portland cement. The parging shall be coved at the footing.

Exception: Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

4.5.1.6.3 Water proofing

Floors and walls shall be water proofed where the ground-water investigation indicates that a hydrostatic pressure condition exists, and design does not include a ground-water control system.

4.5.1.6.3.1 Floors

Concrete floors are required to be water proofed and designed and constructed to resist the hydrostatic pressures to which the floors will be subjected. Waterproofing shall be accomplished by placing a membrane of locally available materials or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instruction.

4.5.1.6.3.2 Walls

Concrete walls and masonry walls are required to be water proofed and shall be designed and constructed to withstand hydrostatic pressures and other lateral loads to which the wall be subjected. Water proofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground-water table. The remainder of the wall shall be damp proofed. Water proofing materials for walls shall be used locally available materials or other approved materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instruction.

4.5.1.6.3.2.1 Surface preparation of walls

The walls shall be prepared prior to application of waterproofing materials on concrete or masonry walls.

4.5.1.6.3.3 Joints and penetrations

Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made water-tight utilizing approved methods and materials.

4.5.1.6.4 Subsoil drainage system

Where a hydrostatic pressure condition does not exist, damp proofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter.

4.5.2 Shallow Foundation

4.5.2.1 Design Information and Consideration

4.5.2.1.1 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 Seismic zone of the region.

f) Seismic zone of the region.

4.5.2.1.2 Design Consideration

4.5.2.1.2.1 Design

Footings shall be designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized.

4.5.2.1.2.2 Design loads

Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Structural Design Section. The dead load is permitted to include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Structural Design Section, are permitted to be used in the design of footings. Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the footing design to prevent detrimental disturbances of the soil.

4.5.2.2 Depth of Foundations

The minimum depth of foundations below the undisturbed ground surface shall be 24 inches (609mm). 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. Where shallow sub-soils are of a shifting or moving character, foundation shall be carried to a sufficient depth to ensure stability.

4.5.2.2.1 Foundation at Different Levels.

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 24 inches (609 mm) for rock and 36 inches (914 mm) for soil (see Figure 4.5.2.2.1 (1)).

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 30° (see Figure 4.5.2.2.1 (2)).

c) In case of footing of 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 30° (see Figure 4.5.2.2.1 (3)).

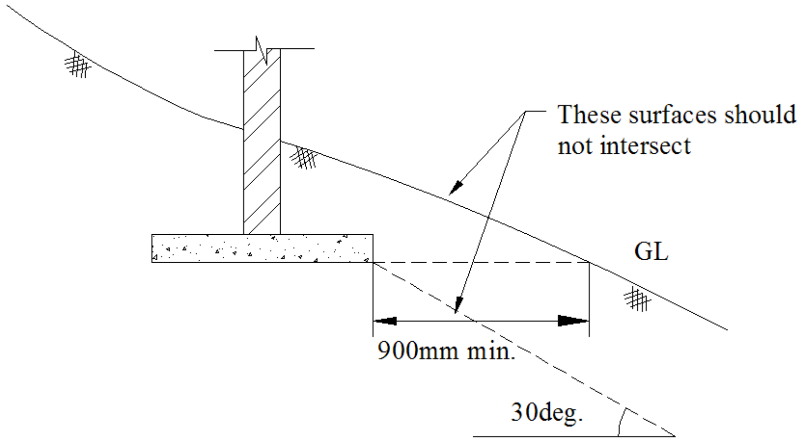

Figure 4.5.2.2.1 (1) Footing in Sloping Ground

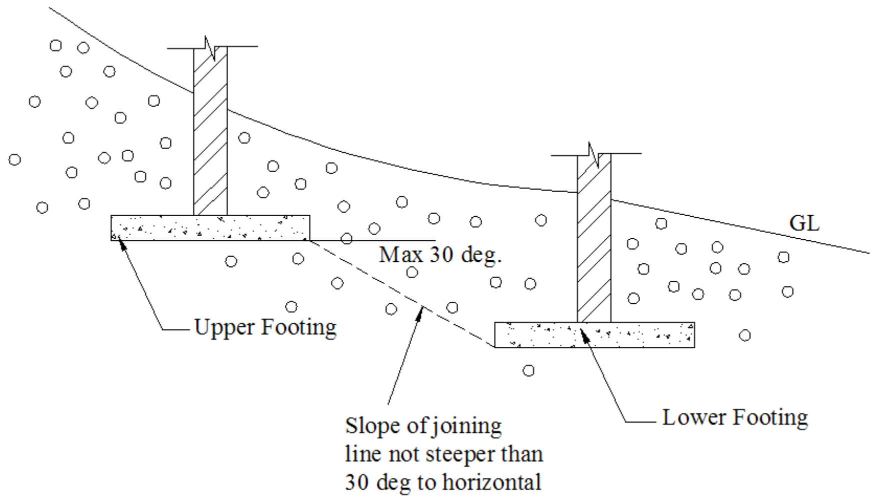

Figure 4.5.2.2.1 (2) Footing in Granular Soil

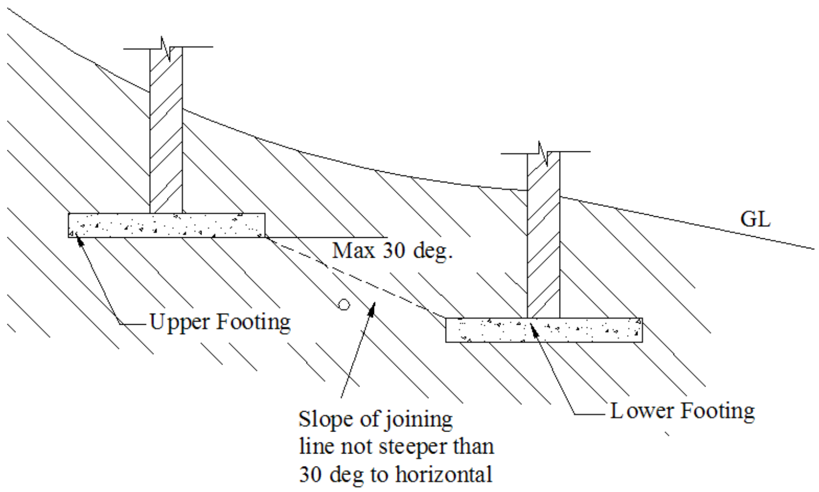

Figure 4.5.2.2.1 (3) Footing in Clayey Soil

4.5.2.3 Foundation on or adjacent to slopes

The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall conform to Sections 4.5.2.3.1 through 4.5.2.3.5.

4.5.2.3.1 Building clearance from ascending slopes

In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage erosion and shallow failures. Except as provided for in Section 4.5.2.3.5 and Figure 4.5.2.3.1 and 4.5.2.3.2, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

4.5.2.3.2 Footing setback from descending slope surface

Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 4.5.2.3.5 and Figure 4.5.2.3.1 and 4.5.2.3.3, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

4.5.2.3.3 Pools

The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

4.5.2.3.4 Foundation elevation

On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

4.5.2.3.5 Alternate setback and clearance

Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slop gradient, load intensity and erosion characteristics of slope material.

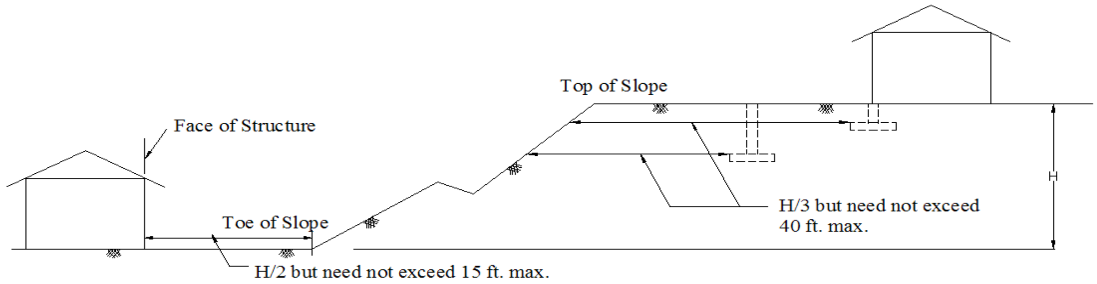

Figure 4.5.2.3.1 Foundation Clearances from Slopes

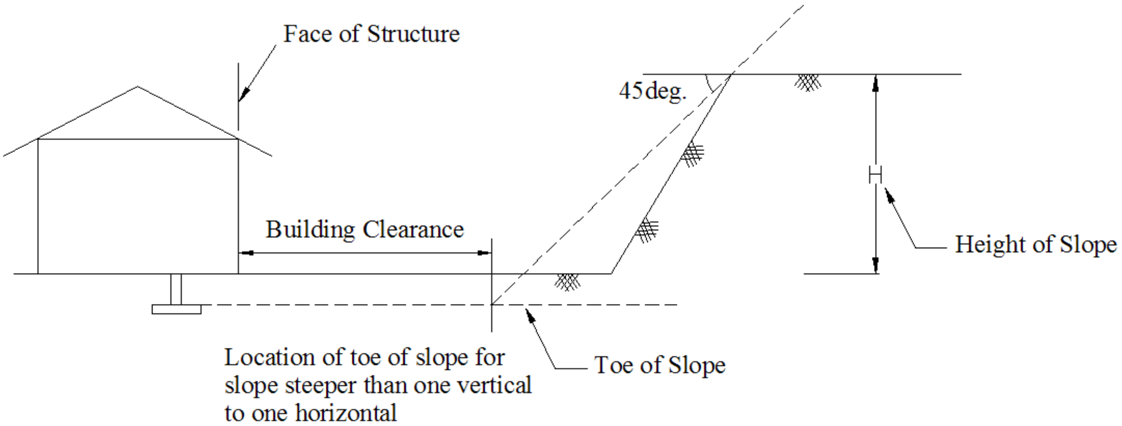

Figure 4.5.2.3.2 Foundation Clearances from Ascending Slopes

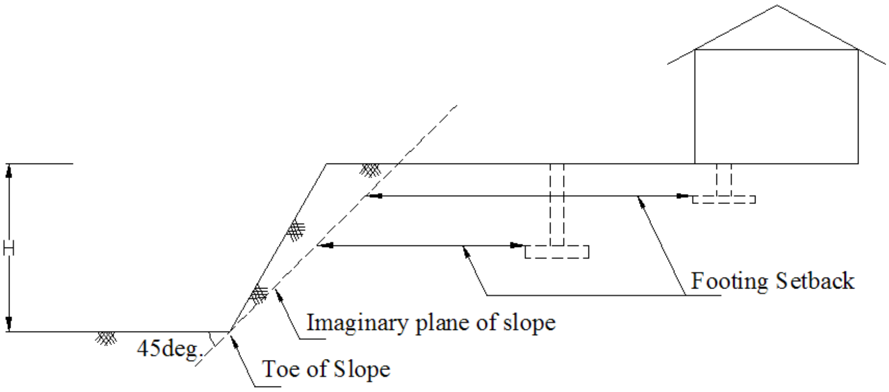

Figure 4.5.2.3.3 Foundation Setback from Descending Slopes

4.5.2.4 Spread Foundations

4.5.2.4.1 Pad Foundation (Isolated Footing)

For buildings such as low rise dwellings and lightly framed structures, pad foundations may be of unreinforced concrete provided that the angle of spread of load from the column or base plate to the outer edge of the ground bearing does not exceed one vertical to ½ horizontal for masonry or one vertical to one horizontal for cement concrete and that the stresses in the concrete due to bending and shear do not exceed permissible stresses. Where brick or masonry foundations have been used, the same rules shall apply.

For buildings other than low rise and lightly framed structures, it is customary to use reinforced concrete foundations. The thickness of the foundation should under no circumstances be less than 6 inches (152 mm) and will generally be greater than this to maintain cover to reinforcement where provided. Where concrete foundations are used they should be designed in accordance with the code of practice appropriate to the loading assumptions.

4.5.2.4.2 Strip foundations

Similar considerations to those for pad foundations apply to strip foundations. On sloping sites strip foundations should be on a horizontal bearing, stepped where necessary to maintain adequate depth.

4.5.2.4.2.1 Continuous wall foundations

In continuous wall foundations it is recommended that reinforcement be provided wherever an abrupt change in magnitude of load or variation in ground support occurs. Continuous wall foundations will normally be constructed in mass concrete provided that the angle of spread of load from the edge of the wall base to the outer edge of the ground bearing does not exceed one (vertical) in one (horizontal). Foundations on sloping ground, and where re-grading is likely to take place, may require to be designed as retaining walls to accommodate steps between adjacent ground floor slabs or finished ground levels. At all changes of level unreinforced foundations should be lapped at the steps for a distance at least equal to the thickness of the foundation or a minimum of 12inches (300 mm). Where the height of the step exceeds the thickness of the foundation, special precautions should be taken. The thickness of reinforced strip foundations should be not less than 6 inches (152 mm), and care should be taken with the excavation levels to ensure that this minimum thickness is maintained.

For the longitudinal spread of loads, sufficient reinforcement should be provided to withstand the tensions induced. It will sometimes be desirable to make strip foundations of inverted tee beam sections, in order to provide adequate stiffness in the longitudinal direction. At corners and junctions the longitudinal reinforcement of each wall foundation should be lapped.

4.5.2.5 Raft foundations

Suitably designed raft foundations may be used in the following circumstances.

a) For lightly loaded structures on soft natural ground where it is necessary to spread the load, or where there is variable support due to natural variations, made ground or weaker zones. In this case the function of the raft is to act as a bridge across the weaker zones. Rafts may form part of compensated foundations.

b) Where differential settlements are likely to be significant. The raft will require special design, involving an assessment of the disposition and distribution of loads, contact pressures and stiffness of the soil and raft.

c) Design of the raft and structure to accommodate subsidence requires consideration by suitably qualified persons; the effects of mining may often involve provision of a flexible structure.

d) When buildings such as low rise dwellings and lightly framed structures have to be erected on soils susceptible to excessive shrinking and swelling, consideration should then be given to raft foundations placed on fully compacted selected fill material used as replacement for the surface layers.

e) For heavier structures where the ground conditions are such that there are unlikely to be significant differential settlements or heave, individual loads may be accommodated by isolated foundations. If these foundations occupy a large part of the available area they may, subject to design considerations, be joined to form a raft.

4.5.2.6 Short Piling

Where it is necessary to transmit foundation loads from buildings such as low rise dwellings or lightly framed structures through soft or made ground, or unstable formations or shrinking/swelling clays more than about 6.5 ft (2000 mm) deep, the use of short piles should be considered as an alternative to shallow foundations, particularly where a high groundwater table is encountered. The type, method of construction, size and load capacity should be carefully considered in relation to the associated requirements of pile caps and ground beams necessary to transfer loads from the superstructure to the piles.

4.5.3 Deep Foundation

4.5.3.1 General requirements

4.5.3.1.1 General

Pier and pile foundations shall be designed and installed on the basis of a foundation investigation as defined in section 4.2, unless sufficient data upon which to base the design and installation is available.

The investigation and report provisions of Section 4.2 shall be expanded to include, but not be limited to, the following:

1) Recommended pier or pile types and installed capacities.

2) Recommended center-to-center spacing of piers or piles.

3) Driving criteria.

4) Installation procedures.

5) Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).

6) Pier or pile load test requirements.

7) Durability of pier or pile materials.

8) Designation of bearing stratum or strata.

9) Reductions for group action, where necessary.

4.5.3.1.2 Special types of piles

The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.

4.5.3.1.3 Pile caps

Pile caps shall be of reinforced concrete, and shall include all elements to which piles are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of piles shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.

4.5.3.1.4 Stability

Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centroid of the group not less than 60 degrees apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official. Piles supporting walls shall be driven alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories or 35 feet (10668 mm) in height, provided the centers of the piles are located within the width of the foundation wall.

4.5.3.1.5 Structural integrity

Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of piles being installed or already in place.

4.5.3.1.6 Splices

Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pier or pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Splices shall develop not less than 50 percent of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 10 feet (3048 mm) of the embedded portion of the pier or pile shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the pier or pile load of 3 inches (76 mm), or the pier or pile shall be braced in accordance with Section 4.5.3.1.4 to other piers or piles that do not have splices in the upper 10 feet (3048 mm) of embedment.

4.5.3.1.7 Allowable pier or pile loads.

4.5.3.1.7.1 Determination of allowable loads

The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.

4.5.3.1.7.2 Driving criteria

Allowable compressive load on any pile shall be determined by the application of an approved driving formula. Allowable loads shall be verified by load tests in accordance with Section 4.5.3.1.7.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven piles. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

4.5.3.1.7.3 Load tests

Where design compressive loads per pier or pile are greater than those permitted or where the design load for any pier or pile foundation is in doubt, control test piers or piles shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test pier or pile as assessed by one of the published methods with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with settlement analysis. In subsequent installation of the balance of foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are of the same type, size and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and, for driven piles, where the rate of penetration (e.g.,net displacement per blow) of such piles is equal to or less than that of the test pile driven with the same hammer through a comparable driving distance.

4.5.3.1.7.3.1 Load test evaluation

It shall be permitted to evaluate pile load tests with any of the following methods:

1) Davisson Offset Limit.
2) Brinch-Hansen 90% Criterion.
3) Butler-Hoy Criterion.
4) Other methods approved by the building official.

4.5.3.1.7.3.2 Non-destructive testing

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to construction of beam or caps.

4.5.3.1.7.4 Allowable frictional resistance

The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material as set forth in Table 4.1, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official after a soil investigation, is submitted or a greater value is substantiated by a load test.

4.5.3.1.7.5 Uplift capacity

Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 4.5.3.1.7.3 divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1) The proposed individual pile uplift working load times the number of piles in the group.
2) Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

4.5.3.1.7.6 Load-bearing capacity

Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

4.5.3.1.7.7 Bent piers or piles

The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative pier or pile.

4.5.3.1.7.8 Overloads on piers or piles

The maximum compressive load on any pier or pile due to mis-location shall not exceed 110 percent of the allowable design load.

4.5.3.1.8 Lateral support

4.5.3.1.8.1 General

Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

4.5.3.1.8.2 Unbraced piles

Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

4.5.3.1.8.3 Allowable lateral load

Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25.4 mm) at the ground surface.

4.5.3.1.9 Use of higher allowable pier or pile stresses

Allowable stresses greater than those specified for piers or for each pile type are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1) A soils investigation in accordance with Section 4.2.
2) Pier or pile load tests in accordance with Section 4.5.3.1.7.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

4.5.3.1.10 Piles in subsiding areas

Where piles are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the piles by the subsiding upper strata. Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter are permitted to be increased where satisfactory substantiating data are submitted.

4.5.3.1.11 Negative Skin Friction or Down Drag Force

When a soil stratum, through which a pile shaft has penetrated into an underlying hard stratum, compresses as a result of either its being unconsolidated or its being under a newly placed fill or as a result of re-moulding during driving of the pile, a drag down force is generated along the pile shaft up to a point in depth where the surrounding soil does not move downwards relative to the pile shaft. Recognition of the existence of such a phenomenon shall be made and a suitable reduction shall be made to the allowable load, where appropriate.

4.5.3.1.12 Settlement analysis

The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

4.5.3.1.13 Pre-excavation

The use of jetting, augering or other methods of pre-excavation shall be subject to the approval of the building official. Where permitted, pre-excavation shall be carried out in the same manner as used for piers or piles subject to load tests and in such a manner that will not impair the carrying capacity of the piers or piles already in place or damage adjacent structures. Pile tips shall be driven below the pre-excavated depth until the required resistance or penetration is obtained.

4.5.3.1.14 Installation sequence

Piles shall be installed in such sequence as to avoid compaction the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.

4.5.3.1.15 Use of vibratory drivers

Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 4.5.3.1.7.3. The installation of production piles shall be controlled according to power consumption, rate of penetration or other approved means that ensure pile capacities equal or exceed those of the test piles.

4.5.3.1.16 Pile driveability

Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.

4.5.3.1.17 Protection of pile materials

Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the piles so as not to be rendered ineffective by driving. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

4.5.3.1.18 Use of existing piers or piles

Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the piers or piles are sound and meet the requirements of this code. Such piers or piles shall be load tested or redriven to verify their capacities. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests or re-driving data.

4.5.3.1.19 Heaved piles

Piles that have heaved during the driving of adjacent piles shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the pile shall be verified by load tests in accordance with Section 4.5.3.1.7.3.

4.5.3.1.20 Identification

Pier or pile materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

4.5.3.1.21 Pier or pile location plan

A plan showing the location and designation of piers or piles by an identification system shall be filed with the building official prior to installation of such piers or piles. Detailed records for piers or individual piles shall bear an identification corresponding to that shown on the plan.

4.5.3.1.22 Spacing of Piles

The centre to centre spacing of a pile is considered from two aspects as follows:

a) Practical aspects of installing the piles; and

b) The nature of the load transfer to the soil and possible reduction in bearing capacity of a group of piles thereby.

In the case of piles founded on a very hard stratum and deriving their capacity mainly from end bearing, the spacing will be governed by the competency of the end bearing strata. The minimum spacing in such cases shall be 2.5 times the diameter of the shaft. In case of piles resting on rock, a spacing of two times the diameter may be adopted. Piles deriving their bearing capacity mainly from friction shall be sufficiently apart to ensure that the zones of soil 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 three times the diameter of the shaft. In the case of loose sand or filling, closer spacing than in dense sand may be possible, in driven piles since displacement during the piling may be absorbed by vertical and horizontal compaction of the strata. The minimum spacing in such strata may be two times the diameter of the shaft.

4.5.3.1.23 Special inspection

4.5.3.1.23.1 Pier foundations

Special inspections shall be performed during installation and testing of pier foundations as required by Table 4.5. The approved soils report, required by Section 4.2, and the documents prepared by the registered design professional in responsible charge shall be used to determine compliance.

4.5.3.1.23.2 Pile foundations

Special inspections shall be performed during installation and testing of pile foundations as required by Table 4.6. The approved soils report, required by Section 4.2, and the documents prepared by the registered design professional in responsible charge shall be used to determine compliance.

Table 4.5.3.1.23.2 (1) Required Verification and Inspection of Pier Foundations

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1.Verify pile materials, sizes and lengths comply with the requirements.	X	✓
2. Verify placement locations and plumbness, confirm pier diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end capacity.	X	✓
3. For concrete piers, perform additional inspections as specified in special inspection required for concrete construction	✓	✓
4. For masonry piers, perform additional inspections as specified in special inspection required for masonry construction	✓	✓

Table 4.5.3.1.23.2 (2) Required Verification and Inspection of Pile Foundations

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1.Verify pile materials, sizes and lengths comply with the requirements.	X	✓
2. Determine capacities of test piles and conduct additional load tests, as required.	X	✓
3. Observe driving operations and maintain complete and accurate records for each pile.	X	✓
4. Verify placement locations and plumbness, confirm type and size of hammer, record number of blows per foot of penetration, determine required capacity, record tip and butt elevations and document any pile damage.	X	✓
5. For steel piles, perform additional inspections as specified in special inspection required for steel construction	✓	✓
6. For concrete piles and concrete-filled piles, perform additional inspections as specified in special inspection required for concrete construction	✓	✓
7. For specialty piles, perform additional inspections as determined by the registered design professional in responsible charge.	✓	✓
8. For augured uncased piles and caisson piles, perform inspections in accordance with pier foundations.	✓	✓

4.5.3.1.24 Seismic design of piers or piles

4.5.3.1.24.1 Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Part 3 (Structural Design), the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, SDS, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade, reinforced concrete slabs on grade, confinement by competent rock, hard cohesive soils or very dense granular soils.

Exception: Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios of Group R-3 and U occupancies not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

4.5.3.1.24.1.1 Connection to pile cap

Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region. Ends of hoops, spirals and ties shall be terminated with seismic hooks, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns. For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

Exception: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile. Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Structural Design Section.

4.5.3.1.24.1.2 Design details

Pier or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a registered design professional. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid. Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center- to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cutoff.

4.5.3.1.24.2 Seismic Design Category D, Eor F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Structural Design (Section 1613), the requirements for Seismic Design Category C given in Section 4.5.3.1.24.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 4.5.3. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:1. Group R or U occupancies of light-framed construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2MPa) at 28 days. 2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.3. Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.

4.5.3.1.24.2.1 Design details for piers, piles and grade beams

Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites, as determined in Structural Design Section, shall be designed and detailed in accordance with Sections 21.4.5.1, 21.4.4.2 and 21.4.5.3 of ACI 318 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. For precast prestressed concrete piles, detailing provisions as given in Section 4.5.3.2.2.3.2.1 and 4.5.3.2.2.3.2.2 shall apply. Grade beams shall be designed as beams in accordance with ACI 318, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Structural Design Section, they need not conform to ACI 318, Chapter 21.

4.5.3.1.24.2.2 Connection to pile cap

For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:

1) In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations of Structural Design Section 4.1.5.
2) In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Structural Design Section 4.1.5 or development of the full axial, bending and shear nominal strength of the pile.

4.5.3.1.24.2.3 Flexural strength

Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Structural Design Section 4.1.5

4.5.3.2 Driven Pile Foundation

4.5.3.2.1 Timber piles

Timber piles shall be designed with the prevailing code. Only structural timber shall be used for piles.

4.5.3.2.1.1 Materials

Round timber piles shall conform to ASTM D 25.

4.5.3.2.1.2 Preservative treatment

Timber piles used to support permanent structures shall be treated unless it is established that the tops of the untreated timber piles will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative-treated timber piles shall be subject to a quality control program administered by an approved agency. Pile cutoffs shall be treated.

4.5.3.2.1.3 Defective piles

Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

4.5.3.2.1.4 Allowable stresses

The allowable stresses of timber pile shall not exceed values specified in Table 4.7.

Table 4.5.3.2.1.4 Allowable Working Stresses for Sawn Timbers (Psi)

Symbol	Description	Pyinkado	Teak	Padauk	In/Kanyin
F_b	Bending at fiber stress	2500	2000	2500	1500
F_v	Longitudinal shear	240	120	175	130
F_c	Axial compression	1900	1200	1700	760
F_{cb}	Axial compression when combine with bending	1900	1200	1700	760
$F_{c(per)}$	Compression perpendicular to grain	970	450	1050	400
$F_{t(par)}$	Tension parallel to grain where reduced by notches, daps, connectors or abrupt changes in section	1600	960	1350	610
$F_{t(par)}$	Tension parallel to grain where no stress concentration exists	1900	1200	1700	760
$F_{t(per)}$	Tension perpendicular to grain	60	40	60	60
E	Modulus of Elasticity	2.0+E6	1.44+E6	1.65+E6	1.3+E6

4.5.3.2.2 Precast concrete piles

4.5.3.2.2.1 The materials, reinforcement and installation of precast concrete piles

It shall conform to Sections 4.5.3.2.2.1 through 4.5.3.2.2.4.

4.5.3.2.2.1.1 Design and manufacture

Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

4.5.3.2.2.1.2 Minimum dimension

The minimum lateral dimension shall be 6 inches (152 mm).

4.5.3.2.2.1.3 Reinforcement

Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 4 inches (102 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends of the pile; and not more than 6 inches (152 mm) elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 1 inch (25.4 mm) center to center. The gage of ties and spirals shall be as follows:

For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller 6 mm.

For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 8 mm.

For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than 9 mm.

4.5.3.2.2.1.4 Installation

Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.

4.5.3.2.2.2 Precast non prestressed piles

Precast non prestressed concrete piles shall conform to Sections 4.5.3.2.2.2.1 through 4.5.3.2.2.2.5.

4.5.3.2.2.2.1 Materials

Concrete shall have a 28-day specified compressive strength (*f'c*) of not less than 3,000 psi (20.68 MPa).

4.5.3.2.2.2.2 Minimum reinforcement

The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.

4.5.3.2.2.2.2.1 Seismic reinforcement in Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. Longitudinal reinforcement with a minimum steel ratio of 0.01shall be provided throughout the length of precast concrete piles. Within three pile diameters of the bottom of the pile cap, the longitudinal reinforcement shall be confined with closed ties or spirals of a minimum 3/8 inch (10 mm) diameter. Ties or spirals shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, not to exceed 6 inches (152 mm). Throughout the remainder of the pile, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal bar diameter not to exceed 8 inches (203 mm).

4.5.3.2.2.2.2.2 Seismic reinforcement in Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C in Section 4.5.3.2.3.2.1 shall apply except as modified by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2 and liquefiable sites and where spirals are used as the 21.4.4.3 of ACI 318 within three pile diameters of the bottom of the pile cap. For other than Site Class E or F, or transverse reinforcement, a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.4.4.1(a) of ACI 318 shall be permitted.

4.5.3.2.2.2.3 Allowable stresses

The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (*f'c*) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (*fy*) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (*fy*) or a maximum of 24,000 psi (165 MPa).

4.5.3.2.2.2.4 Installation

A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (*f'c*), but not less than the strength sufficient to withstand handling and driving forces.

4.5.3.2.2.2.5 Concrete cover

Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm). Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars. Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).

4.5.3.2.2.3 Precast prestressed piles

Precast prestressed concrete piles shall conform to the requirements of Sections 4.5.3.2.2.3.1 through 4.5.3.2.2.3.5.

4.5.3.2.2.3.1 Materials

Prestressing steel shall conformto ASTM A 416. Concrete shall have a 28-day specified compressive strength (*f'c*) of not less than 5,000 psi (34.48 MPa).

4.5.3.2.2.3.2 Design

Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length. Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

4.5.3.2.2.3.2.1 Design in Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Structural Design Section 1613, the following shall apply. The minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

$$\rho_s = 0.12 f'c / f_{yh} \qquad \text{(Equation 4.5-1)}$$

where:

$f'c$ = Specified compressive strength of concrete, psi (MPa).

f_{yh} = Yield strength of spiral reinforcement $\leq$ 85,000 psi (586 MPa).

ρ_s = Spiral reinforcement index (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 4-1 shall be provided below the upper 20 feet (6096 mm) of the pile.

The pile cap connection by means of dowels as indicated in Section 4.5.3.1.24 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.

4.5.3.2.2.3.2.2 Design in Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C in Section 4.5.3.2.2.3.2.1 shall be met, in addition to the following:

1) Requirements in ACI 318, Chapter 21, need not apply, unless specifically referenced.

2) Where the total pile length in the soil is 35 feet (10668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.

3) In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 8 inches (203 mm), whichever is smaller.

4) Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of the spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Sec. 12.14.3 of ACI 318.

5) Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$\rho_s = 0.25(f'c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'c A_g)] \qquad \text{(Equation 4.5-2)}$$

but not less than:

$\rho_s = 0.12(f'c/f_{yh})[0.5 + 1.4P/(f'c\ A_g)]$ (Equation 4.5-3)

and need not exceed:

$\rho_s = 0.021$ (Equation 4.5-4)

where:

A_g = Pile cross-sectional area, square inches (mm^2).

A_{ch} = Core area defined by spiral outside diameter, square inches (mm2).

$f'c$ = Specified compressive strength of concrete, psi (MPa).

f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

P = Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-6.

ρ_s = Volumetric ratio (vol. spiral/ vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6) When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacings, and perpendicular to dimension, h_c, shall conform to:

$A_{sh} = 0.3sh_c\ (f'_c\ /f_{yh})(A_g\ /A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$ (Equation 4.5-5)

but not less than:

$A_{sh} = 0.12sh_c\ (f'c\ /f_{yh})[0.5 + 1.4P/(f'cA_g)]$ (Equation 4.5-6)

where:

f_{yh} = ≤ 70,000 psi (483 MPa).

h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).

s = Spacing of transverse reinforcement measured along length of pile, inch (mm).

A_{sh} = Cross-sectional area of transverse reinforcement, square inches (mm2).

$f'c$ = Specified compressive strength of concrete, psi (MPa).

The hoops and cross ties shall be equivalent to deformed bars not less than 10 mm in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

4.5.3.2.2.3.3 Allowable stresses

The allowable design compressive stress, fc, in concrete shall be determined as follows:

$fc = 0.33 f'c - 0.27f_{pc}$ (Equation 4.5-7) where:

$f'c$ = The 28-day specified compressive strength of the concrete.

f_{pc} = The effective prestress stress on the gross section.

4.5.3.2.2.3.4 Installation

A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f 'c), but not less than the strength sufficient to withstand handling and driving forces.

4.5.3.2.2.3.5 Concrete cover

Prestressing steel and pile reinforcement shall have a concrete cover of not less than 1 1/4 inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and 1 1/2 inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than 2 1/2 inches (64 mm).

4.5.3.2.3 Structural steel piles

Structural steel piles shall conform to the requirements of Sections 4.5.3.2.3.1 through 4.5.3.2.3.4.

4.5.3.2.3.1 Materials

Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform toASTMA36, ASTMA252, ASTMA283, ASTMA572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A992.

4.5.3.2.3.2 Allowable stresses

The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (Fy).

Exception: Where justified in accordance with Section 4.5.3.1.9, the allowable axial stress is permitted to be increased above 0.35Fy, but shall not exceed 0.5Fy.

4.5.3.2.3.3 Dimensions of H-piles

Sections of H-piles shall comply with the following:

1) The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.

2) The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).

3) Flanges and web shall have a minimum nominal thickness of 3/8 inch (10 mm).

4.5.3.2.3.4 Dimensions of steel pipe piles

Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 8 inches (203 mm). The pipe shall have a minimum cross section of 0.34 square inch (219 mm2) to resist each 1,000 foot-pounds (1356 N-m) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than

35,000 psi (241 Mpa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where pipe wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided.

4.5.3.3 Cast-In-Place Concrete Pile Foundations

4.5.3.3.1 General

The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 4.5.3.3.1.1 through 4.5.3.3.1.3.

4.5.3.3.1.1 Materials

Concrete shall have a 28-day specified compressive strength (*f'c*) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (203 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

4.5.3.3.1.2 Reinforcement

Except for steel dowels embedded 5 feet (1524 mm) or less in the pile and as provided in Section 4.5.3.3.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semi fluid state.

4.5.3.3.1.2.1 Reinforcement in Seismic Design Category C

Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. A minimum longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers or caissons in the top one-third of the pile length, a minimum length of 10 feet (3048 mm) below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum 3/8 inch (9 mm) diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcement with a maximum spacing of 6 inches (152 mm) or 8-longitudinal-bar diameters, whichever is less, shall be provided within a distance equal to three times the least pile dimension of the bottom of the pile cap.

4.5.3.3.1.2.2 Reinforcement in Seismic Design Category D, E or F

Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given above shall be met, in addition to the following. A minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top one-half of the pile length a minimum length of 10 feet (3048 mm) below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement provided in the pile in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three times the least pile dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of ACI 318 for other than Class E, F or liquefiable sites is permitted. Tie spacing throughout the remainder of the concrete section shall neither exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of 10 mm bars for piles with a least dimension up to 20 inches (508 mm), and 12 mm bars for larger piles.

4.5.3.3.1.3 Concrete placement

Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.

4.5.3.3.2 Enlarged base piles

Enlarged base piles shall conform to the requirements of Sections 4.5.3.3.2.1 through 4.5.3.3.2.5.

4.5.3.3.2.1 Materials

The maximum size for coarse aggregate for concrete shall be 3/4 inch (19.1 mm). Concrete to be compacted shall have a zero slump.

4.5.3.3.2.2 Allowable stresses

The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the 28-day specified compressive strength (*f'c*). Where the concrete is placed in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the 28-day specified compressive strength (*f 'c*).

4.5.3.3.2.3 Installation

Enlarged bases formed either by compacting concrete or driving a precast base shall be formed in or driven into granular soils. Piles shall be constructed in the same manner as successful prototype test piles driven for the project. Pile shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the pile shaft shall be filled sufficiently to reestablish lateral support by the soil. Where pile heave occurs, the pile shall be replaced unless it is demonstrated that the pile is undamaged and capable of carrying twice its design load.

4.5.3.3.2.4 Load-bearing capacity

Pile load-bearing capacity shall be verified by load tests in accordance with Section 4.5.3.1.7.3

4.5.3.3.2.5 Concrete cover

The minimum concrete cover shall be 21/2 inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.

4.5.3.3.3 Drilled or augered uncased piles

Drilled or augered uncased piles shall conform to Sections 4.5.3.3.3.1 through 4.5.3.3.3.5.

4.5.3.3.3.1 Allowable stresses

The allowable design stress in the concrete of drilled or augered uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (*f 'c*). The allowable compressive stress of reinforcement shall not exceed 40 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).

4.5.3.3.3.2 Dimensions

The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

> **Exception:** The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved construction documents.

4.5.3.3.3.3 Installation

Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure. Where concrete is placed by pumping through a hollow- stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in continuous increments. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place piles shall not be installed within six pile diameters center to center of a pile filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed pile drops due to installation of an adjacent pile, the pile shall be replaced.

4.5.3.3.3.4 Reinforcement

For piles installed with a hollow- stem auger where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through the hollow stem of the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

4.5.3.3.3.5 Reinforcement in Seismic Design Category C, D, E or F

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the corresponding requirements of Sections 4.5.3.3.1.2.1 and 4.5.3.3.1.2.2 shall be met.

4.5.3.3.4 Driven uncased piles

Driven uncased piles shall conform to Sections 4.5.3.3.4.1 through 4.5.3.3.4.4.

4.5.3.3.4.1 Allowable stresses

The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (*f'c*) applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.

4.5.3.3.4.2 Dimensions

The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved design.

4.5.3.3.4.3 Installation

Piles shall not be driven within six pile diameters center to center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete pile rises or drops, the pile shall be replaced. Piles shall not less than 48 hours old unless approved by the building official. If the concrete surface in any completed be installed in soils that could cause pile heave.

4.5.3.3.4.4 Concrete cover

Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.

4.5.3.3.5 Steel-cased piles

Steel-cased piles shall comply with the requirements of Sections 4.5.3.3.5.1 through 4.4.3.3.5.4.

4.5.3.3.5.1 Materials

Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 8 inches (203 mm).

4.5.3.3.5.2 Allowable stresses

The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (*f'c*). The allowable concrete compressive stress shall be 0.40 (*f'c*) for that portion of the pile meeting the conditions specified in Sections 4.5.3.3.5.2.1 through 4.5.3.3.5.2.4.

4.5.3.3.5.2.1 Shell thickness

The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage (0.068 inch) (1.75 mm) minimum.

4.5.3.3.5.2.2 Shell type

The shell shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

4.5.3.3.5.2.3 Strength

The ratio of steel yield strength (*fy*) to 28-day specified compressive strength (*f'c*) shall not be less than six.

4.5.3.3.5.2.4 Diameter

The nominal pile diameter shall not be greater than 16 inches (406 mm).

4.5.3.3.5.3 Installation

Steel shells shall be mandrel driven their full length in contact with the surrounding soil. The steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in steel shells within heave range of driving.

4.5.3.3.5.4 Reinforcement

Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell. Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

4.5.3.3.5.4.1 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the reinforcement requirements for drilled or augered uncased piles in Section 4.5.3.3.3.5 shall be met.

Exception: A spiral-welded metal casing of a thickness no less than the manufacturer's standard gage No. 14 gage [0.068 inch (1.7 mm)] is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

4.5.3.3.6 Concrete-filled steel pipe and tube piles

Concrete- filled steel pipe and tube piles shall conform to the requirements of Sections 4.5.3.3.6.1 through 4.5.3.3.6.5.

4.5.3.3.6.1 Materials

Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 4.5.3.3.1.1. The maximum coarse aggregate size shall be 3/4 inch (19.1 mm).

4.5.3.3.6.2 Allowable stresses

The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (*f c*). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (*Fy*), provided *Fy* shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.

Exception: Where justified in accordance with Section 4.5.3.1.9, the allowable stresses are permitted to be increased to 0.50 *Fy*.

4.5.3.3.6.3 Minimum dimensions

Piles shall have a nominal outside diameter of not less than 8 inches (203 mm) and a minimum wall thickness in accordance with Section 4.5.3.2.3.4. For mandrel-driven pipe piles, the minimum wall thickness shall be 1/10 inch (2.5 mm).

4.5.3.3.6.4 Reinforcement

Reinforcement steel shall conform to Section 4.5.3.1.9. Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.

4.5.3.3.6.4.1 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than 3/16 inch (5 mm).

4.5.3.3.6.5 Placing concrete

The placement of concrete shall conform to Section 4.5.3.3.1.3, but is permitted to be chuted directly into smooth-sided pipes and tubes without a centering funnel hopper.

4.5.3.3.7 Caisson piles

Caisson piles shall conform to the requirements of Sections 4.5.3.3.7.1 through 4.5.3.3.7.6.

4.5.3.3.7.1 Construction

Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.

4.5.3.3.7.2 Materials

Pipe and steel cores shall conform to the material requirements in Section 1809.3. Pipes shall have a minimum wall thickness of 3/8 inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a 28-day specified compressive strength (*f c*) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).

4.5.3.3.7.3 Design

The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The minimum outside diameter of the caisson pile shall be 18 inches (457 mm), and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.

4.5.3.3.7.4 Structural core

The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

4.5.3.3.7.5 Allowable stresses

The allowable design compressive stresses shall not exceed the following: concrete,0.33 *f'c*; steel pipe, 0.35 *Fy* and structural steel core, 0.50 *Fy*.

4.5.3.3.7.6 Installation

The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used

4.5.3.3.8 Micropiles

Micropiles shall conform to the requirements of Sections 4.5.3.3.8.1 through 4.5.3.3.8.5.

4.5.3.3.8.1 Construction

Micropiles shall consist of a grouted section reinforced with steel pipe or steel reinforcing. Micropiles shall develop their load-carrying capacity through a bond zone in soil, bedrock or a combination of soil and bedrock. The full length of the micropile shall contain either a steel pipe or steel reinforcement.

4.5.3.3.8.2 Materials

Grout shall have a 28-day specified compressive strength (*f 'c*) of not less than 4,000 psi (27.58 MPa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcement steel shall be deformed bars in accordance with ASTM A 615 Grade 60 or 75 or ASTM A 722 Grade 150. Pipe/casing shall have a minimum wall thickness of 3/16 inch (4.8 mm) and as required to meet Section 4.5.3.1.6. Pipe/casing shall meet the tensile requirements of ASTM A 252 Grade 3, except the minimum yield strength shall be as used in the design submittal [typically 50,000 psi to 80,000 psi (345 MPa to 552 MPa)] and minimum elongation shall be 15 percent.

4.5.3.3.8.3 Allowable stresses

The allowable design compressive stress on grout shall not exceed 0.33 *f'c*. The allowable design compressive stress on steel pipe and steel reinforcement shall not exceed the lesser of 0.4 *Fy*, or 32,000 psi (220 MPa). The allowable design tensile stress for steel reinforcement shall not exceed 0.60 *Fy*. The allowable design tensile stress for the cement grout shall be zero.

4.5.3.3.8.4 Reinforcement

For piles or portions of piles grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Piles or portions of piles grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe is used for reinforcement, the portion of the cement grout enclosed within the pipe is permitted to be included at the allowable stress of the grout.

4.5.3.3.8.4.1 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the pile down 120 percent times the flexural length. The flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam. Where a structure is assigned to Seismic Design Category D, E or F, the pile shall be considered as an alternative system. The alternative pile system design, supporting documentation and test data shall be submitted to the building official for review and approval.

4.5.3.3.8.5 Installation

The pile shall be permitted to be formed in a hole advanced by rotary or percussive drilling methods, with or without casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile.

The following requirements apply to specific installation methods:

1) For piles grouted inside a temporary casing, the reinforcing steel shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile to ensure that the grout completely fills the drill hole.

2) During withdrawal of the casing, the grout level inside the casing shall be monitored to check that the flow of grout inside the casing is not obstructed. For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.

3) For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
4) Subsequent piles shall not be drilled near piles that have been grouted until the grout has had sufficient time to harden.
5) Piles shall be grouted as soon as possible after drilling is completed.
6) For piles designed with casing full length, the casing must be pulled back to the top of the bond zone and reinserted or some other suitable means shall be employed to verify grout coverage outside the casing.

4.5.3.4 Composite Piles

4.5.3.4.1 General

Composite piles shall conform to the requirements of Sections 4.5.3.4.2 through 4.5.3.4.5.

4.5.3.4.2 Design

Composite piles consisting of two or more approved pile types shall be designed to meet the conditions of installation.

4.5.3.4.3 Limitation of load

The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.

4.5.3.4.4 Splices

Splices between concrete and steel or wood sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.

4.5.3.4.5 Seismic reinforcement

Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 4.5.3.3.1.2.1 and 4.5.3.3.1.2.2 or the steel section shall comply with Section 4.5.3.3.6.4.1.

4.5.3.5 Pier Foundations

4.5.3.5.1 General

Isolated and multiple piers used as foundations shall conform to the requirements of Sections 4.5.3.5.2 through 4.5.3.5.10, as well as the applicable provisions of Section 4.4.3.1.

4.5.3.5.2 Lateral dimensions and height

The minimum dimension of isolated piers used as foundations shall be 2 feet (610 mm), and the height shall not exceed 12 times the least horizontal dimension.

4.5.3.5.3 Materials

Concrete shall have a 28-day specified compressive strength (*f'c*) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pump able concrete.

4.5.3.5.4 Reinforcement

Except for steel dowels embedded 5 feet (1524 mm) or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete.

> **Exception:** Reinforcement is permitted to be wet set and the 21/2- inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for Group R-3 and U occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official. Reinforcement shall conform to the requirements of Sections 4.5.3.3.1.2.1 and 4.5.3.3.1.2.2.
>
> **Exceptions:**
>
> 1) Isolated piers supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than a minimum of one No. 4 bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.
> 2) Isolated piers supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than one No. 4 bar, without ties or spirals, when the lateral load, *E*, to the top of the pier does not exceed 200 pounds (890 N) and the soil is determined to be of adequate stiffness.
> 3) Piers supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than two No. 4 bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, *Em*, and the soil is determined to be of adequate stiffness.
> 4) Closed ties or spirals where required by Section 4.5.3.3.1.2.2 are permitted to be limited to the top 3 feet (914 mm) of the piers 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

4.5.3.5.5 Concrete placement

Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chute directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.

4.5.3.5.6 Belled bottoms

Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. When the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

4.5.3.5.7 Masonry

Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ACI 530/ASCE 5/TMS 402.

4.5.3.5.8 Concrete

Where adequate lateral support is not provided, and the unsupported height to least lateral dimension does not exceed three, piers of plain concrete shall be designed and constructed as pilasters in accordance with ACI 318. Where the unsupported height to least lateral dimension exceeds three, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in ACI 318.

> **Exception:** Where adequate lateral support is furnished by the surrounding materials as defined in Section 4.5.3.1.8, piers are permitted to be constructed of plain or reinforced concrete. The requirements of ACI 318 for bearing on concrete shall apply.

4.5.3.5.9 Steel shell

Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 4.5.3.1.17. Horizontal joints in the shell shall be spliced to comply with Section 1808.2.7.

4.5.3.5.10 Dewatering

Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.

APPENDIX A

Problematic Soils

a) Expansive Soil

Foundation materials that exhibit volume change when there are changes in their moisture content are referred to as expansive or swelling clay soils.

Typical expansive or swelling materials are highly plastic clays and clay shale that often contain colloidal clay minerals such as the montmorillonites.

Expansive soils include marls, clayey siltstones, sandstones and saprolites.

Problems that may occur in structures on expansive soils relate to the ‘differential movement of the soils (i.e., heave or settlement caused by change in soil moisture).

b) Dispersive Soil

Soils which disperse in the presence of water and can therefore be easily scoured are described as dispersive. The most predominant soil type is CLAY and SILT combinations with some amount of sand. Index properties (Atterberg limits) give no indication about this treacherous soil.

1) Dispersive soils are structurally unstable and disperse in water, back into their basic particles: sand, silt and clay.
2) Dispersible soils are highly erodible and present problems for successfully managing erosion and sedimentation.
3) Dispersion is caused by the presence of sodium.
4) The ratio of salinity (EC) to sodicity (SAR) determines the effects of salts and sodium on soils.
5) The swelling factor predicts whether sodium-induced dispersion or salinity-induced flocculation will have the greatest affect on the soil physical properties.
6) Soils are divided in accordance with the Principle of Emerson Aggregate Test into seven classes on the basis of their coherence in water with one further class being distinguished by the presence of calcium-rich minerals.
7) Determining Emerson Class Number of Aggregate

When immerse air-dry aggregates in water:

Slaking occurred after 2 hours and 20 hours.

Class-1 : Complete dispersion.

Class-2 : Some dispersion

Class-3 : Dispersion

Sub-classes for Type 2 and 3 Aggregates

(i) Slight milkiness

(ii) Obvious milkiness, < 50 % of aggregate affected

(iii) Obvious milkiness, > 50 % of aggregate affected

(iv) Total dispersion leaving only sand grains

Class-4 : No dispersion (with the presence of carbonate or gypsum)

Class-7 : No dispersion but swelling, No slaking.

Class-8 : No dispersion, No slaking, No swelling.

(i) After the preparation of 1:5 Soil:Water suspension and shaking for 10 minutes and standing for 5 minutes

Class-5 : Dispersion DP≥6

Class-6 : Complete Flocculation DP≤ 6

(Other classifications such as the Pinhole Test, SCS dispersion test (Double Hydrometer test) and Soil Chemical test are also used to assess soil dispersivity.

Slaking

When water is applied to most soils, the aggregates within the soil tend to 'melt' or break down. This process of slaking is common in most soils and results in problems such as crusting and hard setting, particularly in soils with loamy surfaces, such as the red brown earths.

In situations where the degree of slaking is considered important, a slaking subclass is allowed;

0 No change

1 Aggregate breaks open but remains intact

2 Aggregate breaks down into smaller aggregate

3 Aggregate breaks down completely into sand grains

Thixotropic

A term applied to certain types of solid/liquid systems which are effectively solid when stationary but become mobile liquids when subjected to shearing stresses.

c) Peat

Peat is a fibrous mass of organic matter in various stages of decomposition and dark brown and black in color and of spongy consistency.

d) Black Cotton Soil

It is inorganic clay of medium to high compressibility. Black Cotton Soils form a major soil group in middle parts of Myanmar. They are predominantly montmorillonitic in structure and black or blackish grey or greenish brown in color. They are characterized by high shrinkage and swelling properties.

APPENDIX B

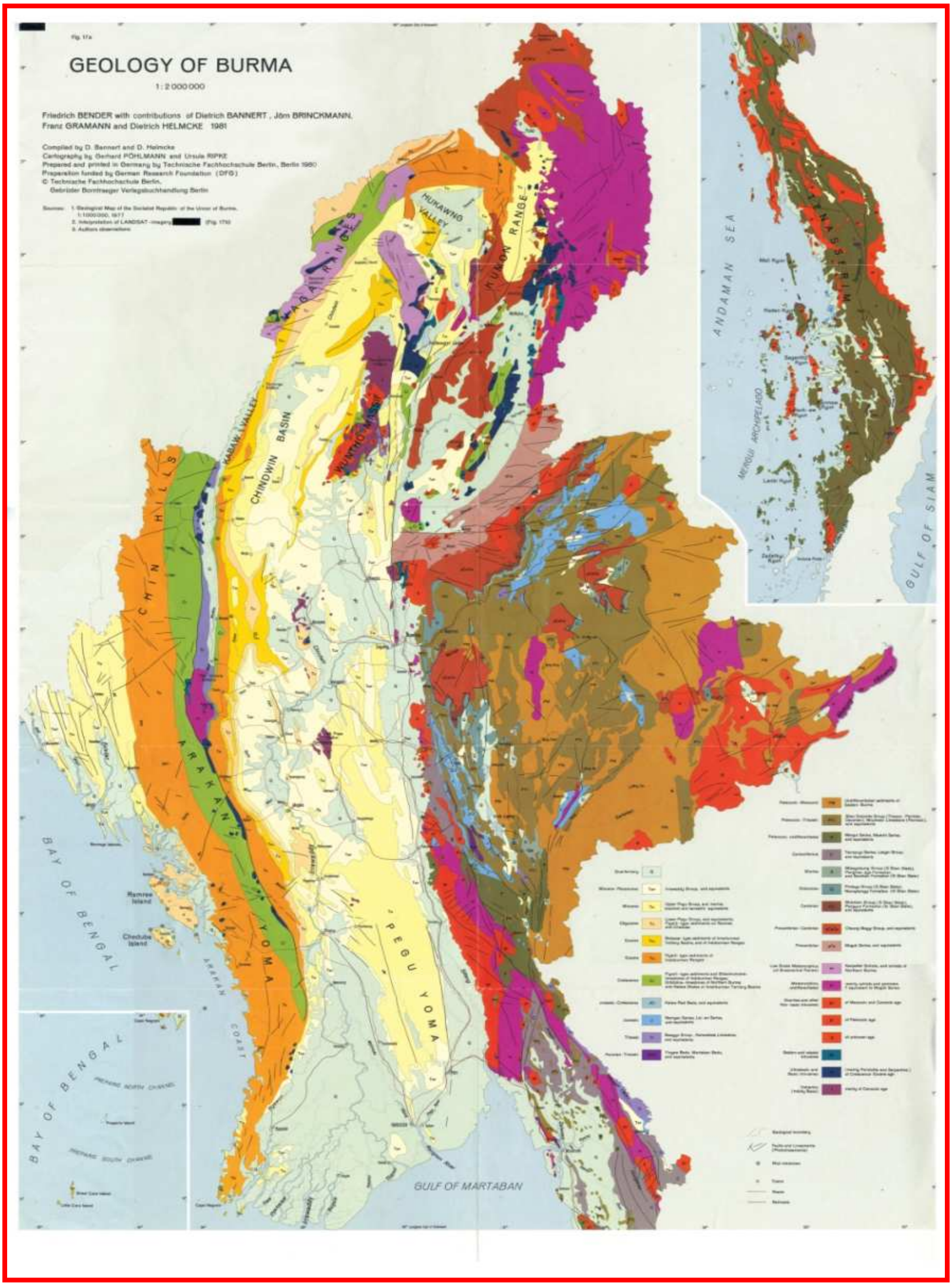

Figure B-1 Geological Map of Myanmar (Bender F., et al, 1981)

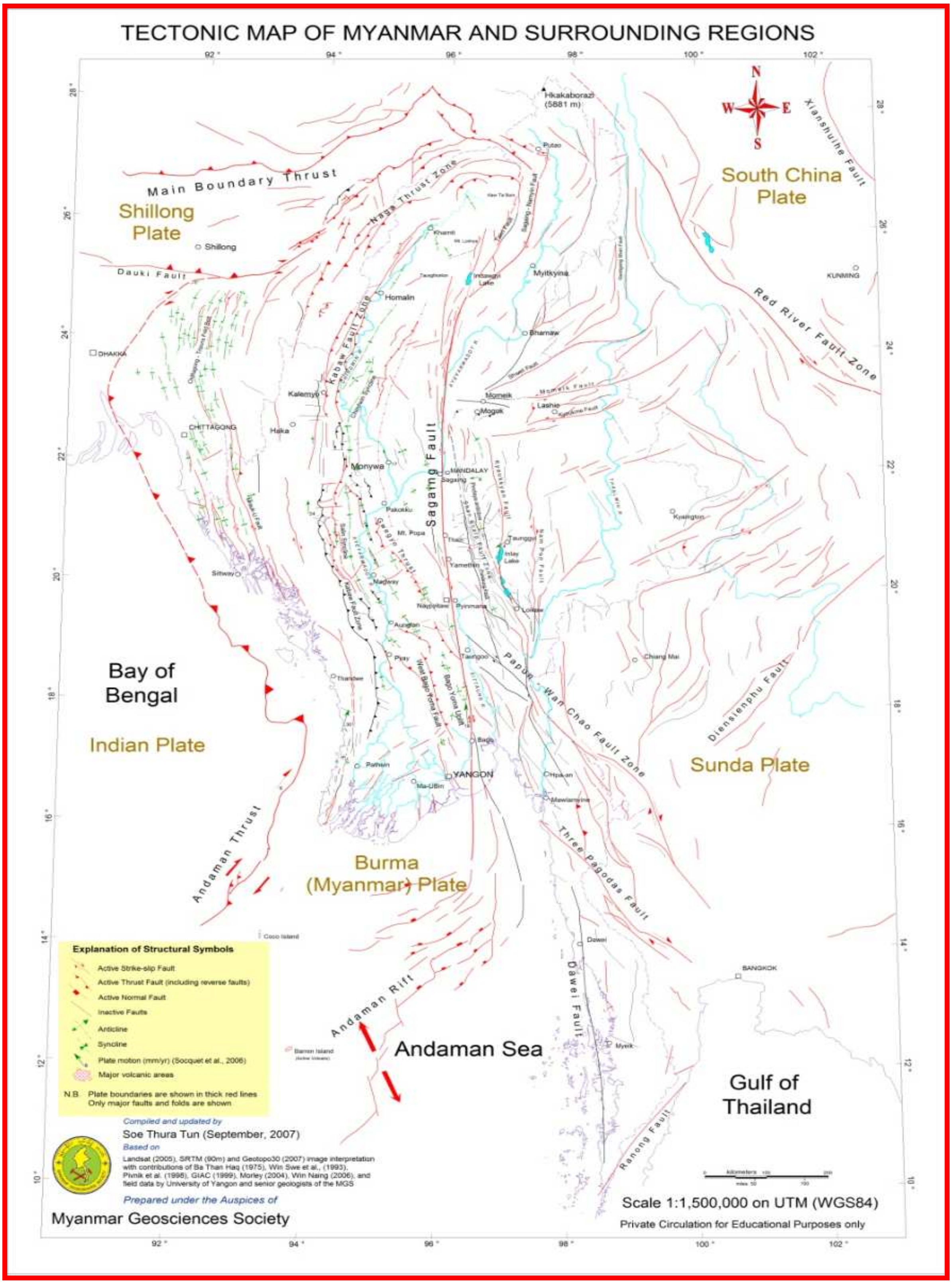

Figure B-2 Structural Geology Map of Myanmar (MGS, 2007)

APPENDIX C

Methods of Site Investigation

a) Open Trial Pits (Test Pits) Method

This method consists of excavating trial pits to expose the subsurface soil layers thereby enabling the collection of undisturbed samples from the side-walls and bottom of the pits. Unlike boring, soil can be visually observed from the walls of test pits. Both the material and mass properties of the ground within an excavation must be logged, as well as any observable lateral and vertical variations. Other information to record includes, machine type, make and model, trench or pit size, shape and orientation, bucket size and teeth type. A good photograph can also convey a substantial amount of information.

Test pitting is suitable for all types of formations, but should be used for shallow depths of investigation (up to 3 m or 10 ft.). . Safety is a major consideration in the excavation of test pits. Test pits which are excavated in soil materials or loose rock and which are more than 1.5m deep should not be entered unless the excavation is fully supported by engineer designed or specified trench shoring, mesh protection or timber support; or the sides of the excavation have been battered back to a safe angle. It is often impracticable to excavate pits or trenches in areas with groundwater levels near the surface.

Unless specifically requested by the client to do otherwise, conventional practice for backfilling of excavations is to use the machinery that dug the hole to backfill it. Care should therefore be taken to avoid excavating test pits at the exact location of future footings.

Whether test pits are used instead of shallow boreholes depends on the objectives and economics of the investigation.

Test pitting is a suitable means of investigation for low rise buildings of up to two storeys, warehouse, buildings and material surveys for road and airfield construction.

b) Auger Boring (Hand Auger Method)

Various types of hand augers can be used, depending upon the soil conditions, to obtain soil samples to a depth of approximately 30 ft. The holes are typically 0.05m to 0.2m in diameter. A hand auger system consists of an auger bit connected with a bucket type cylinder to a string of rods. .The auger may be advanced by rotating and pressing the drilling head down into the soil by means of a "T – Handle" on the upper rod. Depending on the soil characteristics there are various designs of hand augers e.g. sand augers, clay/mud augers, and augers for more typical mixed soils. .

Disturbed samples are typically collected every 2 ft interval and stored in sealed plastic bags. Undisturbed samples may also be obtained by using thin wall steel tubes of 2 inches inner diameter and 1to 2 ft in length. The recovery of soil samples by hand auger of non-cohesive materials below the water table may not be successful because of the hole's instability or loss of samples upon bit removal from the hole. The recovery of samples of dry sand material or weathered rock materials may not be possible due to the lack of cohesion. In such cases water may be added to the hole in limited amounts to provide a temporary cohesion until the samples are recovered at the surface.

Hand auger boring is a cheap method to take undisturbed and disturbed soil samples. This method may apply to shallow foundations for buildings and it is also suitable to take soil samples for highway and airfield constructions where large sample volumes are not required. This method of investigation may not be suitable in gravelly and boulder soils due to the likelihood of refusal in such soils.

c) Shell and Auger Boring

Portable power–driven helical augers (76 mm to 305 mm in diameter) are available for making deeper boreholes. The soil samples obtained from such boring are highly disturbed. In some non-cohesive soils or soils having low cohesion, the walls of boreholes will not stand unsupported. In such circumstances, a metal pipe is used as a casing to prevent the sides of the hole from caving in.

When power is available, continuous – flight augers are probably the most common method used for advancing a borehole. The power for drilling is delivered by tracked, or tractor – mounted drilling rigs. Boreholes up to 60 – 70 m in depth may easily be drilled by this method. Continuous – flight augers are available in sections of about 1 – 2 m in length with either a solid or hollow stem. Some of the commonly used solid – stem augers have outside diameters of 67 mm, 83mm, 102 mm and 114 mm. Common commercially available hollow – stem augers have dimensions of 63.5 mm ID and 158.75 mm OD, 69.85 mm ID and 177.8 OD, 76.2 mm ID and 203.2 OD, and 82.5 mm ID and 228.6 mm OD.

A cutter head (bit) is attached to the tip of the auger. Auger strings are usually fitted with one of two types of bit, the "V" bit and the tungsten carbide "TC" bit. The "V" bit usually will not penetrate competent rock and for this reason the depth to "V" bit refusal provides useful information. The "TC" bit is used for drilling in rock or to penetrate fill, concrete, boulders etc. During drilling, sections of auger can be added as the hole is extended downwards. The flight of the auger brings the loose soil from the bottom of the hole to the surface. The driller can detect changes in types of soil by noticing changes in the speed and sound of the drilling. When solid – stem augers are used, the augers must be withdrawn at regular intervals to obtain soil samples and also to conduct other operations such as standard penetration tests. Hollow – stem augers have a distinct advantage over solid – stem augers in that they do not have to be removed frequently for sampling and other tests. The outside of the hollow – stem auger acts as a casing supporting the sides of the borehole.

The hollow – stem auger system includes the following components.

Outer component: (a) hollow auger section, (b) hollow auger cap, and (c) drive cap.

Inner component: (a) pilot assembly, (b) center rod column, and (c) rod-to-cap adapter

During drilling, if soil samples are to be collected at a certain depth, the pilot assembly and the center rod are removed. The soil sampler is then inserted through the hollow stem of the auger column to the required sampling depth.

d) Wash Boring

Wash boring is another method of advancing boreholes. In this method, a casing about 6 – 10 ft long is driven into the ground at the collar of the borehole. The soil inside the casing is removed by means of a chopping bit that is attached to a drilling rod. Water is forced through the drilling rod, and it goes out at a very high velocity through the holes at the bottom of the chopping bit. The water and the chopped soil particles rise upward in the drill hole and overflow at the top of the casing through a "T" connection. The wash water is then collected in a container. The casing can be extended with additional pieces as the borehole progresses; however, such extension is not necessary if the borehole can stand without it.

e) Standard Penetration Test

Test Procedure

1) Drill a 2.5 to 8 inches (60-200 mm) diameter exploratory boring to the depth of the first test.
2) Insert the SPT sampler (also known as a split-spoon sampler) into the boring. It is connected via steel rods to a 140 lb (63.5 kg) hammer.
3) Using either a rope and cathead arrangement or an automatic tripping mechanism, raise the hammer a distance of 30 inches (760 mm) and allow it to fall. This energy drives the sampler into the bottom of the boring. Repeat this process until the sampler has penetrated a distance of 18 inches (450 mm), recording the number of hammer blows required for each 6 inches (150 mm) interval. Stop the test if more than 50 blows are required for any of the intervals, or if more than 100 total blows are required. Either of these events is known as refusal and is so noted on the boring log.
4) Compute the N-value by summing the blow counts for the last 12 inches (300 mm) of penetration. The blow count for the first 6 inches(150 mm) is retained for reference purpose, but not used to compute N because the bottom of the boring is likely to be disturbed by the drilling process and may be covered with loose soil that fell from the sides of the boring. Note that the N-value is the same regardless of whether the engineer is using English or SI units.
5) Withdraw the SPT sampler from the borehole; remove and save the soil sample.

Drill the boring to the depth of the next test and repeat steps 2 through 6 as required.

Remarks: N-values may be obtained at intervals no closer than 18 inches (450 mm).

The test results are sensitive to the variations of test procedure and poor workmanship and the principal variants are as follows:

1) Method of drilling
2) How well the bottom of the hole is cleaned before the test
3) Presence or lack of drilling mud
4) Diameter of the drill hole
5) Location of the hammer (surface type or down-hole type)
6) Type of hammer, especially whether it has a manual or automatic tripping mechanism
7) Number of turns of the rope around the cathead
8) Actual hammer drop height (manual types are often as much as 25 percent in error)
9) Mass of the anvil that the hammer strikes
10) Friction in rope guides and pulleys
11) Wear in the sampler drive shoe
12) Straightness of the drill rods
13) Presence or absence of liners inside the sampler (this seemingly small detail can alter the test results by 10-30%)
14) Rate at which the blows are applied

As the result of these variations, the following criteria should be met as a standard approach when carrying out SPT testing in Myanmar:

1) Use the rotary wash method to create a boring that has a diameter between 4 and 5 inches (100-125 mm). The drill bit should provide an upward deflection of the drilling mud (tricone or baffled drag bit).

2) If the sampler is made to accommodate liners, then these liners should be used so the inside diameter is 1.38 inches (35 mm).

3) Use A or AW size drill rods for depths less than 50 feet (15 m) and N or NW size for greater depths.

4) Use a hammer that has an efficiency of 60 %.

5) Apply the hammer blows at a rate of 30 to 40 per minute.

   Three types of hammer are recognized:

   (i) Donut Hammer

   (ii) Safety Hammer

   (iii) Automatic Hammer

6) SPT testing should not be carried out below the water table without the borehole being supported by casing or mud. Failure to do this may result in "blowing" on the bottom of the boreholes and a low SPT value recorded in the disturbed material.

Correction of SPT Test Data: Raw SPT N-value can be improved by applying the following equation.

$$N_{60} = \frac{E_m . C_B . C_S . C_R . N}{0.6}$$

Where,

N_{60} = SPT N-value corrected for field procedures

E_m = hammer efficiency

C_B = borehole diameter correction

C_S = sampler correction

C_R = rod length correction

N = measured SPT N-value

$$N'_{60} = N_{60} \times C_N$$

where,

$_{60}$ = Corrected N-value

C_N = Overburden correction factor

f) Cone Penetration Test

1) Three types of cones are commonly used: the mechanical cone, the electric cone (CPT) and the cone penetration test with pore water pressure measurement (CPTU) often referred to as the piezocone. For detailed information on the operation and interpretation of the cone penetration test see *"Cone Penetration Testing in Geotechnical Practice" by T Lunne, P.K. Robertson and J.J.M Powell*.

2) The test equipment consists of a 60° cone with $10cm^2$ base area (35.7 mm diameter) and a $150cm^2$ friction sleeve (133.7 mm long)located above the cone ($15cm^2$ cones are also being increasingly used). With the CPTu, pore water pressure is measured at typically one, two or three positions, on the cone; behind the cone; and behind the friction sleeve.

3) A hydraulic ram pushes this assembly into the ground and instruments measure the resistance to penetration on the cone tip, friction on the sleeve of the cylinder and in the case of the CPTU, pore water pressure.

4) The mechanical cone is advanced in stages and resistance to penetration is typically measured at intervals of about 20 cm. In homogeneous soils without sharp variations in cone resistance, mechanical cone data can be adequate but the quality of the data is somewhat operator dependent. The electric cone is typically advanced at a rate of about 20mm/sec and includes built-in strain gages enabling measurements to be taken continuously with depth. Most systems are set up to convert the data to digital form at selected intervals of typically 10mm to 50mm.

5) The CPT has many advantages over the SPT, but there are at least two important disadvantages:

   (i) No soil sample is recovered, so there is no opportunity to inspect the soils.

   (ii) The test is unreliable or unusable in soils with significant gravel content.

   CPT equipment is available to be operated using standard drilling rigs but due to the limited resistance force available from standard truck mounted rigsit is common to mobilize a special rig to perform the CPT. The cost per foot of penetration is less than for boring but, depending on availability, establishment costs for the special rig may be high.

g) Percussion (or) Churn Boring

1) Percussion boring is operated by air or hydraulic-driven hammer-like pistons.

2) This type of boring method consists of breaking the soil and foundation rock by a steel chisel. The chisel is attached to a steel cable which is wound onto the winch drum of the drilling rig.

3) After lifting the chisel, it falls by its weight on the ground.

4) After each blow, the chisel is turned a little so as to bore a circular hole.

5) Previously, a chisel with rods was suspended on a brake-staff which enabled the chisel to be lifted regularly.

6) In shallow boreholes, the tool can be lifted by hand and it can be worked by four to six men.

7) In firm rocks of medium hardness, not strongly jointed, flat straight-edged chisels are generally used.

8) In hard rock, the weight of the chisel is increased by a bar which is inserted between the jar and the chisel.
9) The cuttings and slurry must be removed regularly from the borehole so that percussion blows are not damped.
10) The form of the bit depends on the hardness and character of the rock.
11) Two types of bits are in general use for percussion drilling; button bits and chisel bits.
12) Button bits have a studded face, with the individual studs consisting of cylindrical inserts of tungsten carbide.
13) Chisel bits have chisel-like tungsten carbide inserts arranged in a cross-like pattern, which typically has a waterway at the center.
14) Drive sampling, using thin wall samplers are possible by using cable drill.
15) Deep percussion holes tend to present problem for the use of packers, as the bit undergoes a significant decrease in diameter during drilling.

h) Rotary Boring

Rotary drilling refers to the method of advancing a borehole with a rotary bit and with the removal of cuttings by the circulation of a fluid. It therefore does not include such rotating equipment as bucket or plate augers or continuous flight augers where the removal of cuttings is by mechanical means.

Drilling is effected by the cutting section of the rotating bit which is kept in firm contact with the face of the hole. The bit is carried on hollow jointed drill rods which are rotated by a suitable chuck. The drilling fluid, which may be water or a specially prepared mud is pumped through the hollow rods, discharged at the bit, and returns to the surface in the annular space between the rod and the sides of the hole.

The circulating fluid serves to cool and lubricate the bit and carries the cuttings to the surface.

There are two distinct types of rotary drilling; non-core drilling and core drilling.

1) **Non-core rotary drilling** is used when high rate and output is demanded as in deep oil wells. In this system, the whole bottom of the borehole is ground by a rotating bit so that only crushed rock is obtained, which is washed out by the drilling fluid. Rotary boring offers two advantages over percussion boring, it produces smooth-walled holes of uniform diameter, facilitating the use of packers; and it produces straighter holes than does conventional percussion equipment. The equipment usually consists of a relatively small air-operated drill motor, together with bits and rods. The drill motor can be operated at any one of the three rotational speeds by use of a gear shift, may be mounted on a column. Column-mounted drills advance by a crew-feed. Two types of drills can be recognized: air-track mount and column-mounted. The air-track mount is preferred because the advance of the drill is accomplished by a chain feed, and the rate of advance can be readily controlled by the driller as necessary to accommodate the rock conditions, and enables drilling to be done continuously for the full 10 feet (3 meter) length of the rod, whereas the "stroke" of the column mounted equipment is only about 2 feet (0.61 meter). Diamond bits are used in hard rock, and drag bits faced with tungsten carbide or other relatively economic materials are used in softer rocks. Non-core rotary drilling is the most effective method for penetrating relatively soft materials such as claystones, weathered sandstones and weak shale, where the waterways of percussion bits may tend to become plugged.

2) **Core rotary drilling** is usually carried out in situations where it is important to recover intact rock cores with a high percentage of core recovery to reveal defects and discontinuities such as joint opening and fillings, shear zones and cavities. In core rotary drilling the bit is designed to cut an annular hole leaving a central core which is retained in a core barrel to which the bit is attached. Core barrels used for geotechnical site investigation should at least be of “N” size (nominal hole diameter 76mm). Where weak or fractured rocks occur it is often advantageous to use larger diameter core barrels which can improve core recovery. Assuming that optimum equipment is used, it is probably the skill of the operator which is the most important element in minimizing core losses by adjusting the controls on the drilling rig to meet different rock conditions. The main variables are the rotation speed of the bit; the pressure exerted on the bit by the weight of the rods plus the feed pressure and the fluid circulation rate of flow which must be high enough to cool the bit and remove cuttings but not so high as to erode the core. Before commencing coring it is preferable to run casing into the upper surface of the rock to provide a seal for the cuttings return.

APPENDIX D

Groundwater Investigation

Water is generally collected and moves in interconnected voids, pore spaces, cracks, fissures, joints, bedding planes and other openings in soil and rock formations beneath the ground surface. The level of the water table is not stationary. It fluctuates according to the rainfall or seasons. During and after the rainy season, it gets raised considerably due to accession of water. This is called natural recharge and during the dry months, it falls. The zone between the maximum and minimum water level is called the zone of intermittent saturation. The zone below the minimum water level is called the zone of permanent saturation. The main types of flows are as follows:

1) The intermediate saturated flows above the near-surface impervious layers. The upper surface of these flows is sometimes described as perched water table.
2) A major saturated flow zone usually defined at the top of the water table by a discharge source and at the bottom by an impermeable layer.
3) An unsaturated flow zone between the surface and the water table through which water percolates or is held by capillary action.

A rough illustration of the flow zones is shown in Figure D-1

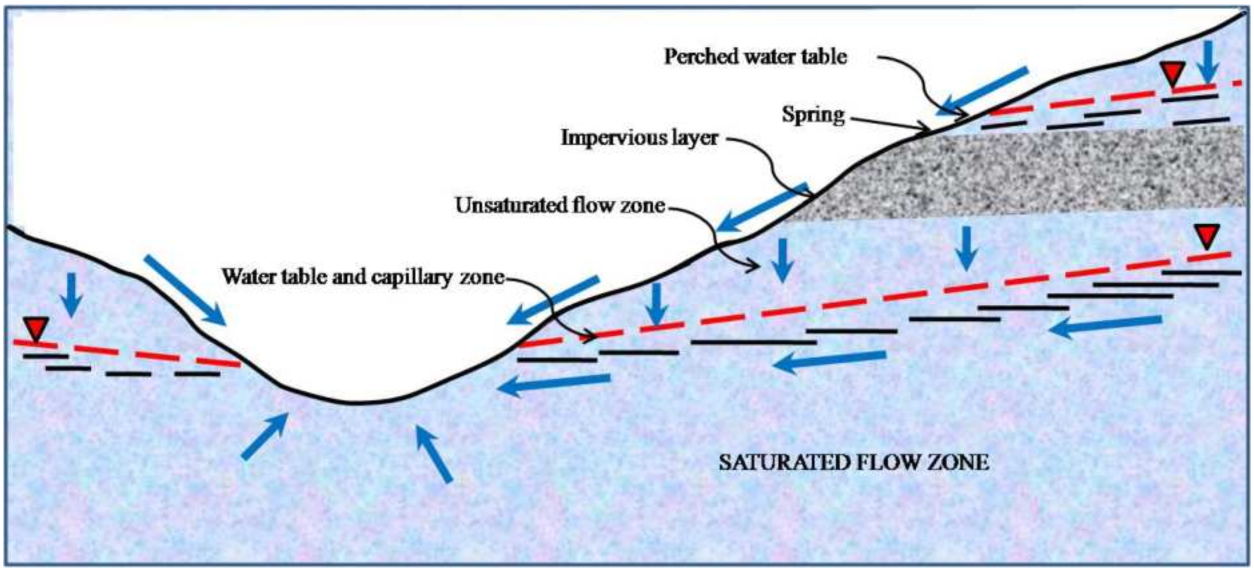

Figure D-1 Major zones of water saturation

APPENDIX E

Geotechnical Instrumentation

The primary requirement of any instrument is that it should be capable of determining a required parameter, such as water pressure, or displacement, without leading to a change in that parameter as a result of the presence of the instrument in the soil. In addition, since most soil instruments will be placed in an hostile environment, it is important that they should be robust and reliable. Most instrumentation cannot be recovered from the ground if it fails, and it will often be abused during installation or during construction of the works.

Pore water pressure and groundwater level measurement

This is the most common form of *in situ* measurement, and fortunately only one measurement is required at any point to define the regime. Quite simple devices are often used to determine water pressure in the ground, but these devices are unsuitable under many conditions.

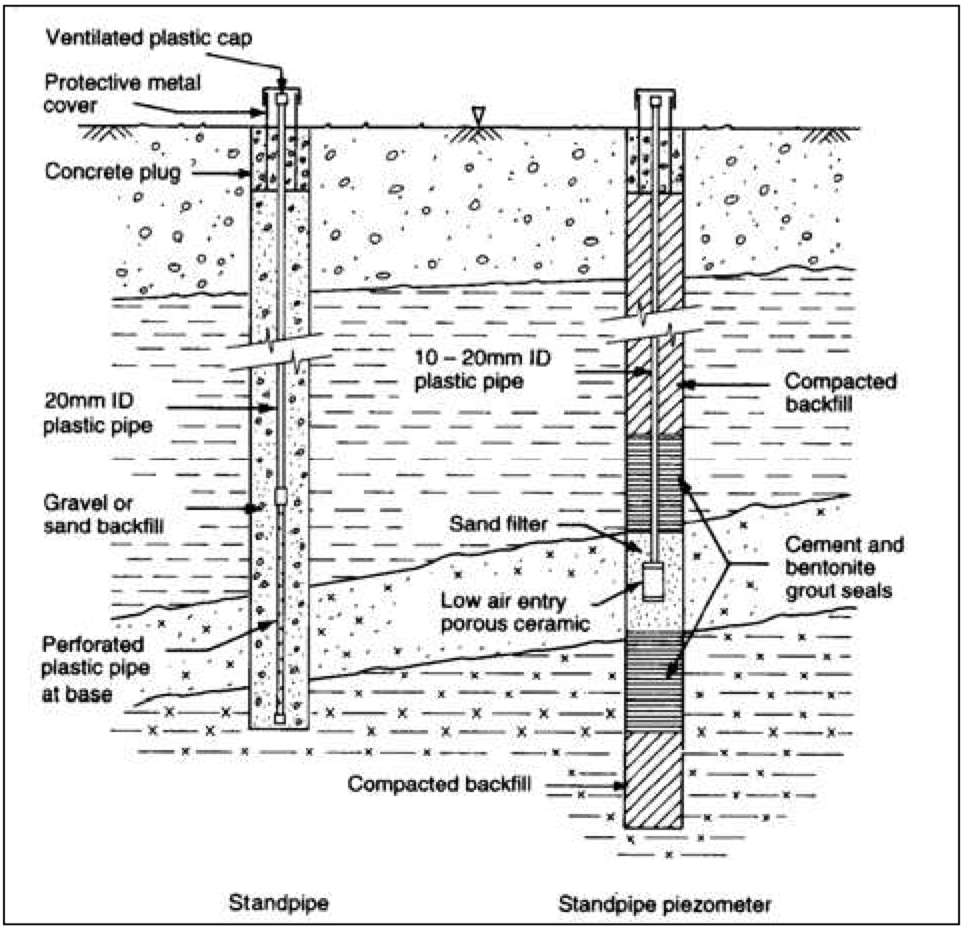

Figure E-1 Installation of standpipe and standpipe (or Casagrande) piezometer

Hanna (1973) has defined the requirements of any piezometer as:

1) to record accurately the pore pressures in the ground;
2) to cause as little interference to the natural soil as possible;
3) to be able to respond quickly to changes in groundwater conditions;
4) to be rugged and remain stable for long periods of time; and

5) to be able to read continuously or intermittently if required.

Displacement measurement

Measurements of displacement may be made relative to time, and to some datum remote from the point of measurement. A straightforward method of monitoring absolute displacement is to use conventional surveying techniques: the type of datum required for such a scheme will depend upon

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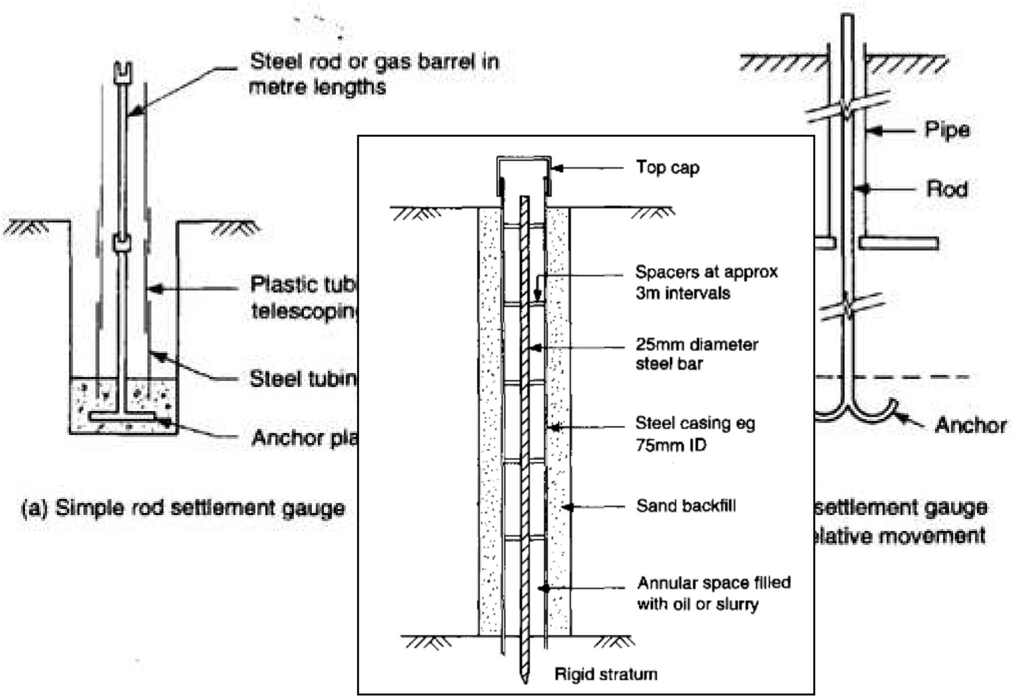

Figure E-2 Bench mark driven to bedrock.

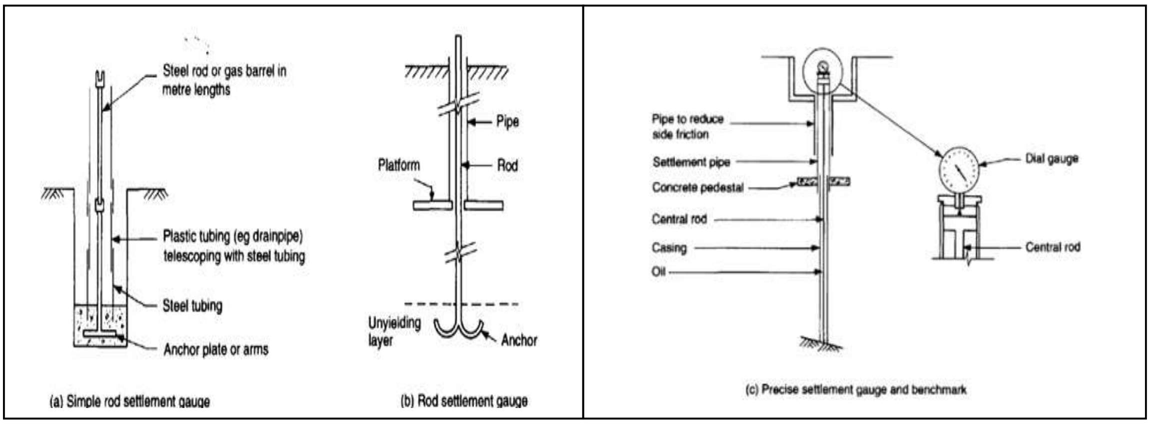

Figure E-3 Rod settlement gauges (from Bjerrum *et al.* 1965; Dunnicliff 1971; Hanna, 1973).

APPENDIX F

a) Seismic Survey

Elastic waves initiated by some energy source travel through geological media at characteristic velocities and are refracted and reflected by material changes or travel directly through the material, finally arriving at the surface where they are detected and recorded by instruments.

Seismic survey investigations are generally divided into three methods as follows:

Refraction survey method

Reflection survey method

Direct survey method

Requirements

Geophones (Vertical and Horizontal)

Energy Source (Blast or Hammer)

Seismograph (12 channels or 24 channels)

Accessories

Interpretation

The interpretation is based on the velocity values.

Seismic refraction techniques are used to measure material velocities from which depths of changes in strata are computed. Seismic reflection methods are used to obtain a schematic representation of the subsurface in terms of time and large amounts of data can be obtained rapidly over large areas. Seismic direct methods are used to obtain data on the dynamic properties of soils and rocks.

For shallow depth investigation, the refraction methods are typically used. This method is particularly valuable for reconnaissance in areas with practically unknown subsurface geology. In engineering practice, the depth to bedrock and the detection of fracture zones in hard rocks and exploration of groundwater are generally conducted by seismic refraction survey.

Table F-1 Compressional wave velocities (V_p) in various medium

Types of medium	V_p (m/s)
Air	330
Water	1400 – 1500
Ice	3000 – 4000
Permafrost	3500 – 4000
Weathered layer	250 – 1000
Alluvium, sand (dry)	300 – 1000
Sand (water saturated)	1200 – 1900
Clay	1100 – 2500
Glacial moraine	1500 – 2600
Coal	1400 – 1600

Sandstone	2000 – 4500
Slate and Shale	2400 – 5000
Limestone and Dolomite	3400 – 6000
Anhydrite	4500 – 5800
Rocksalt	4000 – 5500
Granite and Gneiss	5000 – 6200
Basalt flow top (highly fractured)	2500 – 3800
Basalt	5500 – 6300
Gabbro	6400 – 6800
Dunite	7500 – 8400

Note: For a more extensive compilation of compression and shear wave velocity data, the reader may refer to Bonner and Schock (1981).

Application depths

Up to 60 m depth by hammering

Up to 200 m depth and above by blasting

b) Resistivity Survey

Various subsurface materials have characteristic conductance for direct currents of electricity. Electrolytic action made possible by the presence of moisture and dissolved salts within the soil and rock formation permit the passage of current between the electrodes placed in the surface soils.

An electric current is transmitted into the ground and the resulting potential differences are measured at the surface using electrodes in various configurations. The following different electrode configurations are commonly used in resistivity survey.

Wenner

Gradient

Schlumberger

Pole – Dipole

Dipole – Dipole

Requirements

Resistivity meter

SAS 300 C (ID department)

SAS 4000 C (ID department)

Steel electrodes

Accessaries

Application depths

100 m to 200 m

APPENDIX G

Table G-1 Sample form of common test results

Sr. No.	SAMPLE NO.	GRAIN SIZE DISTRIBUTION				ATTERBERG'S LIMITS			SPECIFIC GRAVITY	STANDARD PROCTOR COMPACTION		DIRECT SHEAR		PERMEA-BILITY	DISPE-RSIVE	SOIL TYPE
		Clay (%)	Silt (%)	Sand (%)	Gravel (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)		OMC (%)	MDD lb/ft^3	Cohesion ($kg.cm^2$) C	Angle of internal friction, ϕ	K (cm/sec)	Crumb Test Grade	

APPENDIX H

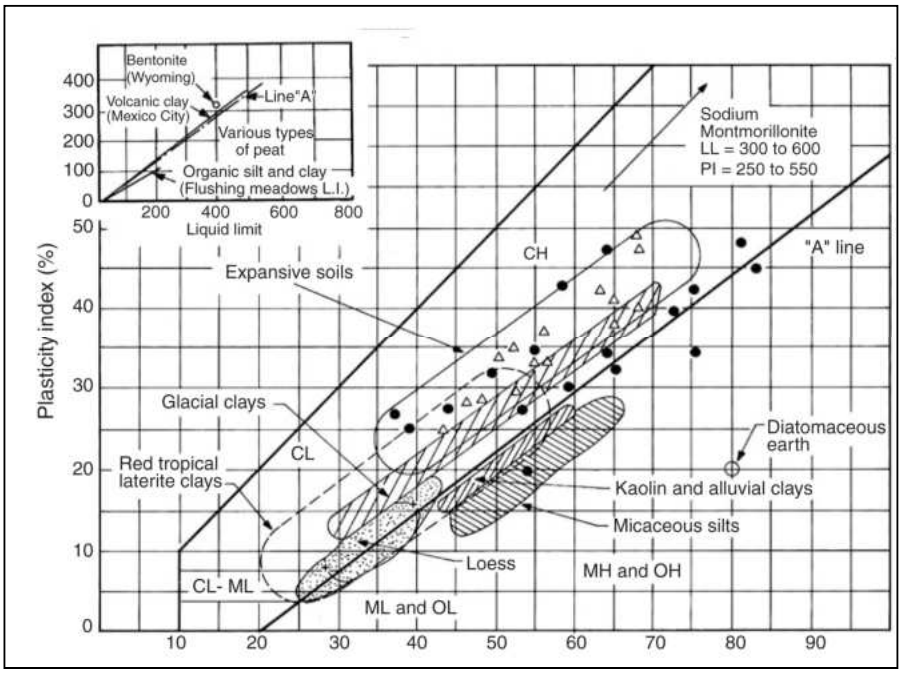

Figure H-1 Plasticity Chart

Table H-1 Unifined soil classification system and soil symbols ASTM D-2487-00

Major Divisions (1)	(2)	Letter (3)	Symbols Hatching (4)	Color (5)	Name (6)	Value for Embankments (7)	Permeability cm per sec (8)
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GP			Poorly graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GM		Yellow	Silty gravels, gravel-sand-silt mixtures	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-6}
		GC			Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}
	Sand and Sandy Soils	SW		Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$
		SP			Poorly graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$
		SM		Yellow	Silty sands, sand-silt mixtures	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-6}
		SC			Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core or flood-control structures	$k = 10^{-6}$ to 10^{-8}
Fine-Grained Soils	Silts and Clays LL < 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-6}
		CL			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}
		OL			Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}
	Silts and Clays LL $\geq$ 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor stability, core of hydraulic-fill dam, not desirable in rolled-fill construction	$k = 10^{-4}$ to 10^{-6}
		CH			Inorganic clays of high plasticity, fat clays	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-6}$ to 10^{-8}
		OH			Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}
Highly Organic Soils		Pt		Orange	Peat and other highly organic soils	Not used for construction	

APPENDIX I

General procedure of 1D seismic response analysis

1) Construction of subsurface soil model based on borehole data, SPT data and laboratory results.
2) Calculation of shear wave velocity structures of proposed site from SPT data or by measuring geophysical methods.
3) Generation of synthetic bedrock motion for the most suitable seismic sources of proposed site.
4) Performing 1D seismic response analysis by using input parameters from items 1 – 3
5) Final results will be Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), Peak Ground Displacement (PGD), amplification factor, predominant period and fundamental frequency of proposed site.

APPENDIX J

Basic Design Consideration for Potential Landslide Areas

Myanmar has two mountainous provinces: namely the Western Ranges and the Eastern Highland. These provinces are inherently unstable areas of the country. The steep slopes, unstable geologic conditions, and heavy monsoon rains combine to make these mountainous areas two of the most hazard-prone areas in Myanmar.

More recently there has been an increase in human settlement in hazard-prone areas as a result of rapid population growth, as well as improvement in accessibility by road and the onset of other infrastructure developments. Consequently, natural and man-made disasters are on the increase and each event affects people more than before. Even in central low land between the two mountainous provinces, landslide features occur along the banks of lower Ayeyarwady River and its tributaries.

The main causes that influence landslide hazard in Myanmar are: (i) gravity and the gradient of the slope, (ii) hydrogeological characteristics of the slope, (iii) presence of troublesome earth material, (iv) process of erosion, (v) man-made causes, (vi) geological conditions, and (vii) occurrence of a triggering event.

a) Geotechnical Data Collection and Testing

1) Measure the slope height and slope gradient
2) Collect disturbed and undisturbed samples of slope material
3) Measure field permeabilty, if possible and determine the ground water table
4) Identify the possible recharge sources of surface water near the slope
5) Collect the rainfall data of the area
6) Perform the following laboratory tests
   i) Sieve Analysis
   ii) Permeability Test
   iii) Direct Shear or Triaxial Test
   iv) Atterberg's Limit's Tests
   v) Specific gravity and unit weight of materials

b) Slope Stability Analysis

Various methods can be applied for slope stability analysis. One or two of them should be used according to the data available at the time of analysis. The slopes that should be analyzed may include natural slopes, cutting slopes and artificial embankments. The slope stability analysis is generally performed under the following two main analyses.

a) Limit Equilibrium Analysis
b) Stress Deformation Analysis

Some applicable methods for slope stability analysis are as follows:

1) Friction Circle Method
2) Bishop's Simplified Method of Slices
3) Newmark Sliding Block Analysis
4) Makdisi – Seed Analysis

d) Potential Landslide Hazard Zone Map of Myanmar

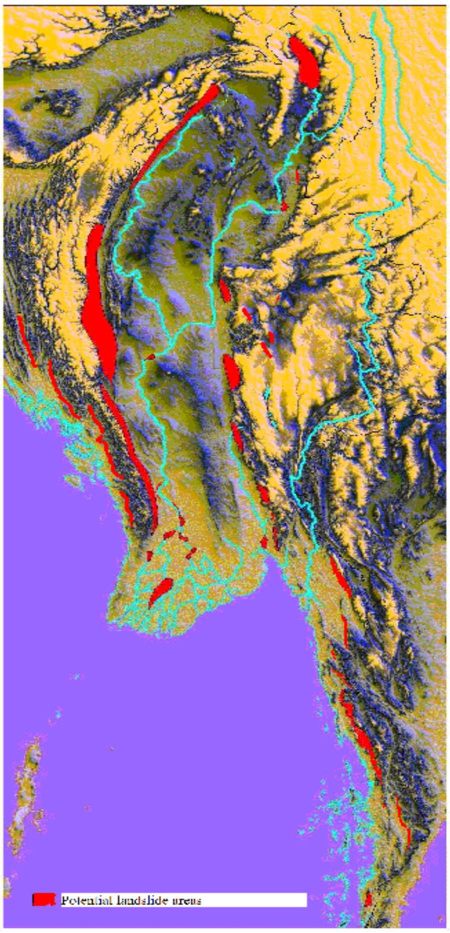

Figure 16 Preliminary Landslide Potential Map of Myanmar

Figure J-1 Potential Landslide Hazard Map of Myanmar (Kyaw Htun, 2011)

c) Stabilization and Prevention

Passive Preventive Intervention

a) Choose a safe location to build your home, away from steep lopes and places where land-slides have occurred in the past

b) Prevent deforestation and vegetation removal

c) Avoid weakening the slope

Active Preventive Intervention

a) **Reforestation:** Root systems bind materials together and plants do both prevent water percolation and take water up out of the slope. Natural vegetation should be retained where practicable.

b) **Earthworks**: Retain natural contours where possible. Large scale unsupported cuts and benching should be avoided. Cut and fill heights should be minimized and should be supported by engineer designed retaining walls or battered to an appropriate slope. Vegetation and topsoil should be stripped prior to filling and fill should be keyed into the natural slope by benching.

c) **Retaining Walls**: Should be engineer designed to resist applied soil and water forces. Walls should be founded on rock where practicable and subsurface drainage should be provided within the wall backfill and surface drainage on the slope above.

d) **Footings**: Should be founded within rock where practicable. Rows of piers or strip footings should be oriented up and down the slope and should be designed for lateral creep pressure if necessary. Footing excavations should be backfilled to prevent ingress of surface water.

e) **Proper Surface Drainage** must be ensured, especially where houses and roads have disrupted the natural flow patterns. This can be achieved by providing a proper canalization network. Drains should be provided at the tops of cut and fill slopes and should discharge to street drainage or natural water courses.

f) **Subsurface Drainage**: good ground drainage is essential to prevent saturation and consequent weakening of the soil and rock structure. Filters should be provided around subsurface drains. Drainage should always be provided when any kind of civil work, like retaining walls, have been constructed. Where possible flexible pipelines should be used with access for maintenance.